

**HAZARD ASSESSMENT OF EARTHQUAKE-INDUCED LANDSLIDES ON
NATURAL SLOPES**

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ABSTRACT

Earthquake Geotechnical Hazards are related with phenomena such as amplification of ground motion, landsliding and liquefaction. Slopes failures during earthquakes have claimed a great number of casualties and have been a major cause of damage to structures and facilities constructed on or in the vicinity of the slopes. The scale of such landslides on natural slopes can be large enough to devastate entire villages or towns. Evidences of earthquake-induced landslides and their effects were compiled and used to evaluate available methods for earthquake-induced hazard assessment and to propose a new model.

Based on historical cases, contributing factors in earthquake-induced landsliding were identified and classified as triggering and susceptibility factors. The way as these factors influence the landsliding process was discussed and current use of those values in different hazard assessment models analysed. A summary of principles, techniques and models available for landslide hazard assessment is presented, and different proposals were analysed by comparing them with the data compiled in the present project.

As starting point of the new database, a summary of historical evidences of earthquake-induced landslides, in different regions world-wide, was compiled and regional relationships between earthquakes and landslides were also presented and discussed. Two databases were compiled: a landslide-inducing earthquakes and a earthquake-induced landslides. Data compiled in these databases were used to proposed two hazard assessment model, a first model for regional hazard assessment, in terms of potential effects of seismic events on slopes, and a second model for landslide hazard assessment based on a combination of triggering and susceptibility factors to classify hazard in six qualitative categories. The proposed models were used to validate the applicability of some of the available methods.

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CHAPTER 1

INTRODUCTION

Earthquakes can cause damage, casualties and injuries population in two main ways. First there is the direct effect that the dynamic forces, created by the earthquake, have on structures and facilities. Then there are secondary effects such as liquefaction and landslides, which greatly increase the social and economical impact of an earthquake. Of these hazards, landslides are potentially the most destructive; it is well documented that over the years earthquake-induced landslides have caused thousands of deaths, huge economical losses and severe environmental changes. These landslides have occasionally been on an immense scale, such as those produced by the Alaska earthquake (1964, $M_s=8.4$), devastating large areas; however, an earthquake can also set off hundreds of small landslides in an area, each with the potential to cause damage and fatalities as was the case during the 1999 Taiwan earthquake ($M_s=7.4$).

There is awareness that population growth and urbanisation is increasing the exposure to earthquakes, and thus to earthquake-induced landslides. This is particularly so in the Third World, where land use is generally uncontrolled. It has been shown that there is a correlation between the degree of development and capacity to plan and mitigate the effects of natural hazards (Mora, 1997a). Hence the importance of developing techniques that allow the assessment of hazards related to earthquake-induced landslides.

The significant contribution of earthquake-induced landslides to natural hazards on population and environment is presented by Table 1.1, which shows a list of the most catastrophic landslides of the twentieth-century as proposed by Schuster (1996). From this list of 25 cases, which were selected based on their social and environmental impact, nine were triggered by earthquakes and two additional cases were related to volcanic activity. Most of these cases have occurred within the circum-Pacific region; Varnes (1981) noted that from the nearly 600 people per year who were killed

by slope failures from 1971 to 1974, 90% were from nations in this region, and probably most of them were caused by earthquake-induced landslides.

The principal effects of earthquake-induced landslides on population and environment and applicability of hazard analysis to reduce these effects, are presented in this Chapter.

1.1 Effects of earthquake-induced landslides

Some landslides induced by earthquakes are extremely large and have made major changes in the landscape, but have had little effect on people or structures. On the other hand, some are smaller landslides that have had only very local effect on the terrain but have been disastrous to people and homes, structures or lifelines. Still others have had major effects on the landscape as well as on the people and their belongings. Hence effects of landslides can be divided into two groups: socio-economical and environmental.

1.1.1 Socio-economical effects

Although the future location of natural hazards can be predicted in many instances, there is an inability to predict their occurrence in terms of time and intensity. Therefore, the fundamental problem of economical losses caused by natural disasters comes from their sudden occurrence and their intensity.

Floods and droughts cause the main losses to the world's food production, while earthquakes generate the largest direct losses of human lives and infra-structure damages in urban areas (UNDRO, 1991). The 1991 Valle de La Estrella earthquake in Costa Rica ($M_w=7.6$) caused the greatest damage in the history of the country (Mora, 1997a). The environmental damage produced included extensive landsliding, destruction of primary tropical rainforest, soil erosion, floods, silting of rivers and the Caribbean Sea, liquefaction, tectonic uplift and exposure of extensive areas of coral reefs. It was shown that this earthquake costs the country 8.5% of the 1991 Gross National Product (GNP) and an average loss of 2% of the GNP in the years following the earthquake. As is

shown in Table 1.2, these losses are much larger than losses produced by other effects. Similar figures can be found for other areas affected by earthquakes.

Table 1.1. Major catastrophic landslides of the 20th century. (After Schuster, 1996)

YEAR	LOCATION	NAME	TYPE	TRIGGERING PROCESS	VOLUME (m ³) (Area)
1911	Tadzhik Rep.	Usoy	Rock slide	Usoy Earthquake M=7.4	2.0 * 10 ⁹
1919	Indonesia (Java)	Kelut lahars	Volcanic mud flows	Eruption of Kelut volcano	(185 km ²)
1920	China (Ningxia)	Haiyuan landslides	Landslides	Haiyuan earthquake	?
1921	Kazakh Rep.	Alma-Ata	Debris flow	Snowmelt	?
1933	China (Sichuan)	Deixi	Landslides	Deixi earthquake M=7.5	>150 * 10 ⁶
1939	Japan (Hyogo)	Mount Rokko	Slides and mud flows	Heavy rain	?
1949	Tagzhik Rep.	Khait	Rock slide	Khait earthquake M=7.5	?
1953	Japan (Wakayama)	Arita River	Slides, debris flows, Mud flows	Heavy rain	?
1953	Japan (Kyoto)	Minamiyamashiro	Slides, debris flows, Mud flows	Heavy rain	?
1958	Japan (Shizuoka)	Kanogawa	Slide, debris flows, Mud flows	Heavy rain	?
1962	Perú (Anchas)	Nevados Huascarán	Debris avalanche	?	13 * 10 ⁶
1963	Italy (Friuli-Venezia-Gruillia)	Vaiont	Rock slide	Filling of Vaiont reservoir	250 * 10 ⁶
1964	United States (Alaska)	Alaska	Landslides	Prince William Sound earthquake M=9.4	?
1965	China (Yunnan)		Rock slide	?	450 * 10 ⁶
1966	Brazil (Rio de Janeiro)	Rio de Janeiro	Slides, avalanches, Debris flows, mud flows	Heavy rain	?
1967	Brazil (Serra das Araras)	Serra das Araras	Slides, avalanches, Debris flows, mud flows	Heavy rain	?
1970	Perú (Ancash)	Nevados Huascarán	Debris avalanche	Earthquake M=7.7	?
1974	Perú (Huanca-Velica)	Mayunmarca	Rock slide Debris avalanche	Rainfall River erosion?	1.6 * 10 ⁹
1980	United States (Washington)	Mount St. Helen	Rock slide Debris avalanche	Eruption of Mt. St. Helen volcano	2.8 * 10 ⁹
1983	United States (Utah)	Thistle	Debris slide	Snowmelt and Heavy rain	21 * 10 ⁶
1983	China (Gansu)	Saleshan	Landslide	?	35 * 10 ⁶
1985	Colombia (Tolima)	Nevado del Ruiz	Debris flow	Eruption of Nevado del Ruiz volcano	?
1986	Papua New Guinea (East New Britain)	Bairaman	Rock slide Debris avalanche	Bairaman earthquake M=7.1	200 * 10 ⁶
1987	Ecuador (Napo)	Reventador	Landslide	Reventador earthquakes M=6.1 and M=6.9	75-110 * 10 ⁶
1994	Colombia (Cauca)	Paéz	Landslides	Paéz earthquake M=6.4	(250 km ²)

The number of people severely affected by natural disasters in the world increases at a rate of 6% per year, almost double the increase of the population (UNDRO, 1991). In Costa Rica since 1950 most human losses have been caused by landslides: 18% of total deaths have been direct effect of earthquake-induced landslides as shown in Figure 1.1.

Table 1.2. Assessment of direct costs due to the 1991 Valle de La Estrella earthquake (Costa Rica). (US \$M). (Adapted from Mora 1997a)

Activity, property and services	Losses	%
Handicraft, small industry, mining	2.7	0.28
Heavy industry	2.5	0.26
Electricity and communication	1.5	0.16
Harbours and facilities	17.5	1.81
Other port facilities	8.0	0.83
Oil refinery and oil pipelines	8.0	0.83
Aqueducts, drinkable water systems, sewage	25.0	2.59
Railways, rails and bridges	10.0	1.04
Roadways	15.0	1.55
Bridges	6.0	0.62
Tourism	2.0	0.21
Agriculture	29.0	3.00
Banana crops and transport facilities	30.0	3.10
Channels	10.0	1.04
Airports	1.0	0.10
School's infrastructure	2.0	0.21
Health infrastructure	13.0	1.35
Housing	71.3	7.39
Trading, infrastructure and equipment	27.0	2.80
Direct cost for emergency activities	5.0	0.52
SUB TOTAL	285.0	29.54
	200.0	20.72
Cost of commercial wood lost from landslides in forests (20,000 Ha.)		
Cost of exploitable soils lost by landslides (20,000 Ha.)	480.0	49.74
	965.0	100.00
TOTAL		

Total deaths and those caused by landslides for some past earthquakes are presented in Figure 1.2. In some cases, death toll due to earthquake-induced landslides are extremely high, such as those produced by the 1970 Perú earthquake ($M_W=7.96$). In other cases, the entire death toll due to an earthquake has been due to earthquake-induced landslides, as was reported for the 1959 Hebgen Lake earthquake ($M_W=7.28$).

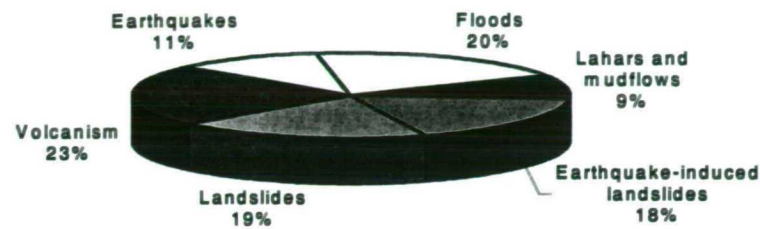


Figure 1.1 *Casualties due to Natural disasters in Costa Rica since 1950 (Based on Mora, 1997a)*

1.1.2 Environmental effects

The impact of landsliding caused by large magnitude earthquakes on the environment has been little studied. Effects on sediment yield rates and denudation over short, medium and long periods have not been sufficiently analysed. Pearce and Watson (1986) showed that a few studies, principally in tropical rainforest areas, have concluded that the pulse of sediment generated by earthquake-induced landslides is short-lived, with most of the sediments supplied to rivers flushed out of the affected drainage basin within 0.5 to 2 years. Other less detailed observations of short-lived damming or blocking of large rivers by natural dams produced by earthquake-induced landslides support this conclusion (Costa and Schuster, 1988). However, there are cases where the process takes a much longer time, as is the case of the Sarez Lake in Tadjikistan produced by the 1911 Khait earthquake ($M_w=7.25$), which still represents a hazard for more than five million people that live downstream along the valley.

Long-term hazards due to earthquake-induced landslides and their importance in slope erosion have been studied by Keefer (1994), who presents a model to determine the contribution of landslides to long-term erosion and landslide hazard. Keefer (1994) points out the importance of earthquake-induced landslides on the geomorphological development from observations of previous works (Muir, 1912; Matthes, 1930) on the Yosemite Valley (California). The conclusion is that 90% of all the talus in Sierra Nevada of California was generated by earthquakes, however, while

earthquakes were important in talus formation, most of recent talus have been the result of more gradual and ongoing processes. Further more, Keefer shows how recent studies in the same valley (Wieczoreck, *et al.*, 1992) have demonstrated that at least 12% by volume of historical landsliding was produced by earthquakes (53% of that definitely attributable to a specific cause).

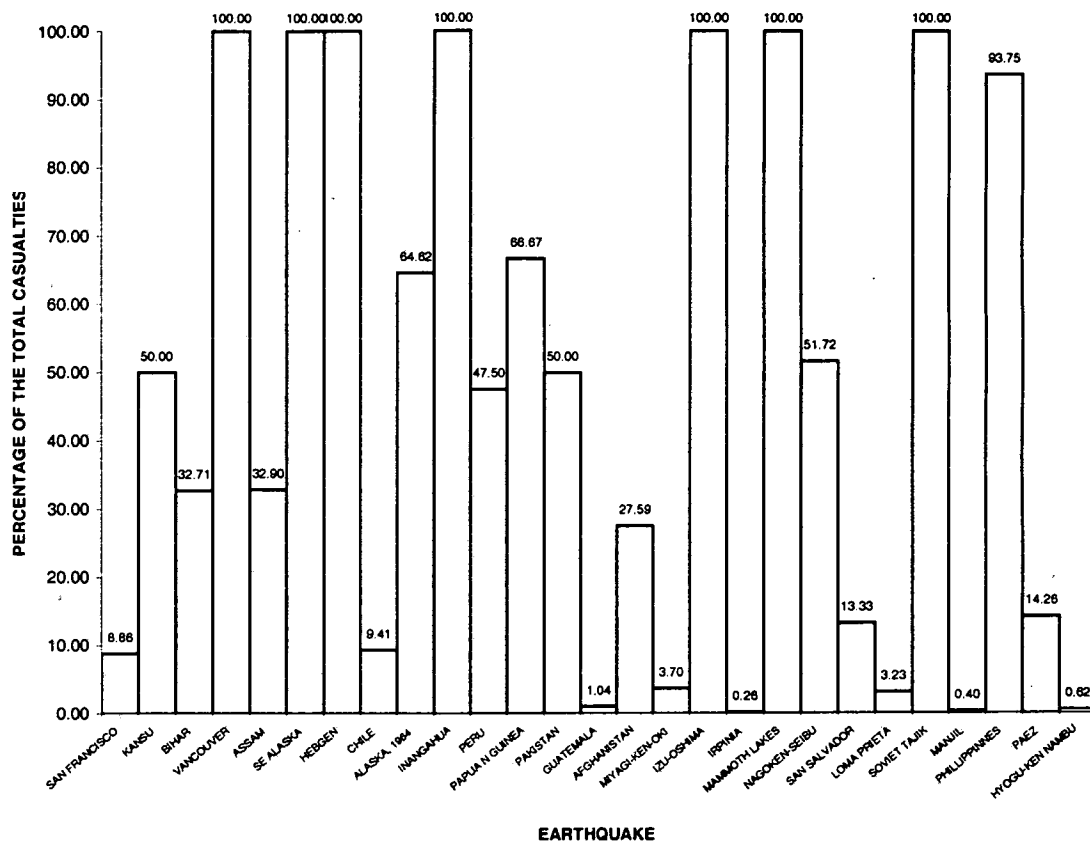


Figure 1.2 Casualties due to earthquake-induced landslides

Estimates of long-term rates of erosion from earthquake-induced landslides have also been reported for areas in New Zealand and Papua New Guinea. In New Zealand, between 40 and 67 percent of long-term erosion rates from an area of 154,000 km² on the South Island has been found to be produced by earthquake-induced landslides (Adams, 1980). Additional analyses by Pearce and Watson (1986) indicate that while earthquake-induced landslides were the main regional sediment supply mechanism, much of the landslide material probably remained in storage for long periods of time before being transported out of the river basin.

In Papua New Guinea, Simonett (1967) found that for the Torricelli Mountains, the sediment production during the 1935 earthquake ($M_w=8.07$) was about six times higher than the normal sediment production of adjacent mountain areas of low seismicity.

Comparison of erosion rates from earthquake-induced landslides to rates produced by other hillslope processes for different regions around the world are presented by Keefer (1994). These results show the significant variation among the regions in the relative contribution of earthquake-induced landslides to the overall slope erosion, with the seismically-induced processes being predominant in some regions such as San Francisco Bay and Hawaii. A relatively small contribution can be seen in other areas as Yosemite Valley (California) and Central Japan.

Mean erosion rates for earthquake-induced landslides vary from 5 to 553 $\text{m}^3/\text{km}^2\text{-yr}$. This value is slightly lower than the range of 10 to 1200 $\text{m}^3/\text{km}^2\text{-yr}$, calculated for total erosion rates on steep slopes world-wide by Young (1969). The areally weighted mean erosion rate for earthquake-induced landslides for twelve regions studied by Keefer (1994) is 56 $\text{m}^3/\text{km}^2\text{-yr}$, as shown in Figure 1.3, which is about one-ninth the world-wide mean of 500 $\text{m}^3/\text{km}^2\text{-yr}$ for all processes on steep slopes as determined by Young (1969).

The two most important processes in initiating dam-forming landslides are excessive precipitation and earthquakes, as is shown in Figure 1.4. These processes account for 90% of the case studies presented by Costa and Schuster (1988). The 1929 Buller earthquake ($M_w=7.6$) in the north-western part of the South Island in New Zealand, produced slope failures that formed at least 11 landslide-dammed lakes (Adams, 1981). The 1783 Calabrian earthquake ($M_w=7.1$) triggered mass movements that formed 215 landslide-dammed lakes (Cotecchia, 1987). In 1911, an earthquake in the former Soviet Union triggered a rock avalanche (the Usoy landslide), with a volume of 2.0 to 2.5 billion cubic metres, which dammed the Murgab River. This landslide-formed dam of about 500 m high created the Sarez lake, which, as stated previously, still constitutes a high hazard for about 5 million people living downstream along the river valley.

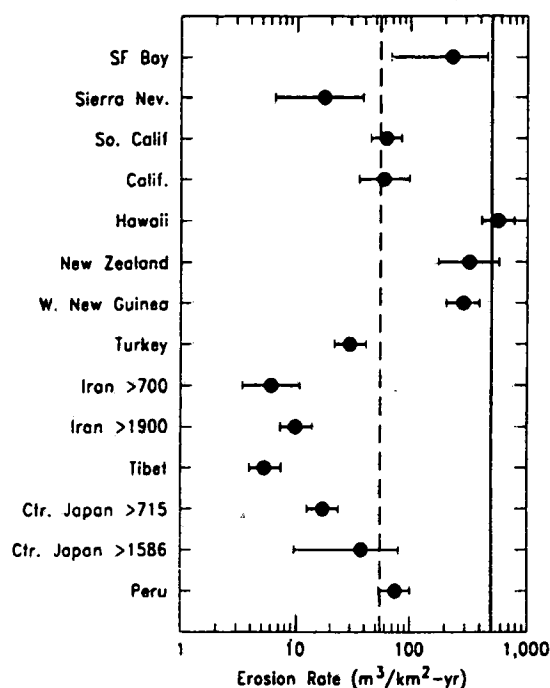


Figure 1.3 Erosion rate due to earthquake-induced landslides (After Keefer, 1994). Vertical dashed line is the mean value of erosion by earthquake-induced landslides for studied regions, and vertical solid line is the world-wide mean value of erosion due to different processes.

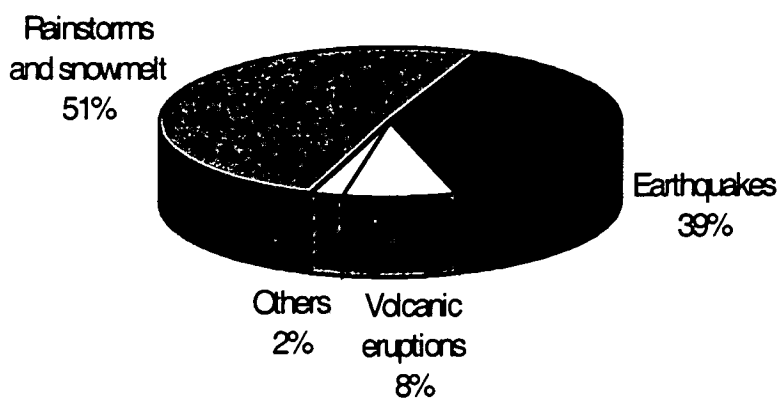


Figure 1.4 Causes of landslides that have formed natural dams (Based on Costa and Schuster, 1988)

The Tsao-Ling landslides in Central Taiwan, which dammed the Schi-chi River, as a result of an earthquake in 1862, illustrate the long-term seismic hazard due to earthquake-induced landslides (Costa and Schuster, 1988). In 1898, the natural dam failed for unknown reasons. In December 1941, a major earthquake formed another landslide dam, 140 m high, at the same

location. In August 1942, heavy rains caused the slide reactivation and the dam increased in height from 140 to 217 m. In May 1951, several days of intensive rainfall led to the overtopping and failure of the dam. Due to the subsequent flood, 154 people were killed and 564 homes damaged. On August 1979, heavy rains again reactivated the landslide, which dammed the river with a natural barrier of 90 m height. Heavy precipitation continued, and 9 days later the landslide dam was overtopped and failed causing severe flooding. Many other cases of natural dams produced by earthquake-induced landslides have been reported, some of which are presented in Table 1.3.

Extensive damage to continental slopes along seacoasts due to submarine earthquake-induced landslides has been well-documented (Seguret *et al.*, 1984; Allen, 1986; Montessus de Ballore, 1884, 1924). However, a detailed inventory and analysis of these cases have been very difficult especially when they do not present clear surficial evidence of their occurrence. Among the effects produced by this kind of landslides, it is worth noting the dramatic coastal topographic changes such as those produced during the 1964 Alaska earthquake ($M=8.5$). Other effects that have been associated with these landslides are submarine cable ruptures and the redeposition of sediments (megaturbidities), among others by the 1908 Messina Strait earthquake (Italy, $M=7.5$) and the 1929 Grand Bank earthquake in New Zealand ($M=7.2$). Cases of submarine earthquake-induced landslides are listed in Table 1.4.

Other geological features such as suckungen and sedimentary structures have been associated with earthquake origin (Jibson, 1996; Allen, 1986), although, strong evidence for this relation has not been provided yet.

1.2 Definition, purpose and applicability of landslide hazard assessment

Landslide is a collective term for most slope movements of the mass movement type. *Mass movement* is the outward or downward gravitational movement of earth material without the aid of running water as a transport agent. Skempton and Hutchinson (1969) decided that the generic term, landslide, embraces those down slope movements of soil and rock masses as a result of shear failure at the boundaries of the moving mass.

Table 1.3. Natural dams caused by earthquake-induced landslides

Earthquake	Date	M _s	River	Landslide	Natural dam description					Lake dimensions		Ref.
					Vol. (m ³)	Dam Dimensions				Vol. (m ³)		
					H (m)	L (m)	W (m)	L (m)				
China	1786	7.5	Dadu								S.W & H: 98	
Colombia	1827	7.0	Suaza					13000	2.03x10 ⁶		R. 75	
India	1841		Indus		>200			65			S.W & H: 98	
West Wairarapa	1855		Bruce						10x10 ⁶		A. 81	
San Francisco	1906	7.7	Eel Cache Creek Gualala Maacama Purisima Creek San Gregorio Creek Los Gatos Creek Hinckley Creek		30 8-10 2 20				15x10 ⁶		Y&H, 78	
Khait, Tadjikistan	1911	7.2	Murgab	Uzoy	2.5x10 ⁶	301	1000	1000	53	-	C&Sc, 88	
Arthur Pass. N. Z.	1929	7.8	Burgoo Earthquake lakes Ellis Elmer Falla Glasseys Lower Lindsay Marina Moonstone Mud Perrine Sandra Sandstone Six Mile Stanley Thompson Upper Matiri						0.15x10 ⁶ 1.0x10 ⁶ 5.0x10 ⁶ 4.0x10 ⁶ 15.0x10 ⁶ 3.5x10 ⁶ 2.5x10 ⁶ 22.0x10 ⁶ 0.3x10 ⁶ 15.0x10 ⁶ 0.5x10 ⁶ 4.0x10 ⁶ 1.6x10 ⁶ 0.1x10 ⁶ 40.0x10 ⁶ 16.0x10 ⁶ 0.2x10 ⁶		A. 81	
Sichuan, China	1933	7.3	Minjian	Deixi	150x10 ⁶	255	400	1360	17000	400x10 ⁶	C&Sc, 88	
Taiwan	1941/42	7.0	Chin-Shui	Tse-Ling	250x10 ⁶	217	1300	2000		157x10 ⁶	C&Sc, 88	
Hobgen Lake USA	1959	7.5	Madison	Madison Canyon	21x10 ⁶	60-70	500	1600	10		C&Sc, 88	
Chile	1960	8.5	San Pedro	Lake Rinihue						3x10 ⁶	W, 63	
Inangahua. N.Z	1968	7.4	Butler Downey Ram Creek Waitahiri Langanuco Lake Napeffe Safuna Alta							4.3X10 ⁶ 0.8X10 ⁶ 1.5X10 ⁶ 0.8X10 ⁶	A. 81	
Peru	1970	7.6			3x10 ⁶		600	300	480	1.3x10 ⁶	P et al., 71 C&Sc, 88	
Kinnaur, India	1975	6.8	Parachu	Parachu		60	150					
Guatemala	1976	7.5	Teculcheyá Quemayá Pibcayá Polima	San José poaquil San Martín Jilotepeque Estancia de la Virgen Rio Polima Bairaman					1000 250 800 200		H et al., 81	
Papua N. Guinea	1985	7.0	Bairaman	Bairaman		150				50x10 ⁶	S.W & H: 98	
Loma Prieta	1989	7.1	Soquel Creek (W.B) Hinckley Creek Corralitos Creek (U D) Corralitos Creek (L D) Browns Creek		50000 1800 3500 3500 200	4 4 7 3 2	10 20 10 20 8	55 15 25 20 8	180	1800 3500 6000 1300 150	K, 94	

A, 81: Adams, 1981; C&Sc, 88: Costa & Schuster, 1988; K, 94: Keefer, 1994; P et al., 71: Pfafker et al., 1971; R, 75: Ramirez, 1975; S, W & H, 98: Schuster, Weikzorek & Hope, 1998; W, 63: Wiekzorek, 1963; Y&H, 78: Youd & House, 1978

Natural slope failures in the future will most likely be in similar geologic, geomorphologic and hydrogeologic conditions that have led to past or present failures, as it was proposed by Varnes (1984). Thus, the possibility exists to estimate the style, extent, frequency of occurrence and consequences of failures that may occur in the future.

Table 1.4. Subaqueous landslides triggered by earthquakes (from Saguret et al., 1984).

Earthquake	Date	M _s	Slope failure
Strait of Messina, Italy	1908	7.5	Several slope failures on Continental slope
Chile	1922	8.3	Continental slope
Kanto, Japan	1923	8.0	Sagami Bay, 600 km ² , 10 km ³ on Continental slope
Grand Bank, Canada	1929	7.0	Slump 1700 km ³ on Continental slope
Suva, Fiji	1953	6.8	Carbonate shelf, 5 km ² , 0.15 km ³
Orleansville, USA	1954	6.8	1x100 km on Continental slope
Aegean Sea	1956	7.5	Continental slope
Alaska	1964	8.4	Several slope failures on Continental slope. Biggest: 0.075 km
New Britain Trench New Guinea	1968	6.7	Continental slope
Eureka	1980	7.2	1x20 km

The basic causes of slope instability are fairly well known from the study of cases of particular failures. Some are inherent to the rock or soil, to its composition or structure; some, like inclination of undisturbed slopes, are relatively constant; others are variable, such as groundwater level; some are transient such as seismic vibration, and some are imposed by new events, such as construction activities. In a given area most of these can be recognised and their effects rated or weighted, some can be mapped and correlated with each other and with past failures. The aim should always be to develop an understanding of the processes involved, for this permits prediction of susceptibility by extension of point or site information to larger areas.

1.2.1 Landslide hazard assessment

When conditions and processes that promote instability can be identified, it is often possible to estimate their relative contribution and give them some qualitative or semi-quantitative measure at each location. Thus, a summary of the degree of potential hazard in areas can be built up depending on the number of failure-inducing factors present, their severity and their interaction. Generally, the outcome of a hazard assessment is presented as a zoning map in which locations or zones with different levels of hazard potential are identified.

Hansen (1984) stated that the extent of landslide hazard can be divided into four components: (1) danger to life; (2) danger to structures; (3) danger to social well-being and (4) danger to land loss. The relative magnitude of these components will depend on the society at risk and on the way in which the hazard is perceived, as well as on the actual hazard process itself. In order to identify and quantify each of these components, a precise definition of the hazard characteristics such as type, location, magnitude and occurrence must be done. Different hazard characteristics lead to differences in effects over the elements at risk, hence a landslide hazard assessment must identify as precisely as possible each of these characteristics.

1.2.1.1 Mass movement classification

The type or style of landslide hazard can be defined in terms of landslide origin and mechanism. Landslides can be due to natural or man-made factors. In many cases, man-made modifications of the environment have decreased stability and allowed a natural event (i.e. precipitation or seismic events) to trigger the slope failure. On the other hand, the variety of slope movements is so great that a rigorous classification is hardly possible. Many classification systems for landslides have been proposed, however, these classifications reflect the needs of the user; if recognition is the main aim of the classification, then a great number of factors or characteristics can be included making the systems more complex. The combination of materials, slopes, and agents responsible for movements produces considerable opportunities for different slope failure types.

Numerous classifications already exist in the literature, some of which are presented in Table 1.5. The simplest classifications are based on the use of one factor, whereas more complex hierarchical and tabular groupings are based on two or more factors. This table shows the complexity of discriminating factors applied in different systems, which are denoted as main factors and secondary factors. Main factors separate types at the primary level of classification, while secondary factors are used in the subdivision of the initial categories.

Table 1.5 Comparison of classificatory factors used by different authors (After Hansen, 1984)

Author	Climate	Material moved	Coherence of material	Size of material	Geology	Type of movement	Speed of movement	Water/surface	Triggering Mechanism	Morphological attributes	Complex group
Blong 1973				o +		o x				x	o P
Coates 1977		x	+	+		x	+				A
Crozier 1973						o x				x	A
Hutchinson 1968				+		x					A
Ladd 1935					x				+		A
Sharpe 1938				+		x	+	x			A
Varnes 1978		x				x	+	+			P
Ward 1945	+	+				x		+			A
Zaruba & Menci 1969		x	x		+	+					A

x main factors; + secondary factors; o factors used in comparative classifications; P present; A absent

The most significant discriminating factors can be found in three distinct areas: type of material and/or type of movement; morphology of the material moved or of the surface of movement; and geotechnical properties.

In the present study landslide types are defined based on the Varnes' (1978), classification which is based on types of movements primarily and type of materials secondarily. Types of movement are divided into five main groups: falls, topples, slides, spreads and flows. A sixth group, complex slope movements, includes combinations of two or more of the other five types. Figure 1.5, shows schematically these main failure mechanisms. Materials are divided into two classes: rock and engineering soil; soil is further divided into debris and earth. Some of the various combinations are shown in Table 1.6. Keefer (1984) has further grouped mass movements resulting from this classification into three categories: coherent slides, disrupted slides and falls, and laterals spreads and flows. This simple classification is based on the Varnes (1978) system but additionally considers internal disruption and movement velocity, and is used in this study to discriminate hazard style.

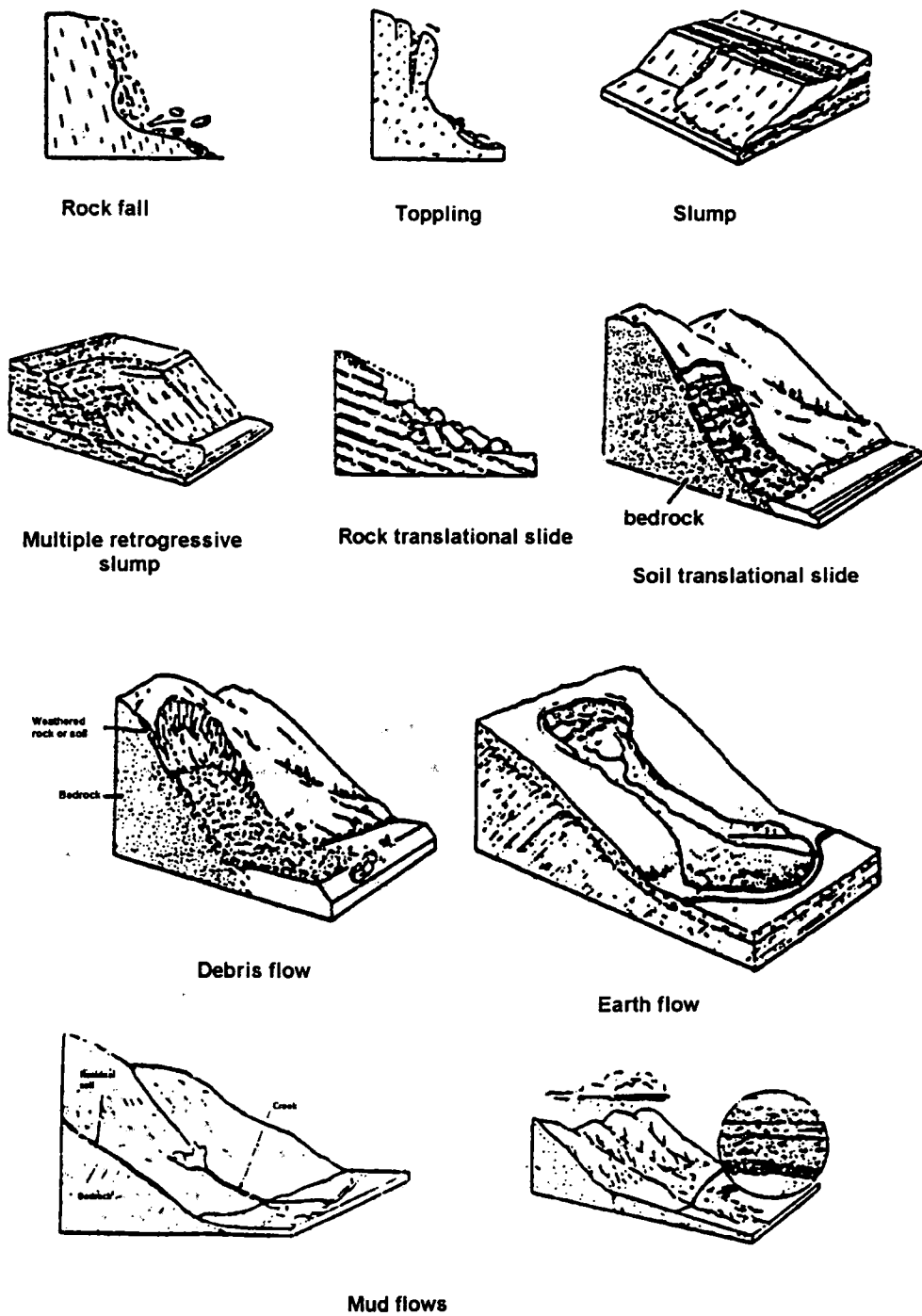


Figure 1.5 Principal failure mechanisms considered by Varnes' (1978) classification system.

Table 1.6 Varnes Mass movements classification (After Varnes, 1978)

TYPE OF MOVEMENT			TYPE OF MATERIAL		
			BEDROCK	ENGINEERING SOIL	
				COARSE	FINE
FALLS			ROCKFALL	DEBRIS FALL	EARTH FALL
TOPPLES			ROCK TOPPLE	DEBRIS TOPPLE	EARTH TOPPLE
SLIDES	ROTATIONAL	FEW UNITS	ROCK SLUMP ROCK BLOCK SLIDE	DEBRIS SLUMP DEBRIS BLOCK SLIDE	EARTH SLUMP EARTH BLOCK SLIDE
	TRANSLATIONAL	MANY UNITS	ROCK SLIDE	DEBRIS SLIDE	EARTH SLIDE
LATERAL SPREADS			ROCK SPREAD	DEBRIS SPREAD	EARTH SPREAD
FLOWS			ROCK FLOW	DEBRIS FLOW	EARTH FLOW
COMPLEX			COMBINATION OF TWO OR MORE PRINCIPAL TYPES OF MOVEMENT		

1.2.1.2 Location of potential earthquake-induced landslides

The position of a potential landslide is difficult to determine due to the great number of factors that influence the landsliding process and their interrelations. In landslide hazard analysis it is necessary to predict the starting point, however, and for risk assessment, movement path and stopping point are also needed.

Location of the starting point (area) can be determined by slope stability analysis of slopes within a given area. However, on a regional scale, where the area under study may be some tens or hundreds of square kilometres, individual slope stability analysis is not practical, thus it is necessary to define the potential of landsliding based on some slope attributes that have shown to be important in the process during past events within the area.

Recently some methods have been developed to determine potential areas of earthquake-induced landslides by applying pseudo-static analysis or Newmark's sliding block model. On a regional

scale, methods are based on defining the areal distribution of contributing factors and implementing the distributions into GIS models (ISSMFE, 1993). More simple models have also been proposed, in which case location of potential unstable slopes is defined by considering geological, geomorphological and geotechnical attributes. By comparing them with those in past events, potential instability is defined qualitatively. These techniques are described in more detail in Chapter 4.

Although there are several available models to define the position of potential slope failures induced by earthquakes, they are usually based on earthquake magnitude or macroseismic intensity, and most of them do not consider climatic conditions as a contributing factor. In the first case, variability of strong motion due to source, path and site effects is not considered, while for the latter, the prediction of intensity distribution is a difficult task.

Hazard models proposed in the present study define the location of potential slope failures by considering local (site) seismic parameters, as the triggering factor, and the influence of climatic conditions on slope susceptibility. Present models are based on the determination and weighting of the different contributing factors from past case histories. An inventory of world-wide landslide-inducing earthquakes and another of earthquake-induced landslides, are compiled and a thorough statistical analysis of contributing factors performed to define their descriptive statistical parameters. In Chapter 6 these databases are presented. As a hazard assessment model, this approach does not include the trajectory or the stopping point of landslides. In Chapters 7 and 8 a description of the models and methods used to define them are presented.

1.2.1.3 Magnitude of earthquake-induced landslides

Landslide intensity may be defined by landslide density in a homogeneous area or by one landslide characteristic such as the velocity, volume or depth. Although Hungr (1997) argues that the maximum movement velocity is the most important intensity parameter, this is not applicable in all cases due to the fact that this is a parameter that is not measured and usually is based on witness descriptions, which are very subjective.

Landslide density for earthquake-induced landslides during historical cases is not always well defined, and a detailed map of the affected area would be required, which is not commonly available. The magnitude of earthquake-induced landslide hazard can also be defined by the areal extension of landsliding or the total sediment yielding produced by a particular earthquake. It has also found that this areal extension and total volume relate to earthquake magnitude (Keefer, 1984; 1994). However, these relationships vary depending on the local climatic conditions (ISSMEF, 1993). Hence, these parameters were used to define hazard magnitude on a regional scale. Based on new data, previous relations were re-evaluated. Additionally, an extension of current relations was made in order to include climatic influences.

Some regional magnitude-area affected relationships have been published (Mora, 1997b; Maugeri *et al.*, 1994), which show that total area affected by landslides (obtained by encircling all reported cases) is much higher than the area affected by extensive landsliding (high concentration of landslides defined usually as areas with more than 60% of the area affected by landslides). It was possible to determine total area and extensive area affected by landslides for some of the cases. These data were used to define relations on a regional scale, which show that hazard scenarios obtained from world-wide databases may be over-estimated.

Shape, extension and eccentricity of the landslide area with respect to the earthquake source depend on regional slope susceptibility and directivity of the seismic-wave radiation, as was demonstrated for the 1991 Valle de La Estrella earthquake by Mora (1997b); these factors have been shown to contribute to the data scatter in the above-mentioned relationships. Particular geographical settings, as proximity to the coast, also has been identified to contribute to data scattering.

1.2.1.4 Occurrence of earthquake-induced landslides

The estimation or calculation of the return time for the triggering or reactivation of a landslide over a wide area is of particular interest in hazard analysis. This calls for inventories and statistics over a

long period and a critical in-depth analysis of the reliability and meaning of data prior to using them for correlation purposes.

Hazards, according to Hansen (1984), can be assessed in three basic ways: (1) historical methods, in which all previously recorded data about the magnitude, frequency and dates of the hazardous events are collected; (2) geomorphological analysis, where field evidence is collected about the frequency and magnitude of events; and (3) experimentation and calculation, which can provide additional quantitative information about the processes responsible for any hazardous event. Full assessment of hazard occurrence requires that more than a single method be used, and preferably all three.

Determination of the time of occurrence of earthquake-induced landslides requires correlation with all information concerning the triggering event and slope susceptibility. Current hazard analyses have assumed that landslides occur at the same time of the earthquake; however, landslides can take some time after the main shock to develop due to delays in the mechanical and hydrological response of hillslopes. This was the case during the 1980 Irpinia earthquake ($M_S=6.9$), in which slopes, particularly those on the right bank of the Sele River, took place some days after the main shock.

The intensity of the main shock possibly may not be sufficient to cause a slope failure but due to changes in mechanical properties of the material and stress conditions, susceptibility of the slope can be increased in that way that one or more smaller aftershocks can trigger a landslide. Although landslides triggered by aftershocks have been observed in many earthquakes, as was the case for the 1993 Pyrgos earthquake, $M_S=5.1$ (Papadopoulos, 1999), it is not always possible to relate landslides with their particular triggering event.

Climatic conditions, particularly rainfall, can be considered as a factor that influences the slope susceptibility. However, after an earthquake the susceptibility of slopes that did not fail increases in such a way that a new triggering event, like an intense or prolonged rainfall, can produce landslides, as was the case of landslides triggered in Nepal after the 1988 earthquake. The earthquake

influence on slope susceptibility in this case is very difficult to quantify, especially for landslides that occur a long time after the associated event.

Kobayashi *et al.* (1998) found for the Rokko Mountains, near Kobe (Japan), that landslides during the rainy season after the 1995 Hyogoken-Nanbu earthquake ($M_s=6.9$) were different from landslides during normal rainy seasons. Failures during the earthquake had a slightly denser distribution in steeper slopes than those in the post-earthquake period, and post-earthquake landslides were shown to be concentrated in slopes steeper than slopes that fail during normal rainy seasons. The author attributed these differences to the fact that ground shaking effects are stronger in convex slopes, whereas ground water effects are more pronounced in concave slopes. Furthermore, based on these observations, they concluded that post-earthquake failures also may be regarded as a kind of earthquake-induced landslides in a broad sense.

1.2.2 Purpose and applicability of earthquake-induced landslides hazard analysis

By combining the zoning map of landslide hazard with an accepted level of risk relevant to a local area, important guidance can be provided for risk prevention, control and mitigation. In cases where the risk is assessed to be high, more detailed geotechnical investigations may be required to examine the hazard assessment further and to implement specific mitigative measures or restrict land use. These measures for existing structures and facilities can be effectively prioritised based on accurate and detailed zoning maps for landslide hazard, which allows a proper assessment of the risk of future losses.

The initiation or reactivation of slope instability during earthquakes can place individuals or communities at risk. The reaction to the imposed hazard will depend on the level of awareness of the individual or community and will be based on existing knowledge of alternative responses. These include personal loss-bearing, insurance or aid programmes, which in general serve to spread the losses more evenly across the population. In areas of proposed developments, however, it will usually be economical for a community to practice prevention or avoidance schemes, such as land-use zoning, rather than to suffer the consequences of allowing uncontrolled expansion.

Landslide hazard zonation must be the base of preventive, control and mitigative measures. In general, plans should consolidate existing structures, if economically or socially feasible, either by modifying the structures or modifying the environment with both short- and long-term considerations. Much will depend on the historical development of an area: old established settlements may require the consolidation of the present population by restricting further influx, whereas planned development may be permissible in new areas.

The control of landslide hazard is easier for new developments as vulnerability can be restricted. According to Hansen (1984) a dynamic balance often exists between landslides and preventative measures as each wave of losses sustained as a result of an extreme event initiates a public demand for protection.

Disasters have both immediate and long-term implications and plans formulated for disaster-prone areas should cover both these contingencies. It should also be remembered that a disastrous occurrence may initiate a chain of severe hazards in addition to the direct impact damage, as has been seen during many earthquakes in which debris and mud flows produced after many small landslides were triggered have shown to be more destructive than the individual slides by themselves.

Hazard analysis and mapping should be carried out not only to meet the requirements of physical planning but also of sound economic and social development. Maps needed for that purposes should indicate hazard implications of each type of natural phenomenon and attempt to identify and guide the formulation of appropriate action programmes, development controls, land-use zoning regulations and especially building codes. For the respective types of hazards, these should ideally be at the micro-level. It is also necessary to provide a composite risk indicator for guiding policy decisions on development planning and macro-level land-use zoning. This kind of guidance for development planning was presented in Perú after the 1970 earthquake ($M_s=7.5$) by Lliboutry *et al.* (1978) who was in charge of helping the Peruvian authorities in relocate the villages affected by the Huascarán avalanche triggered by the earthquake. The approach to hazard analysis and mapping

should be aimed at meeting these criteria so as to provide useful guidance to planners, administrators, entrepreneurs and to the community at risk.

1.3 Framework for earthquake-induced landslides hazard assessment

Landsliding has for long been one of the potentially most predictable of geologic hazards, and properly conducted landslide hazard assessments are extremely cost effective and could result in dramatic reductions in losses. This statement is based on three basic assumptions:

1. that the uniformitarian principles are applicable, in that the conditions that led to slope instability in the past and present will apply equally well in the future. Thus, the estimation of future instability can be based on the assessment of conditions that led to slope failure in the past;
2. the main conditions that cause landsliding are known, understood and can be identified;
3. where the causes of landsliding can be identified, it is usually possible to estimate the relative significance of individual factors. This facilitates assessment of the degree of hazard by examining the number of failure-inducing mechanisms present in any area.

Landslide prediction is a spatially extensive exercise concerned with establishing different levels of threat by using the spatial distribution and magnitude-frequency characteristics of identifiable landslides together with slope stability analyses to establish probabilities of slope instability.

The level of hazard for slopes can be defined as the product of the magnitude of a triggering event and the susceptibility of the slope to sliding. This approach can be written as:

$$H_s = T \cdot S \quad (1.1)$$

where H_s is the landslide hazard, T is a function representing the triggering event, and S is a function representing the slope susceptibility.

The present model takes the seismic loads as triggering factor, and climatic conditions are included as a factor that alters the slope susceptibility, which in turn is considered dependent on geological, topographical, mechanical and anthropogenic conditions.

The magnitude of the triggering factor is defined by seismic parameters such as peak ground acceleration (PGA), Arias Intensity (AI), or other relevant parameter used for slope instability analyses. These parameters are determined from actual strong-motion records or by modelling the geologic deposit response in order to consider source, path and site effects on the strong motion to estimate the parameters at particular sites where slopes have failed.

Considering those factors that are relevant in earthquake-induced landslide processes, hazard can be expressed by a function as:

$$H_S = T(\text{source, path, site}) * S(\text{climate, geology, topography geotechnics, human action}) \quad (1.2)$$

A brief description of strong-motion parameters, their determination and factors influencing strong-motion is presented in Chapter 2. Based on this description, the methodology used for defining the distribution of strong-motion parameters is also presented. On the other hand, factors considered as part of the susceptibility function are described in Chapter 3. Emphasis is made on their influence on earthquake-induced processes.

Previous earthquake-induced landslide hazard methodologies are studied in Chapter 4. As part of the proposed hazard model, seismic hazard analysis and landslide hazard techniques are also described therein, whereas published data and new databases used to develop the model are described and presented in Chapters 5 and 6, respectively.

CHAPTER 2

EARTHQUAKES AS A LANDSLIDE TRIGGERING FACTOR

2.1 Earthquake effects as landslide triggering agents

Landslides have been related to earthquakes from remote times. One of the first recorded mass movements occurred in 372-373 BC, when Helice, then a large flourishing Greek city on the northern shores of the Peloponnese, slid into the sea after having been razed to the ground (Cotecchia, 1987; Montessus de Ballore, 1924).

The role of earthquakes in the geomorphologic history of some regions can be readily distinguished from that of other possible agents of morphogenetic slope evolution, and recently this relation has been used in palaeoseismology (Jibson, 1996). Based on relationships between area affected by landslides or maximum epicentral distance to landslides and seismic parameters, particularly magnitude, like those presented by Keefer (1984) and Rodriguez *et al.* (1999), magnitude of palaeoseismic events have been estimated and used in seismicity studies (Crozier, 1991).

With the expansion that has taken place recently in the networks of accelerographs, capable of recording even weak tremors, there is now a much broader knowledge of the relationship between earthquakes and landslides. It has thus even been possible to detect various earthquakes that have been caused by landslides. One of these cases was that occurred in Hawaii on 29 November 1975, when a mass of several thousand square kilometres slid down the side of Kilauea volcano towards the sea and was the probable cause of an earthquake with magnitude 7.2 (Cotecchia, 1987). Other cases of earthquakes caused by landslides have been reported in the literature (Radbruch-Hall and Varnes, 1976; Albini and Vogt, 1991). Sometimes the sequence of events in landslide-earthquake phenomena have been difficult to determine and no conclusive evidence has been obtained to

define which was the triggering event, such was the case in the 1929 Sierra Nevada earthquake-landslide event described by Gianella (1933).

During an earthquake mass movements may be induced by a combination of increased shear stresses and reduced in material strength due to the cyclic loads. However, landslides may also be seen along the surface expression of the fault rupture. For the former cases, inertial loads induced by the earthquake are the principal agent of slope instability, while others landslides are part of the fault rupture itself or are generated by gravitational processes along rupture scarp instead of being triggered by seismic shaking. A distinction of different mass movements related to earthquakes, their characteristics and mapping was suggested by Solonenko (1977, 1979) and is schematically presented in Figure 2.1. As can be seen in this sketch, landslides are caused also by secondary earthquakes effects like regional subsidence or uplifting; however, this classification does not consider those cases produced by processes like tsunamis, seiches or hydrological changes in hillslopes.

Seismotectonic deformations may change the slope angle in weak zones, especially within the epicentral area. From pseudostatic analysis it can be determined that the change of the slope angle by tens of seconds may reduce the factor of safety and critical acceleration, increasing the slope susceptibility. Solonenko (1977) reports cases due to this effect during the 1957 Muya earthquake ($M=7.9$) and the 1957 Gobi earthquake ($M_S=8.0$) in Mongolia.

Major earthquakes cause significant hydrogeological changes, some transient in nature, others long-lived or even permanent. The temporary changes include regional variations in ground water discharge, typically lasting a few months, caused by co-seismic volumetric strain affecting crack apertures and crustal porosity. This phenomenon was observed in Sanita Spring at Caposele (Italy) during the 1980 Irpinia earthquake, $M_S=6.9$, (Cotecchia *et al.*, 1986). In this case, the spring discharge started to increase immediately after the earthquake and reached its peak value two months later. Some delayed landslides in deposits on the right hand side of the Sele River have been related with this process (Cotecchia, 1986).

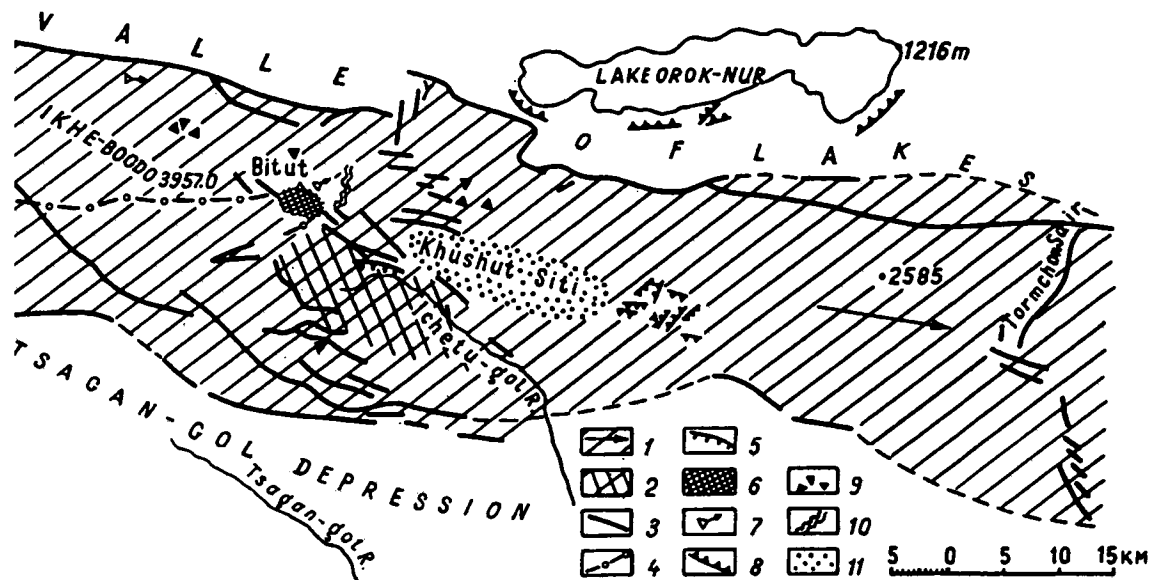


Figure 2.1 Ground deformations due to earthquakes according to Solonenko (1977, 1979). Based on effects of the 1957 Gobi-Altai earthquake ($M_S=8.0$). **Seismotectonic deformations:** (1) zonal uplift and shift of arch blocky mountain massifs; (2) graben; (3) local reverse strike-slip faults, tension fractures, fissures or retreat and toppling; (4) palaeoseismic dislocations. **Gravitational-seismotectonic deformations:** (5) fault-zone collapses; (6) gravitational-tectonic wedges. **Seismo-gravitational deformations:** (7) toppling of mountain peaks; (8) fissures; (9) major collapses. **Associated phenomena:** (10) mudflows connected with breaking of slightly dammed lakes. **Anomalous phenomena:** (11) seismic island (reduction on intensity)

Coastal deposits and river banks near their mouths have been altered by tsunami waves. Many historical cases have been reported by Silgado (1981), in which additional environmental damage along the South America coast has been produced by tsunamis, in some cases being even more important than those produced directly by the seismic shaking. Similar reports have been presented by Montessus de Ballore (1924) for the 1908 Strait of Messina earthquake ($M_S=7.5$) in Calabria (Italy), which produced extensive topographical changes along the Calabrian and Sicilian coasts, not only by direct effects, such as soil deposit liquefaction, but also by secondary effects such as the impact of tsunami waves.

Additional geomorphologic slope features have been related with seismic origin, such as sackurgen, which has been used in paleoseismicity studies (Jibson, 1996), and ground scattering, particularly on very sharp ridges, as observed during the 1978 Santa Barbara (California) earthquake ($M_S=5.7$).

Among the various effects that an earthquake can produce in hillslopes, the instability caused by increasing shear stresses and pore water pressures and reducing shear strength is the most commonly produced and reported. Although some of the effects induced by secondary agents such

as hydrological changes and tsunamis, have been recognised, they are very difficult to identify and analyse.

Some mass movements can occur due to the disturbance of equilibrium already close to instability prior to the shaking. An earthquake can accelerate a movement or reactivate slipped masses that had been previously stabilised. Mass movements on slopes close to failure can be triggered by very weak shocks. In other situations, an earthquake can initiate landslides on intact slopes depending on the susceptibility of the mass. Falls, slips, and mass movements on steep slopes, in general, are triggered even by weak shocks of short duration but high frequency, depending on the seismic response of the deposit, which in turn depends on the mechanical behaviour of the material under dynamic loading. Landslides in cohesive material may occur due to strong, long duration shocks, while even stronger events are needed to mobilise flows or complex movements such as debris avalanches produced by rock falls.

Previous relationships between earthquakes and landslides can be improved by including strong-motion parameters in the analysis. It has been usual practice to define the intensity of the ground shaking in terms of the macroseismic intensity or in terms of the earthquake magnitude and distance. However, the macroseismic intensity represents the earthquake effects, which are determined by considering human reactions and effects on ground and structures. On the other hand, the earthquake magnitude is a measure of the energy released from the source, hence it defines the source but is not an appropriate parameter to define the path and site effects, which can control the level of energy on a specific location.

Based on seismic recordings within the area affected by landslides, better relationships between earthquake parameters and landslides characteristics can be developed. In order to consider the particular conditions of each site it is imperative that strong-motion characteristics for each site be estimated taking into account source, path and site effects.

Strong-motion characteristics and those seismic parameters often used in landslide hazard analysis are described in this chapter. Source, path and site effects are also described and finally a brief

description of the methodology used to estimate these values for landslide locations is also presented.

2.2 Strong-motion parameters

Many strong-motion parameters have been used to characterise amplitude, frequency content and duration; some of these parameters represent one of these characteristics, while others represent two or three of them at the same time. Strong-motion parameters and the characteristics that they aim to represent are shown in Table 2.1. However, earthquake intensity has been commonly used to relate earthquakes and landslides (Keefer, 1984). Hence, a brief description of this parameter is made prior the review of instrumental strong-motion parameters considered during the present study.

Table 2.1 Ground Motion characteristics that are strongly reflected in various Ground Motion parameters (after Kramer, 1996)

Ground Motion Parameter	Ground Motion Characteristic		
	Amplitude	Frequency Content	Duration
Peak acceleration, PHA and PHV	X		
Peak velocity, PHV	X		
Sustained maximum acceleration, SMA	X		
Effective design acceleration, EDA	X		
Predominant period, T_p		X	
Bandwidth		X	
Central frequency, Ω		X	
Shape factor, δ		X	
Power spectrum intensity, G_0	X	X	X
Ground frequency, ω_0		X	
Ground damping, ξ_0		X	
v_{MAX}/a_{MAX}		X	
Duration, T_d			X
rms acceleration, a_{rms}	x	X	
Characteristic intensity, I_C	X	X	X
Arias intensity, I_a	X	X	X
Cumulative absolute velocity, CAV	X	X	X
Response spectrum intensity, $SI(\xi)$	X	X	
Velocity spectrum intensity, VSI	X	X	
Acceleration spectrum intensity, ASI	X	X	
Effective peak acceleration, EPA	X	X	
Effective peak velocity, EPV	X	X	

2.2.1 Earthquake Intensity

The oldest measure of earthquake size is the intensity, which is a qualitative description of the effects produced by the earthquake at a particular location, as evidenced by observed damage and human reactions at that location. Several scales have been developed and some of them have been modified to take into account local and regional characteristics and evolution of building practices. The intensity of an earthquake at any specific point depends on many variables including magnitude, epicentral distance, amplitude, frequency content, and duration of strong-motion, type of ground, geological structure, slope of the ground, groundwater conditions, type of construction, quality of workmanship and the natural vibration period of buildings and geological deposits.

A plot of reported intensities at different locations on a map allows contours of equal intensity, or *isoseismals*, to be plotted. Such a map is called an *isoseismal map*. The intensity is generally greatest in the vicinity of the earthquake epicentre and the term *epicentral intensity* is often used as a very crude description of the earthquake size. This term is not applicable in all cases as a synonym of the maximum intensity since the maximum intensity is not always around the epicentre.

Isoseismal maps show how the intensity decreases or attenuates with increasing source distance. Isoseismals usually do not follow a regular distribution due to source, path and site effects, so it is not possible to define a constant radius for an specific area that experiences the same intensity, but for analysis purposes of seismic hazard, is convenient define an equivalent radius for the area covered by an isoseismal of a specific value.

Intensity level of areas with earthquake-induced landslides have been obtained by superimposing isoseismal maps and landslide distribution. Data have been unified using the Mercalli Modified Intensity (MMI) Scale, and for those cases where isoseismals were defined in other scale, values have been converted by using the comparative scheme proposed by Reiter (1990) and shown in Figure 2.2.

MODIFIED MERCALLI	RUSSE POREL	JMA	MERCALLI CANCANI SEBERG	MEDVEDEV SPONHEUER KARNIK
I	I		II	I
II	II		III	II
III	III	I	IV	III
IV	IV	II	V	IV
V	V	III	VI	V
VI	VI	IV	VII	VI
VII	VII		VIII	VII
VIII	VIII	V	IX	VIII
IX	IX		X	IX
X		VI	XI	X
XI	X		XII	XI
XII		VII		XII

Figure 2.2 Comparison between different intensity scales (Reiter, 1990)

The most widely used intensity scales have included effects on ground surface as a parameter for defining the intensity level as can be seen in Table 2.2. However, these ground effects are strongly dependent on the previous ground susceptibility, hence they can be found in zones of lower intensity than those defined in these scales.

Although the minimum intensity level related to ground effects such as landslides for the MMI scale is VII, and VI for the New Zealand version and MSK scales, landslides have been found in lower intensity levels as can be seen in Figure 2.3 for the 1976 Guatemala earthquake. For the Japanese Meteorological Agency (JMA) Scale, intensity V is the minimum value where landslides have been considered, however; in this case landslides have also been found in lower intensity levels as was the case for the 1993 Hokkaido earthquake. These discrepancies render intensity somewhat limited as a predictive parameter of landslides.

Table 2.2 Ground effects as intensity scale factors

INTENSITY SCALE	INTENSITY GRADE	DESCRIPTION
MERCALLI MODIFIED	VII	In caving to some extent of sand or gravel stream banks.
	VIII	Ejected sand and mud in small amounts. Effects on wet ground and steep slopes.
	IX	
	X	Cracked ground especially when loose and wet up to widths of several centimetres; fissures up to a meter in width ran parallel to canal and stream banks. Landslides considerable from river banks and steep coasts. Shifted sand and mud horizontally on beaches and flat land. Serious damage to dams, dikes and embankments. Open cracks and broad wavy folds in cement pavements and asphalt road surfaces.
	XI	Many and widespread disturbances in ground, varying with ground material. Broad fissures, earth slumps, and land slips in soft, wet ground. Ejected water in large amounts charged with sand and mud. Great damage to dams, dikes and embankment for long epicentral distances.
	XII	Great and varied disturbances on ground. Numerous shear cracks. Landslides, falls of rock of significant character, slumping of river banks numerous and extensive. Wrenched loose, tore off, large rock masses. Fault slips in firm rock with notable horizontal and vertical offset displacements. Dammed lakes, produced waterfalls, deflected rivers.
MERCALLI MODIFIED NEW ZEALAND VERSION (1965)	VI	Loose material may be dislodged from existing slips, talus slopes, or shingle slides.
	VII	Small slips and caving in of sand and gravel banks.
	VIII	Cracks appear on steep slopes and in wet ground. Landslides in roadsides and unsupported excavations
	IX	Minor damage to paths and roadways. Sand and mud ejected in alluvial areas, with formation of earthquake fountains and sand craters.
	X	Dams, dykes and embankments seriously damaged. Cement and asphalt roads and pavements badly cracked or throw into waves. Large landslides on river banks and steep coasts. Sand and mud on beaches and flat land moved horizontally. Large and spectacular sand and mud fountains.
	XI	
MSK	VI	a). In few cases cracks up to widths of 1 cm. Possible in wet ground. Occasional landslips in mountains.
	VII	b). In single instances landslips of roadway on steep slopes, cracks on roads.
	VIII	c). In isolated instances parts of sandy or gravelly banks slip off.
	IX	c). Small landslips in hollows and in banked roads on steep slopes. Cracks on ground up to widths of several cms. c). On flat land overflow of water, sand and mud is often observed. Ground cracks to widths up to 10 cms; on slopes and river banks more than 10 cms, furthermore a large number of slight cracks in ground. Falls of rock, many landslides and earthflows.
	X	b). Road paving and asphalt shoe waves. c). In ground, cracks up to widths of several decimetres, sometimes up to 1 metre. Parallel to water courses occur broad fissures. Loose ground slides from steep slopes. From river banks and steep coasts considerable landslides are possible. In coastal areas displacement of sand and mud. New lakes occur.
	XI	c). Ground considerably distorted by broad cracks and fissures, as well as by movement in horizontal and vertical directions. Numerous landslips and falls of rock.
JMA	XII	The surface of the ground is radically changed. Considerably ground cracks with extensive vertical and horizontal movement are observed. Falls of rock and slumping of river banks over wide areas River dammed and lakes formed.
	V	Landslides on steep mountain are to be observed
	VI	Intensive landslides, fissures on the flat ground accompanied sometimes by spouting of mud and water in low fields.
	VII	Large fissures and fault observed.

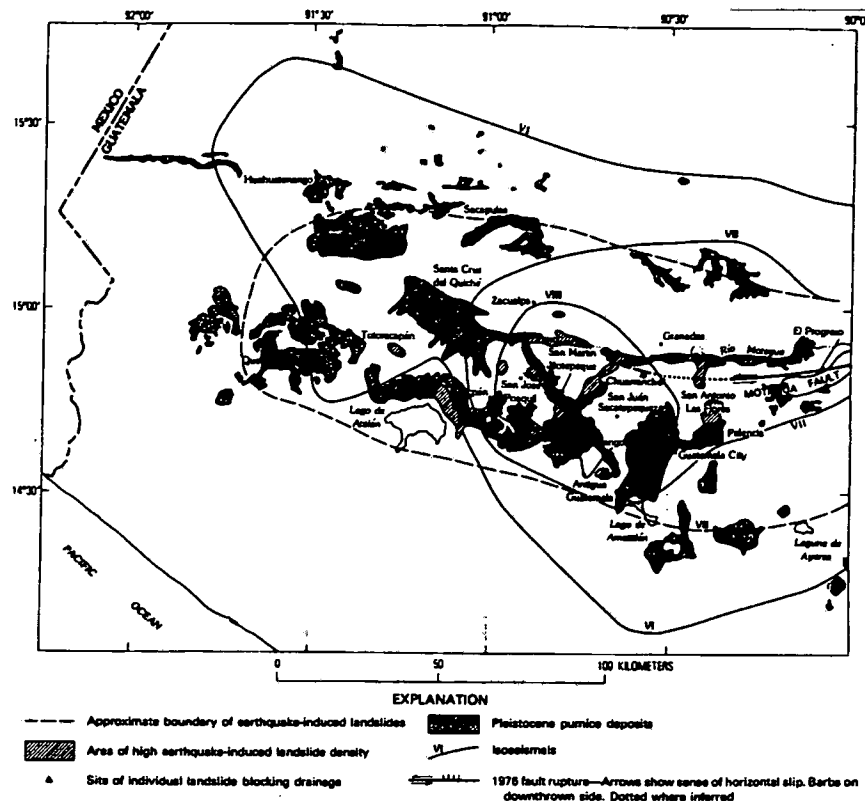


Figure 2.3 Isoseismals and landslides distribution during the 1976 Guatemala earthquake (Harp *et al.*, 1981)

Historical records, consisting of written evidence of the effects of earthquakes on humans, their civilisation and environment, have been used to determine pre-instrumental earthquakes location and size, the strength of ground motion and its frequency of occurrence (Reiter, 1990). Some studies concerned with historical seismicity have used landslides as indicative of high intensity, such as in the case of the Caribbean earthquakes listed by Robson (1964), where historical records of earthquake-induced landslides was considered as evidence of intensities IX and X. Bernard and Lambert (1988) report how Feuillard (1984) found this misinterpretation of landslides as an intensity IX factor to cause an earthquake in 1690 to be located by Robson in Guadeloupe. After re-evaluation of historical reports, this earthquake was located in French islands of Ste. Eustache and St. Christophe, and an intensity of just VI was assigned to Guadeloupe. According to DeGraff *et al.* (1989) the Caribbean islands are very susceptible to landsliding, hence landslides probably are not a clear evidence of high shaking intensity in this region, and consequently, nor of high magnitude as was considered by Robson.

Areal extension of landslides during earthquakes has also been related to areal distribution of intensity, as suggested by Rymer and White (1989). In this case authors assumed that area affected by landslides is defined by the area surrounded by isoseismal VII, however, this relation has rarely been found for case histories analysed during the present study.

2.2.2 Earthquake magnitude

Seismic instruments have allowed for an objective, quantitative measurement of earthquake size, called earthquake magnitude, to be made. Assessment of earthquake magnitude is based on some measures of ground shaking. Different magnitude scales have been developed depending on which characteristic of the ground motion is measured.

Richter (1935) first introduced the magnitude scale called the local magnitude, M_L , for shallow earthquakes in southern California. This M_L scale is determined from the maximum amplitude, A , in microns, on the Wood-Anderson seismograph (natural period $T_0=0.8$ s, static magnification $V=2800$, damping ratio $\beta=0.8$) at an epicentral distance, Δ , smaller than 1.000 km. This scale is the best known magnitude scale, but not always is the most appropriate for description of earthquake size (Kramer, 1996).

Gutenberg (1945a) extended the magnitude scale to seismograms from shallow world-wide earthquakes recorded at distant sites ($15^\circ < \Delta < 130^\circ$), and defined it as surface wave magnitude, M_S . At large epicentral distances, body waves have usually attenuated and scattered sufficiently that the resulting motion is dominated by surface waves. The M_S scale is based on the amplitude of Raleigh or Love.

Gutenberg also used teleseismic body waves for shallow shocks (Gutenberg, 1945b) as well as for deep ones (Gutenberg, 1945c) to define another scale called body wave magnitude, m_b . For deep-focus earthquakes, surface waves are often too small to permit reliable evaluation of the surface wave magnitude. The m_b scale is based on the amplitude of the few first cycles of P-waves or S-wave with period usually around 1 s.

In general, the shorter the period at which the scale is defined the most slowly slower the scale increases with the increase of the seismic moment (Inoue *et al.*, 1993). For this reason, the surface wave magnitude is the most appropriate scale to measure the size of intermediate and large events. However, for large earthquakes, seismic wave magnitude scale is saturated as shown in Figure 2.4. To avoid this saturation, Kanamori (1977) and Hanks and Kanamori (1979) proposed a new scale called the moment magnitude, M_w , defined in terms of the seismic moment, which is a direct measure of the factors that produce rupture along the fault.

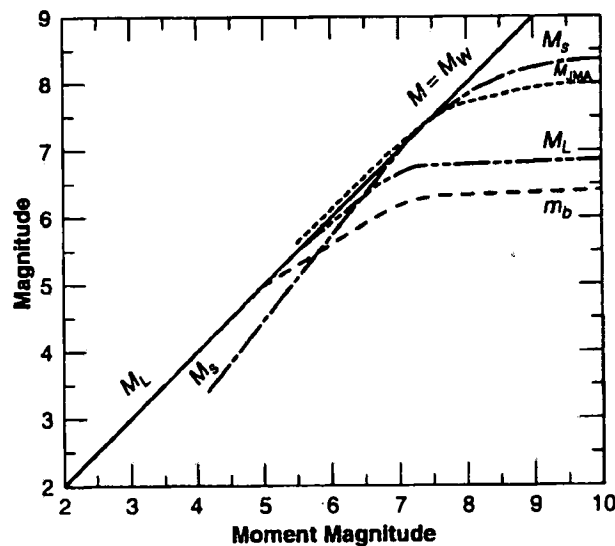


Figure 2.4 Saturation of various magnitude scales (Idriss, 1985)

When a fault breaks suddenly, much of the energy is released in the form of stress waves. Since the energy released by earthquakes is shown to correlate with magnitude, seismic waves characteristics are expected to correlate with magnitude or seismic moment. However, seismic wave characteristics are also influenced by waves scattering along the travel path and by the seismic site response. Hence, strong motion records of two earthquakes with similar magnitude may have very different characteristics as it is shown for events A and B in Figure 2.5. Furthermore, earthquakes from the same source recorded at similar distances may show different strong-motion characteristics as a function of the magnitude as is shown for the different events in Figure 2.5.

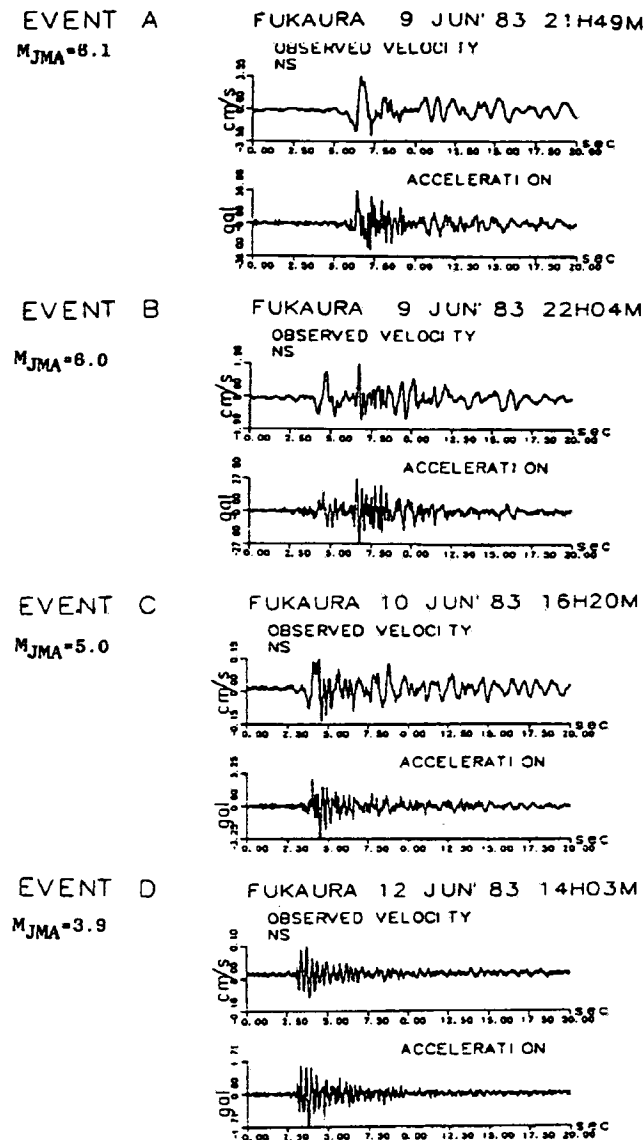


Figure 2.5 Velocity and Acceleration records for shocks of the 1983 Akita-Oki earthquake. Records are for the same source and recorded at the same station. (Irikura and Iwata, 1993).

Earthquake magnitude has been used as predictive parameter of landslide occurrence, as proposed by Keefer (1984). However, magnitude is a parameter that defines the earthquake source size, hence when used as predictive parameter, a uniform slope susceptibility around the affected area is assumed.

Minimum magnitude values have been defined for each landslide type and are presented in Chapter 5. However, as landslides can be developed even under static conditions, minimum values have no significance as landslide predictive parameters. Low magnitudes may be related to failure of very

susceptible slopes as was the case of the 1984 Ledu earthquake, $M_S=2.9$, in China, which with this small magnitude was able to trigger landslides on a aeolian cliff deposit with a slope of 50-60° (Feng and Guo, 1985). On the other hand, low magnitude aftershocks can trigger landslides on susceptible slopes that have been degraded by the influence of major previous shocks. An example of this circumstance was reported by Papadopoulos (1999), who witnessed a landslide in Pyrgos (Greece) due to a $M_S=3.8$ aftershock of the 1993 Pyrgos earthquake, $M_S=5.1$.

Relationships between the extent of the area affected by landsliding and magnitude show a considerable scatter, and even events with the same magnitude produce different affected areas mainly because the exposed area and slope susceptibility are different. Exposed areas for off-shore earthquakes have been shown to be smaller than those for inland shocks. On the other hand, multiple shock events have shown to affect larger areas than single shocks. This can be interpreted as due to the change in slope susceptibility by previous shocks. Mora (1997b) has suggested that the shape, extension and eccentricity of this area with respect to the earthquake source depends on regional slope susceptibility and directivity of the seismic radiation.

2.2.3 Peak Ground Acceleration (PGA)

The peak ground acceleration (PGA) for a given component (horizontal or vertical) is the largest absolute value read off directly from an accelerogram. The peak ground horizontal acceleration (PGHA) is probably the most widely used parameter to characterise ground motion. Physically, PGHA is related to a sudden rupture over a small portion of the causative fault. PGA, although useful, provides no information about frequency content or duration of the strong shaking part of the accelerogram, therefore, its use must be accompanied by some other parameters to give a better indication of the ground motion properties. Figure 2.6 shows the inadequacy of PGA to characterise ground motion, as many records can attain the same level of maximum amplitude but be very different in duration and frequency content.

PGHA has been widely used in pseudo-static slope stability analyses, which consider the effect of the earthquake by applying a horizontal inertial force, given by the product of PGHA and material

mass. In this case failure is defined by a factor of safety (F.S) and the PGHA value required to produce a F.S of 1 is defined as the critical acceleration, a_c . However, a F.S less than unity does not always mean that a failure of the slope has been reached because the exceedence of the critical acceleration could occur only for a very short lapse of time, which may not allow the generation of large displacements required to induce the material failure.

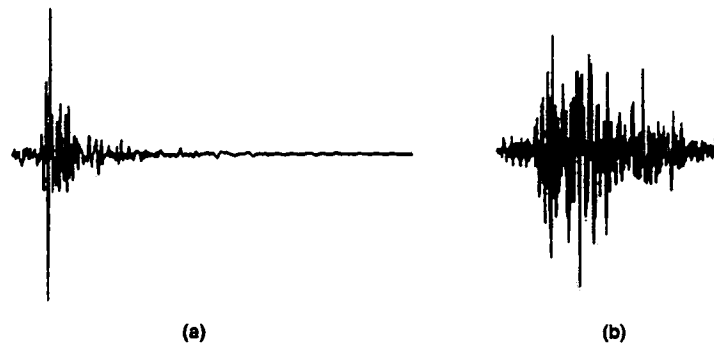


Figure 2.6 Accelerograms from (a) the N29W Melendy Ranch record of the 1972 Stone Canyon ($M=4.6$) earthquake and (b) the longitudinal record from the 1967 Konya ($M=6.5$) earthquake. Time and acceleration scales are identical for both records. (Kramer, 1996).

Some expressions have been proposed to determine the a_c (Newmark, 1965; Ishihara, 1985; Sarma, 1999). However, these expressions are based on pre-established failure surfaces, either planar or log spiral, and a rigid body block. Critical accelerations obtained from back-analyses of actual earthquake-induced landslides have been shown to be smaller than those obtained from empirical relationships, as shown in Figure 2.7. This difference can be explained by the influence of pore water pressure and internal disruption observed in actual cases. Seismic loads induced during an earthquake not only alter the stress state but also produce a change in material properties. Pseudo-static analyses usually do not consider changes in pore water pressures nor changes in material properties, which commonly leads to predicting critical accelerations higher than the actual values.

Pseudo-static stability analyses, furthermore, are based on considering an uniform acceleration within the slide body, but the acceleration has been shown to vary markedly along the slope surface (Sitar and Clough, 1983), and with depth (Arkhippova, 1986). Arkhippova showed that the horizontal acceleration is a maximum near the surface approximately in the middle of the slope, and may be

higher than the average critical value obtained by pseudo-static analyses. Hence, although a pseudo-static analysis gives a F.S higher than 1, a local F.S less than 1 may be induced in the slope and local failure occur, which may lead to subsequent general failure.

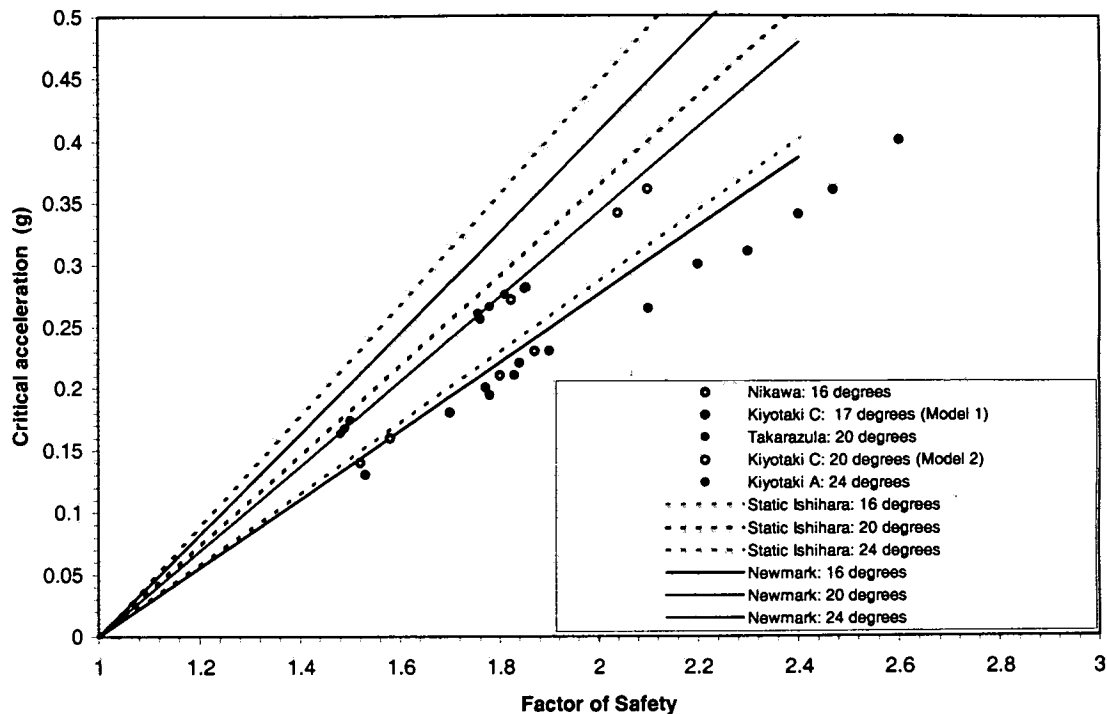


Figure 2.7 Critical acceleration as a function of Static Factor of Safety. Solid and dashed lines represent values obtained by the Newmark and Ishihara equations, respectively; points are from back-analyses of some Japanese earthquake-induced landslides (from Charalambos, 1999).

Some earthquake-induced landslides mechanisms, such as soil and rockfalls, seem to be controlled not by changes in shear stresses but by increases in tensional stresses. In these cases, pseudo-static analyses are not applicable.

Peak ground vertical acceleration (PGVA) generally receives less attention than PGHA, but it can play an important role in slope stability. High values of PGVA are associated with distances close to the seismic source (Martínez-Pereira, 1999). On the other hand, dynamic response of a slope may lead to high and non-uniform vertical accelerations, even comparable in magnitude to horizontal accelerations as shown by Arkhipova (1986). Considering vertical accelerations, which decreases

the normal effective stresses, a pseudo-static analyses leads to lower a_c , as can be seen from expressions such as that presented by Kramer (1996).

2.2.4 Peak Ground Velocity (PGV) and Peak Ground Displacement (PGD)

Ground velocity and displacement are usually obtained by direct integration of the acceleration trace, which makes them sensitive both to noise content and the filtering process of the accelerogram. Velocity is an important parameter due to its direct relation to kinetic energy carried by the earthquake signal. It is also recognised that velocity is less sensitive to higher frequency components of ground motion than acceleration, making it therefore more suitable for characterising earthquake motion at intermediate frequencies (Martínez-Pereira, 1999).

Ground displacement gives an indication of low-frequency energy contained in the ground motion, due to the smoothing from double integration of the acceleration time-history. However, special care should be taken with ground displacement as it is highly affected by the noise and the filtering techniques.

PGV and PGD have not been used directly for slope stability analysis, although, O'Rourke *et al.* (1999) correlated pipeline failures during the 1994 Northridge earthquake with PGA and PGV. They concluded that PGA correlates better than PGV with damage due to the fact that most of the failures were produced by landslides, which, according to these authors, correlate better with ground acceleration than velocity.

2.2.5 Arias Intensity (AI)

Arias (1970) proposed a measure of earthquake intensity based on instrumental records. This parameter represents the amount of energy per unit mass dissipated by single-degree-of-freedom oscillators whose frequencies are uniformly distributed from 0 to ∞ . This AI is obtained by:

$$AI_i = \int_0^\infty \int_0^\infty a_i(t) \frac{dx_i}{dt}(t) dt \cdot dw = \frac{\cos^{-1} \xi}{g\sqrt{1-\xi^2}} \int_0^{T_0} [a_i(t)]^2 dt \quad (2.1)$$

where the sub-index i denotes the direction for which the intensity is calculated, ξ is the damping ratio, g is the acceleration gravity, T_0 is the total length of the accelerogram and a is the ground acceleration.

It follows from Equation (2.1) that if one considers small values of damping ratios ($\xi \approx 2-5\%$), then AI can be approximated as:

$$AI_i = \frac{\pi}{2g} \int_0^{T_0} a_i^2(t) dt \quad (2.2)$$

The total intensity at a site can be expressed as the algebraic summation of individual intensities in each direction, as follows:

$$AI = AI_x + AI_y + AI_z = \frac{\pi}{2g} \int_0^{T_0} (a_x^2(t) + a_y^2(t) + a_z^2(t)) dt \quad (2.3)$$

where x , y and z denote the planar coordinate axes. A plot of the build up of AI is known as *Husid plot*.

AI has been found to correlate well with the distribution of earthquake-induced landslides as shown by Harp and Wilson (1995). From this analysis two shaking thresholds for the limits of falls and slides triggered during the 1987 Whittier Narrows and 1987 Superstition Hills earthquakes were established. A range of 0.08 to 0.6 m/s was observed for Tertiary and younger deposits, while the range of 0.01 to 0.07 m/s was observed for Mesozoic and older deposits. According to Harp and Wilson, these limits were controlled by the relative degree of fracturing and the aperture of fracture systems.

Jibson and Keefer (1993) based their study on 11 strong motion records having AI between 0.2 and 10 m/s, which span the range between the smallest shaking intensity that might cause landslides and the largest shaking intensity ever recorded. For each strong-motion record, they calculated the Newmark displacements for several a_c between 0.02 and 0.4 g, the range of practical interest for most earthquake-induced landslides. Results of this analysis are presented in Figure 2.8, which is proposed as design chart.

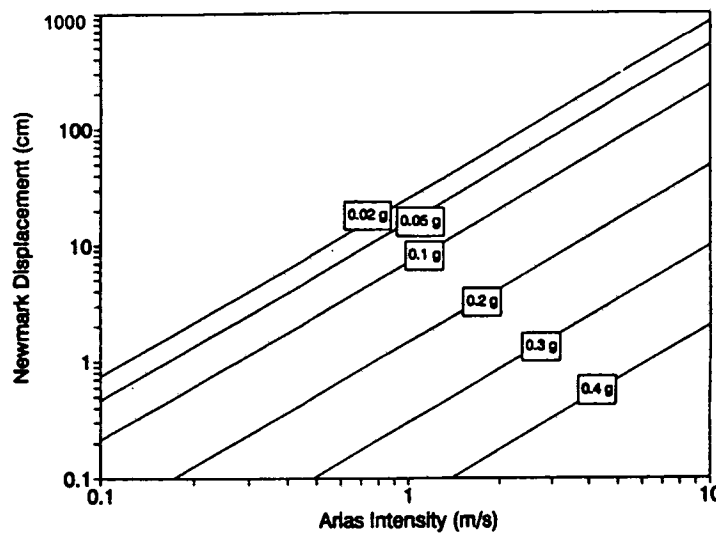


Figure 2.8 Newmark displacement as a function of Arias Intensity for several values of critical acceleration (Jibson and Keefer, 1993).

2.2.6 Energy Density (E')

Sarma (1971) introduced the concept of energy flux, which is defined, for a particular site, as the energy radiated per unit area in the wave front per unit time. It is shown that the energy density at a given site is the total energy that went past during the earthquake per unit area:

$$E = \frac{V\rho}{4} \int_0^{T_0} v^2(t) dt \quad (2.4)$$

where V is the wave velocity, ρ is the mass density and v is the particle velocity. Assuming that ρ is constant and that the energy at a point comes mainly from the contribution of S-waves, the energy density can be simplified as:

$$E' = \int_0^{\tau_0} v^2(t) dt \quad (2.5)$$

The component energy densities, as in the case of AI can be added to obtain the total energy at a site. The energy flux has not been used in analyses of earthquake-induced landslides.

2.2.7 Strong-motion duration

There are a number of situations in which the seismic response depends very strongly not only on the amplitude of the ground motion but also on the number of cycles or duration. Seed (1968) stated that the duration of the 1964 Alaska earthquake was one of the dominant factors causing instability of clay deposits. According to Seed, landslides in Anchorage due to this earthquake, did not began to develop until about one and a half minute or two minutes after the strong ground shaking started. Another earthquake of similar magnitude, such as the 1906 San Francisco earthquake, but of shorter duration, may not cause slope failures in this type of deposits.

For any material with stiffness or strength degrading characteristics, including soils, the duration of earthquake shaking will be of critical importance, as presented by Lee and Focht (1976). These authors stated that because of the strain dependency of stiffness and strength, and because of soil creep, cyclic load conditions that produce the longest duration of high sustained load per cycle will lead to the greatest amount of cyclic strength and stiffness degradation.

A large variety of approaches to identifying and measuring the strong-motion duration have been proposed by different researchers. A review of those approaches was done by Bommer and Martínez-Pereira (1999) who classified these definitions into three generic groups: bracketed, uniform and significant durations.

Duration can be defined as the interval between the first and last acceleration excursions greater than a certain absolute amplitude (threshold) of the accelerogram record, a_0 . This type of duration,

known as bracketed duration, D_b , varies due to different threshold values that can be selected as an absolute value or a fraction of the peak ground acceleration. One disadvantage of this definition is that it considers only the first and last peaks that cross the specific threshold and ignores completely the characteristics of the strong shaking phase, which can result in long durations being estimated for earthquakes with small sub-events occurring after the main shock has passed (Martínez-Pereira, 1999).

Another possibility to define duration is to consider the total sum of time intervals, t_i , during which a particular level of acceleration, a_0 , is exceeded (uniform duration, D_u). As with bracketed duration, the level can be absolute or relative to the peak ground acceleration. This definition is less sensitive to the threshold value than the bracketed duration, but it has the disadvantage that it does not define a continuous time window during which the shaking can be considered strong.

A third group of record-based definitions are called significant duration, D_s , and these are based on the accumulation of energy in the accelerogram presented by the integral of the square of the ground acceleration, velocity or displacement. If the integral of the ground velocity is used, then the quantity is directly related to the energy density (Sarma, 1971), whereas if the integral of the ground acceleration is used then the quantity is related to the Arias Intensity (Arias, 1970). The significant duration is defined as the interval over which some proportion of the total integral is accumulated. The significant duration has the advantage that it considers the characteristics of the entire accelerogram and defines a continuous time window in which motion may be considered as strong.

As stated by Martínez-Pereira (1999), it is important to recognise that it may not be useful to establish a single, universal definition for strong-motion duration, since different definitions may be more or less appropriate in different situations. He also postulated that the use of frequency-dependent durations may provide better constrained prediction of variation with distance, since high-frequency motions attenuate more rapidly than low-frequency motions. There may also be a use for frequency-dependent durations if the objective is to generate synthetic time histories by predicting the amplitude, duration and phase characteristics of different component frequencies of the ground motion. Nonetheless, attenuation relations essentially describe the radiation of seismic energy in

earthquakes, and it is therefore more robust to work with the duration of the complete record rather than isolated frequency components which only carry a limited portion of the wave energy.

Due to the limitations of bracketed and uniform duration definitions, two significant duration definitions were used in the present study. The Trufinac and Brady (1975) and Bommer and Martínez-Pereira (1999) definitions.

The Trufinac and Brady duration is defined as the time interval during which the central 90% of the contribution to the integral of the square of the acceleration takes place. The Bommer and Martínez-Pereira definition, called effective duration, D_e , is based also on the build up of the Arias Intensity. In this case the strong-motion duration is the time interval between an initial, t_0 , and final, t_f , times that mark the start and end of the strong motion. These times are obtained by using absolute Arias Intensity values.

The start of the strong motion phase, t_0 , is considered as the time when the Arias Intensity reaches a particular value, defined by the authors as 0.01 m/s. The end of the strong motion phase, t_f , is defined as the time at which the remaining energy in the records is equal to an absolute value ΔI_{ar} . A value of 0.125 m/s was selected as ΔI_{ar} . Clearly any record whose total AI is less than 0.135 m/s will not have an effective duration. The implicit assumption, therefore, is that an accelerogram with an I_a not greater than 0.135 m/s is not considered as strong motion, from an engineering perspective. Figure 2.9 shows the Husid Plot and these two duration values for the N-S component of the Bagnoli record of the 1980 Irpinia earthquake.

2.3 Factors influencing strong-motion

There are three broad categories in which the factors that influence the ground motion can be grouped: source, path and site effects. The first group refers to the earthquake rupture process that produces seismic waves. The path is associated with all factors that affect ground motion between the source and the point of observation. Site effects account for variables that characterise the location where the recording was obtained.

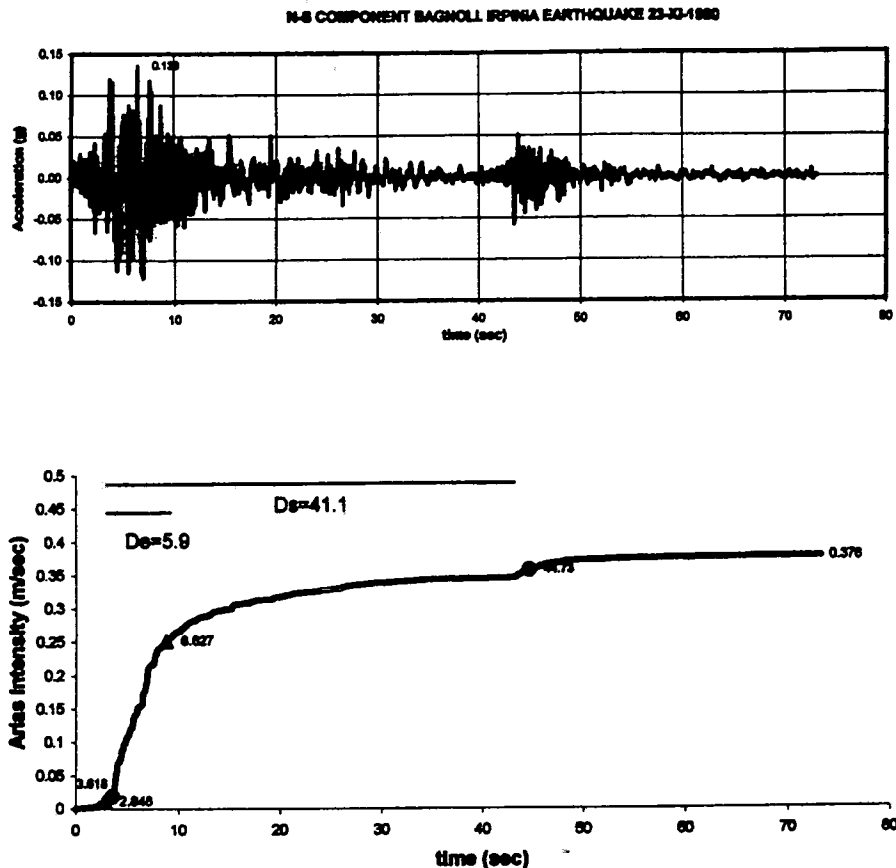


Figure 2.9 Strong motion and Husid plot for the N-S component of Bagnoli record during the 1980 Irpinia earthquake. D_e and D_s also shown.

2.3.1 Source factors

Factors related to the earthquake source that have been found to affect strong motion characteristics are: source mechanism, rupture pattern, radiation pattern, magnitude and depth.

2.3.1.1 Source mechanism

The process of faulting can be compared to sliding two rough boards past one another. Friction along the boundary between the boards may be temporarily slow their motion, but rough edges break off and motion occurs along the boundary. This process is analogous to what happens at tectonic plate boundaries where one plate slides past or overrides another. The rocks undergo strain and, if the stress continues, the rock eventually breaks forming a fault. A fault is defined as a

fracture or fracture system along which rocks have been displaced, that is, material on one side of the fault have moved relative to material on the other side.

Faults are classified according to their geometrical configuration and sense of relative displacement. Figure 2.10 shows the main types of faulting. Fault mechanism is controlled by the direction of the principal stresses and the presence of zones of geological weakness along which rupture may preferentially occur. Strike-slip faults are those whose direction of slip is horizontal. Depending on the relative sense of motion of the two sides of the fault, these can be left- or right-lateral. Dip-slip faults are those where the sense of motion is parallel to the dip of the fault; the dip of the fault being the angle that this makes with respect to the horizontal plane. A dip-slip fault is either reverse or normal, according to the relative movement of the hanging wall and foot wall. The hanging wall and foot wall are those blocks overhanging and underlying the fault plane, respectively. In normal faulting the hanging wall moves down respect to the foot wall, whereas the contrary sense of movement defines a reverse faulting. Many faults are a combination of strike-slip and dip-slip movements, in which case they are referred as to oblique-slip.

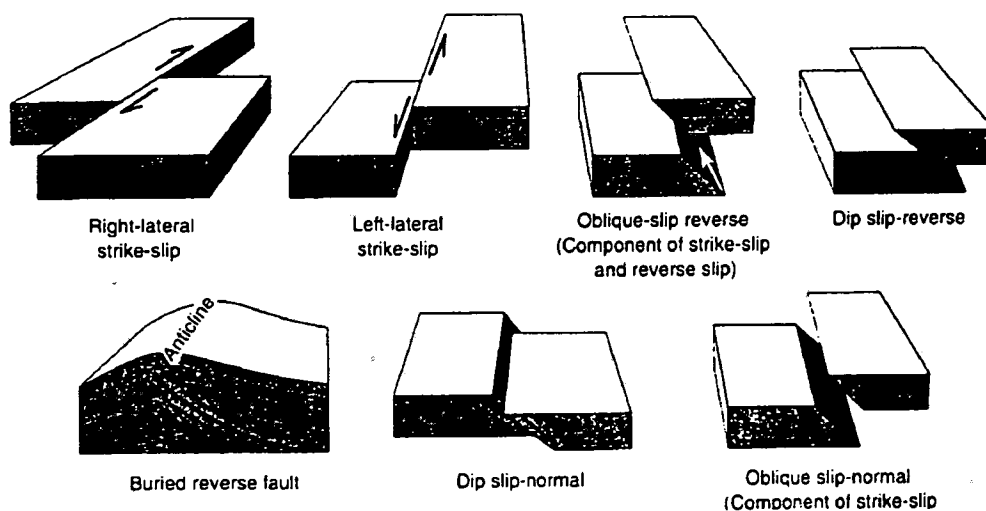


Figure 2.10 Types of fault movement based on sense of fault motion (Keller and Pinter, 1996)

The fault mechanism is thought to affect the ground-motion characteristics, even though there is no conclusive evidence to affirm this as was stated by Martínez-Pereira (1999). Some researches

suggest that thrust (reverse) events produce higher amplitudes than strike-slip and normal events (e.g. Campbell, 1981, 1989; McGarr, 1984; Ambraseys and Simpson, 1996). However, other studies do not show significant differences between ground motions from various types of faulting (Crouse *et al.*, 1988; Westaway and Smith, 1989).

It has been found that peak horizontal accelerations from thrust earthquakes are 20 to 30 % larger than from strike-slip earthquakes for the same magnitude and closest distance (Abrahamson and Somerville, 1996). Somerville (1998) stated that this difference is about 30 to 40 %, and suggested that the difference is due to differences in dynamic parameters, specifically the time of slip on the fault. For a given seismic moment, the rise times for reverse earthquakes are on average about half as long as those for strike-slip earthquakes. However, he stated that available data indicate that strong-motion records from normal faulting earthquakes are several tens of percent lower than those for strike-slip earthquakes. Campbell and Bozorgnia (1994) found that the style of faulting effect decreases with increasing magnitude and distance.

Somerville (1998) also stated that strong motions from subduction zone earthquakes attenuate more gradually with distance than crustal earthquakes, and intra slab earthquakes produce stronger ground motions than subduction zone earthquakes. Mixing these data sets gives rise to attenuation relationships that overpredict the ground motions from crustal and shallow subduction earthquakes and underpredict those from deep slab earthquakes.

2.3.1.2 Rupture pattern

Detailed studies of the spatial distribution of slip on the fault plane, derived from strong motion records, have shown that the slip distribution is highly non-uniform, characterised by asperities (regions of large slip) surrounded by regions of low slip (barriers) (Somerville, 1998).

Prediction of strong ground-motions require modelling the earthquake as a rupture spreading across a fault plane. For some past earthquakes, rupture models have been derived from strong motion records, geodetic data and teleseismic recordings of the earthquake. Irikura and Iwata (1993) stated

that there are two ways to characterise the fault rupture at an earthquake source: kinematic and dynamic source models. The kinematic source models give arbitrary slip space-time functions on the fault plane without physical considerations concerning stress conditions. On the other hand, dynamic source models take into account the real physical process in the source region; these slip models have been used to develop the relationship between seismic moment or earthquake magnitude and a set of fault parameters which include fault length, fault width, rise time (duration of slip on the fault), and size, slip contrast, and location of asperities (Somerville, 1998; Somerville *et al.*, 2000).

Slip direction can also vary spatially over the fault, and can also vary in time at a given point of the fault, therefore, further improvements in existing models must consider these parameters for the strong motion prediction.

2.3.1.3 Magnitude

Amplitude, frequency content and duration of strong motions are influenced by earthquake magnitude as can be seen in Figure 2.11, where different records, obtained at nearly the same epicentral distance from six earthquakes off the Pacific Coast of Mexico, are shown.

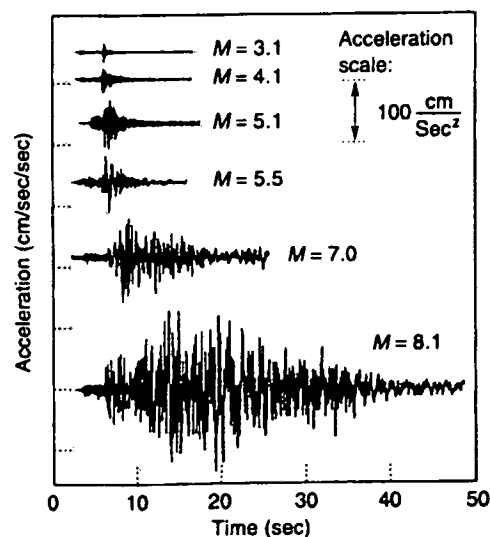


Figure 2.11 Accelerograms from six earthquakes off the Pacific coast of Mexico. All records measured at similar epicentral distance. The record from the M=8.1 continues for another 25 sec. (Kramer, 1996)

2.3.1.4 Depth

Hypocentral depth is an important parameter to characterise the earthquake source. It is, however, more often incorporated as part of the path effects, when defining source-to-site distance. An earthquake is supposed to be nucleated at the hypocenter, from where the rupture propagates outwards along the fault plane. The seismogenic zone has a limited size in depth. Disregarding deep earthquakes, much of the seismicity takes place in the so-called schizosphere, which is the part of the lithosphere associated with brittle behaviour. The schizosphere width varies, but a rough estimate situates it at 15-20 km.

Earthquakes of different sizes involve different volumes of Earth's crust. Therefore, two broad types of earthquakes can be established: small and large earthquakes, for which the hypocentral depth has completely different implications. Small earthquakes are those whose rupture dimensions are smaller than the width of the schizosphere, where they propagate and terminate entirely. Therefore, small earthquakes can be considered as ruptures in an unbounded elastic, brittle solid. Large earthquakes, on the other hand, will reach the bottom of the schizosphere rapidly, are then constrained to propagate only horizontally. These earthquakes therefore are produced by ruptures in bounded elastic, brittle material. This distinction is important since it suggests, as indeed it has been found, that small and large earthquakes obey different scaling laws, and have different spectral shapes, which can be partly explained by the geometry and propagation pattern of the rupture. In the light of these arguments, it can be seen that hypocentral depths is an important variable when describing small earthquakes, since strong ground-motions may be expected for shallow focus, but is less significant or completely irrelevant for large earthquakes.

Crouse *et al.* (1988) have suggested, based on regression analysis of data from subduction-zone events from the northern Honshu zone, that deeper events might have higher amplitude at short period because of higher stress drop or lower anelastic attenuation. They suggested that the deeper events might have lower amplitude at long period because they are less effective in generating surface waves.

2.3.2 Path factors

One of the most important phenomena along the propagation path is seismic attenuation. As the stress waves spread out from the source, they are partially absorbed by the materials in which they travel. If one consider the seismic wave front as spherical, it is expected that, as the waves moved out from the nucleation point, the surface area will be wider and therefore the energy density will decrease. Absorption is a result of internal friction, which depends strongly on the material properties. The phenomenon that the seismic wave amplitude is apparently attenuated by the effect of scattering due to heterogeneities of the medium has been noticed as an important mechanism of the seismic wave attenuation.

Rupture directivity causes spatial variation in ground-motion amplitudes and duration around faults and cause differences between strike-normal and strike-parallel components of horizontal ground motion amplitudes, which also have spatial variation around the fault.

2.3.2.1 Source-to-site distance

Energy density decreases with increasing distance from the source. Since the characteristics of stress waves are strongly related to specific energy, they will also strongly related to distance. A number of ways of defining the source-to-site distance have been proposed as is shown schematically in Figure 2.12. Epicentral and hypocentral distance are possibly the most widely used due to ease of calculation. They refer to the distance of the site to the epicentre and hyocentre, respectively. These definitions have the disadvantage that they treat the earthquake source as a point, which can be a reasonable approximation for distances that are large compared to the dimensions of the fault. On the contrary, when the site is located at distances comparable to or smaller than the dimensions of the fault, these distances tend to overestimate the distance from the actual source of seismic energy (Seed *et al.*, 1976). In such circumstances, the distance from the fault break would be a more appropriate measure, as proposed by Schnabel and Seed (1973) and Bureau (1978). This definition in turn has the problem of being difficult to assess when faulting does



not reach the surface or takes place underwater. The second disadvantage of this definition is that the source is treated as a line, whereas in fact it is more correctly two- or three-dimensional.

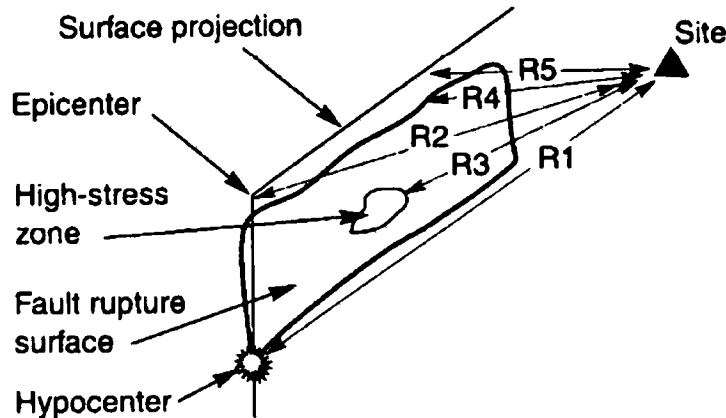


Figure 2.12 Various measure of distance used in strong motion predictive relationships (Kramer, 1996)

There are other definitions that consider the fault rupture as an area or volume. Boore *et al.* (1980) use the shortest distance between fault slippage and the recording point. Campbell (1981) prefers to use the shortest distance to the fault surface. This, however, relies on the identification of the fault geometry which sometimes is really uncertain. Joyner and Boore (1981), and later Ambraseys and Bommer (1991), introduced the distance taken as the shortest to the surface projection of the fault rupture; this also requires the definition of the faulting boundaries, and it implicitly weights the geometrically closest part of the fault to cause the strong-motion characteristics. Campbell (1985) and Duke *et al.* (1977) stated that ground-motion parameters, such as acceleration, are controlled by a limited portion of the total earthquake fault, therefore distance is defined with reference to the centre of energy, inferred from the high-stress zone in the fault.

Usually, the amplitude of ground motion is greatly reduced at distances beyond 100 km. However, unexpectedly strong ground-motion has been observed at relatively large distances due to the simultaneous arrival and constructive interference of direct S-waves and S-waves reflected from the Moho discontinuity. This has been observed in the 1988 Saguenay (Canada) earthquake (Somerville *et al.*, 1990) and the 1989 Loma Prieta earthquake (Somerville and Yoshimura, 1990).

2.3.2.2 Directivity effects

Rupture directivity effects cause spatial variations in ground-motion amplitude and duration around faults and cause differences between the strike-normal and strike-parallel components of horizontal ground-motion amplitudes, which also have spatial variation around the fault (Somerville *et al.*, 1997, Somerville, 2000).

Somerville *et al.* (1997), based on world-wide data, have found that the spectral acceleration is large for periods larger than 0.5-0.6 s, and the variation is larger when rupture propagates toward the site. For sites located close to faults, the strike-normal spectral acceleration is larger than the strike-parallel spectral acceleration at periods larger than 0.6 s, in a manner that depends on magnitude, distance and angle.

The propagation of rupture toward a site at a velocity that is almost as large as the shear wave velocity causes most of the seismic energy from the rupture to arrive in a single large pulse of motion. This pulse of motion represents the cumulative effect of almost all of the seismic radiation from the fault. The radiation pattern of the shear dislocation on the fault causes this large pulse of motion to be orientated in the direction perpendicular to the fault (Somerville *et al.*, 1997; Somerville, 2000).

Forward rupture directivity effects occur when two conditions are met: the rupture front propagate toward the site, and the direction of slip on the fault is aligned with the site. Backward directivity effects, which occur when the rupture propagates away from the site, give rise to the opposite effect: long duration motions having low amplitudes.

The conditions required for forward directivity are also met in dip-slip faulting, including both reverse and normal faults. The alignment of both the rupture direction and the slip direction up the fault plane produces rupture directivity effect at sites located around the surface exposure of the fault (or its up-dip projection if it does not break the surface). Consequently, it is generally the case that all sites located near the surface exposure of a dip-slip fault experience forward rupture directivity

effects when an earthquake occurs. Unlike the case for strike-slip faulting, where it is expected that forward rupture directivity effects will be most concentrated away from the hypocentre, dip-slip faulting produces directivity effects on the ground surface that are most concentrated up-dip from the hypocentre.

2.3.2.3 Hanging wall effect

The use of the closest distance to the fault plane as the distance measure causes a site on the hanging wall to have closer proximity to the fault as a whole than a foot wall site, even though the two sites are at the same closest distance to the fault plane. This causes larger short-period ground motion on the hanging wall than on the foot wall at the same closest distance. The hanging wall adds to the style of faulting factor for short-period ground motions thrust earthquakes, which are typically 1.3 to 1.4 times larger than those from strike-slip earthquakes (Somerville, 2000).

Abrahamson and Somerville (1996) compared the ground motions for sites on the hanging wall with those on the foot wall and those off the end of the fault rupture. They found a significant systematic increase in ground motions for sites over the hanging wall; this effect is greatest (a factor of 1.45) in the closest distance range of 8 to 18 km in the period range of 0 to 0.6 s, and decreases to unity at 5 s; but the decrease in ground motions for sites on the foot wall was not as systematic.

2.3.2.4 Seismic wave scattering

The multiple reflection and refraction of seismic waves by the inhomogeneous geological properties and topography of the propagation path, and the local site conditions, cause spatial and time fluctuations on amplitude and phase characteristics of ground motion. This is the so-called multiple scattering effect. The lithosphere appears to be the most heterogeneous region within which seismic waves are extremely scattered. Models to account for the effect of seismic waves scattering describe the lithosphere statistically as a medium which has randomly inhomogeneous velocity structures (Takemura and Kawano, 1993).

At close distances (within about 50 km), the largest ground motions are caused by waves that travel upward from the earthquake source to the site. However, as distance from the source increases, the direct wave becomes weaker, and the reflections of down going waves from interfaces below the source reach the critical angle and undergo total internal reflection. The strong contrast in elastic moduli at these interfaces, especially the Moho, causes these critical reflections to have large amplitudes. The arrival of these critical reflections, beginning at a distance of about 50 km, causes a reduction in the rate of attenuation of ground motion out to distances of about 100 km or more (Burger *et al.*, 1987). While the elevated ground-motion amplitudes in this distance range are usually not large enough by themselves to cause damage, they may produce damage if combined with the amplifying effects of soft soils. The destructive potential of these effects was dramatically demonstrated in the 1989 Loma Prieta earthquake (Somerville *et al.*, 1994), in which major damage was done to buildings and bridges in the San Francisco Bay area, located 80 to 90 km from the earthquake. The degree of reduction in the rate of attenuation caused by the crustal wave guides, and the distance range over which it occurs, depend on the depth of the earthquake and the thickness and velocity profile of the crust. Consequently, ground-motion attenuation characteristics vary depending on the crustal structure and the depth of the earthquake.

2.3.3 Site factors

Amplification due to subsurface layers can be classified into two categories: geometrical amplification and dynamic amplification (Noda *et al.*, 1993). The former corresponds to the amplification effect at the interface due to the impedance contrast between two contacted media. It depends on the medium properties and the types and approach angle of incident waves, but not frequency. The latter is the so-called resonance effect by flat layering. It depends not only on the medium properties and incident waves but also on frequency. Geometrical effects can be observed at the first arrival of seismic waves, while the dynamic effect can be observed in spectra or spectral ratios.

Actual soft surface layers are not infinitely flat but often laterally confined in the form of a sediment-filled valley. The presence of an edge of the surface layer generates diffracted waves and surface

waves, and they have significant influence on ground-motion records observed for soft sediments. It has been found from recorded strong-motion data that ground motions may be large at the edges of fault controlled basins (Noda *et al.*, 1993).

Complex amplification and deamplification patterns produced by surface topography resulted in significant changes in ground motion, which are generally observed as a frequency-dependant amplification with elevation, with the frequency being related to the slope and size of topographical feature. On the other hand, subsurface topography produces a complex seismic wave scattering, which can generate the focusing of seismic waves in a localised area.

2.3.3.1 Surface geology

The motion experienced at the ground surface is the response of the soil layer, assuming the input motion at the bedrock-soil interface. Usually, soil deposits amplify the ground motion and this is most commonly due to the phenomenon of impedance. As the velocity of propagation of the seismic waves in the bedrock is greater than in the soil, the wave energy transmitted into the soil layer is retarded. In order to maintain the flow of energy, the amplitude of the waves in the soil deposit must increase and thus the ground motion is amplified.

The effect of site conditions on strong-motion amplitude and frequency content can be seen in Figure 2.13, which shows velocity records obtained during an aftershock of the 1983 Coalinga (California) earthquake, and compares the strong motion on alluvium and bedrock.

2.3.3.2 Resonance of sediment basin

If there is a sharp velocity contrast between the soil layer and the underlying bedrock, the waves reflected at the ground surface will be reflected at the soil-rock interface repeatedly. Therefore, the seismic energy become trapped in the soil layer, increasing the duration of the ground motion.

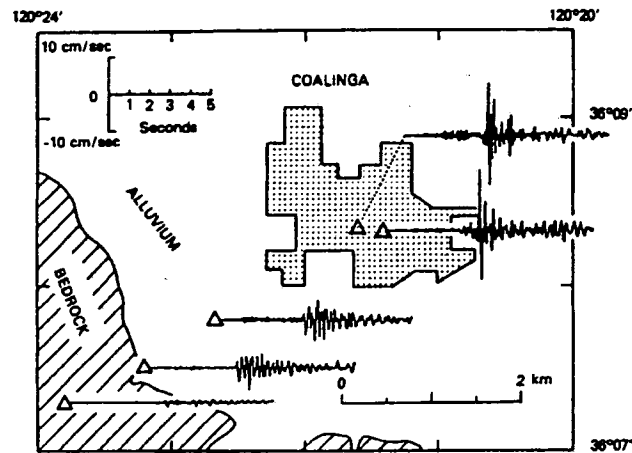


Figure 2.13 Velocity records obtained during a 1983 Coalinga (California) aftershock ($M=4.3$) illustrating the change in strong motion on alluvium compared to the bedrock site (Reiter, 1990).

Strong motion records obtained at different locations during the 1985 Michoacán earthquake are presented in Figure 2.14. It is shown the resonant effects of soft clay, in Mexico City, whose natural period was close to that of surface waves at rocky parts of the city (Celebi *et al.*, 1987).

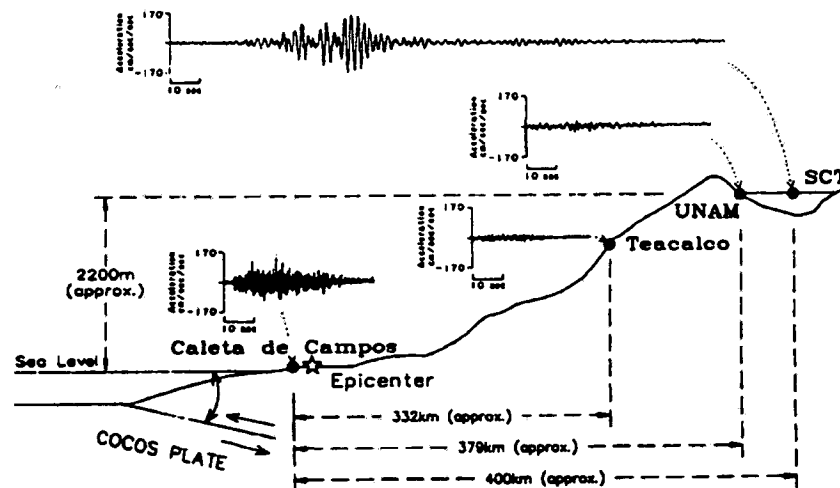


Figure 2.14 Resonant response of soft clay in Mexico City during the 1985 Michoacán earthquake (Celebi *et al.*, 1987)

2.3.3.3 Basin edge effect

Surface waves in sediment-filled valleys can be classified into two categories: basin-induced surface waves and basin-transduced surface waves (Noda *et al.*, 1993). The former is the surface wave

generated at the edge of the basin by an incident body wave, while the latter is the surface wave originated from the source and transmitted into the basin. The effects of these surface waves for the sediment-filled valley is to increase the amplitude and duration of strong motion inside the valley.

Somerville (1998) stated that severe damage to buildings due to the 1995 Kobe earthquake was observed in a zone about 30 km long and 1 km wide, and offset about 1 km southeast of the fault on which the earthquake occurred. According with Somerville (1998), in this case the ground motions were amplified by the basin edge effect. The large ground motions in this zone are believed to have been caused by constructive interference of direct waves and waves diffracted by the edge of the Osaka Basin. The basin structure produces a strongly asymmetrical zone of large amplitudes along the edge of the basin. However, since Osaka Basin is some distance from Kobe, it is possible for the effect to be caused by deep local basin focusing.

Some theoretical studies have been performed in Japan in order to model the edge basin effect. Noda *et al.* (1993) report some cases where actual records were simulated by considering the basin effect. Those studies include, among others, cases for the Kanto plain, the Osaka plain, the Kyoto Basin, the Ashigara basin and the Tokachi plain. Studies that tried to model the San Fernando and Los Angeles basins during the 1971 San Fernando earthquake are also reported by Noda *et al.* (1993).

2.3.3.4 Focusing effect

It has been hypothesised that pattern of damage caused by the Northridge earthquake was characterized by pockets of localized damage, such as those in Sherman Oaks, that were not clearly correlated with superficial soil conditions (Somerville, 2000). These observations have produced important new insights into the causes of localized zones of damage. During the Northridge earthquake, deeper underlying geological structure may have had as much influence on strong motion patterns as the upper 30 meters that are conventionally used to characterize site response. This deeper structure may include sedimentary structures in the upper few kilometres of sedimentary basins, as well as the topography of the underlying sediment/basement interface.

These structures, in the form of folds and buried basins, focus energy (like a lens) in spatially restricted areas on the surface, in some cases becoming the dominant factor in the modification of local ground-motion amplitudes. This focusing effect depends on the azimuth and incident angle of the arriving waves.

2.3.3.5 Topographic effect

Theoretical and numerical models predict a systematic amplification of seismic motion at ridge crests, and, more generally, over convex topographies, such as cliff borders, and a correlative deamplification over concave parts of surface topography, such as valleys and foothills; the extent of these effects has been shown to be rather sensitive to the characteristics of the incident wavefield: wave type, incidence and azimuth angles. Bard (1995) related these effects with three physical phenomena:

1. the sensitivity of the surface motion to the incident angle, which is especially large for SV waves around the critical angle, which results in a significant variation of surface motion with slope angle,
 2. the focusing or defocusing of seismic waves reflected along the topographic surface, and
 3. the diffraction of body and surface waves, which propagates downwards and outwards from the topographic feature, and lead to interference patterns between the direct and diffracted waves.
- The amplitude of the waves propagating along the surface generally remains smaller than the direct waves, at least for smooth topographies comparable to those usually met in nature.

Amplification due to topographic effects is generally greater on horizontal components than the vertical component, and, within the two horizontal components, it seems larger for motion perpendicular to the ridge axis in the case of elongated ridges. It also seems very roughly related to the sharpness of the topography, the steeper the average slope, the higher the topographical amplification. Furthermore, this amplification (deamplification) phenomenon is frequency dependent

and more precisely, band limited. Observations show the maximum effect corresponds to waves that have lengths comparable to the horizontal dimension of the topographical feature (Bard, 1995).

Topographical effects have been often reported after earthquakes in hilly areas. The 1985 Valparaiso (Chile) earthquake caused severe damage to buildings situated on the hilltop, crowned by ridges and canyons, at Canal Beagle. This was interpreted by Celebi (1987) as evidence of a ridge effect. Figure 2.15(a) shows the topography of this area and the location of portable stations, which recorded several aftershocks simultaneously. The strong-motion records of these stations and reference stations (Valparaiso), for an aftershock, are shown in Figure 2.15(b). The topographical influence on the strong-motion characteristics can be clearly seen.

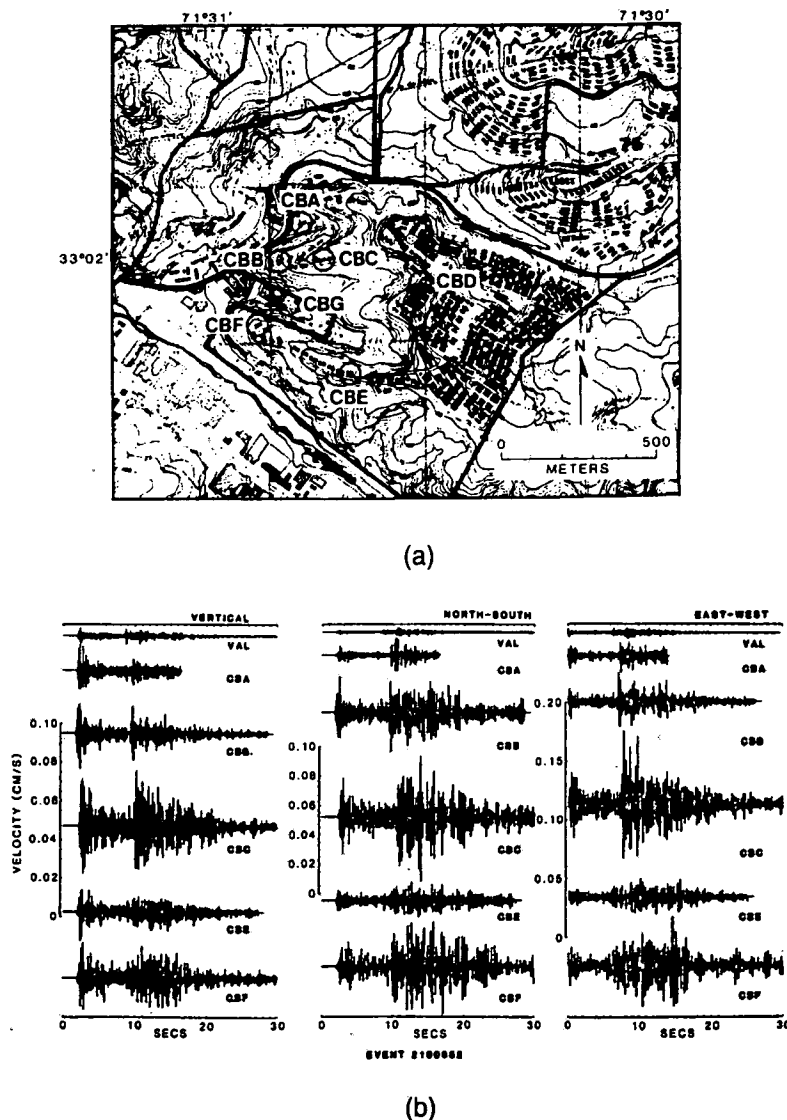


Figure 2.15 (a) Topography and portable station locations at Canal Beagle area. (b) Velocity records for vertical and horizontal components of an aftershock (29 July 1985) of the 1985 Valparaiso earthquake (from Celebi, 1987).

2.4 Simulation and prediction of strong ground-motion

Ground motions generated by an earthquake can be simulated using three different approaches: theoretical, semi-empirical and empirical. The first two are based on the physical background of faulting motion, seismic wave propagation and arrival. The third approach aims at simulating ground motions based on stochastic analyses of observed records from earthquakes (Somerville, 2000).

In the theoretical approach, the displacement caused by a slip dislocation across a fault surface is expressed as a space-time convolution of a slip function on the fault with a Green's function, which represents the Earth's response. Methods that estimate Green's function can be divided into two types: one is an entirely theoretical calculation for the seismic wave response of realistic Earth structures due to a point source, and the other takes advantage of small-event records occurring near the potential source of a large earthquake for empirical Earth responses. The former is called the theoretical approach, whereas the later is the semi-empirical approach.

The above three approaches should be incorporated into a practical prediction method of strong ground-motion. Theoretical and semi-theoretical approaches, however, have been used to understand the processes involved in the generation, propagation and arrival of seismic waves, producing results that have been used for improvement and calibration of empirical approaches. During the present study an empirical approach was used. Description of theoretical approaches is beyond the scope of this project; however, extensive studies about these models, such as Aki and Richards (1980) or Irakura *et al.* (1993), can be consulted.

Prediction of strong motions based on recorded accelerograms has been made by using statistical regression on the data. Spatial variation of data is a very common problem in geology and geotechnical studies. The process of prediction of spatial distributed parameters (variables) based on actual data at known localities is commonly referred as Kriging (Davis, 1986). This method has been used for regional seismic hazard analysis (King and Kiremidjian, 1994) and for prediction of strong-motion parameters (Carr and Glass, 1989; Yamazaki, 2000). However, the most common empirical approach for predicting strong-motion parameters has been the use of attenuation curves to determine maximum amplitude of seismic waves and their response spectra.

2.4.1 Prediction of strong motion using the Kriging approach

Kriging is synonymous with optimal prediction, and is a method of interpolation which predicts unknown values from data observed at certain locations. Ordinary Kriging is “linear” because its estimates are weighted linear combinations of the available data; it is “unbiased” since it tries to have the mean residual or error, m_R , equal to 0; it is “best” because it aims at minimising the variation of errors, σ_R^2 .

In practical situations, we never know the true answer or the actual errors before we attempt our estimation. This means that we never know m_R and therefore cannot guarantee that it is exactly 0. Nor do we know σ_R^2 ; therefore, we cannot minimise it. Kriging approach builds a model of the available data and works with the average error and the error variance for the model. Ordinary Kriging uses a probability model in which the bias and the error variance can both be calculated and then chooses weights for the samples that ensure that the average error for our model, $\tilde{m}_R$, is exactly 0 and that our modelled error variance, $\tilde{\sigma}_R^2$, is minimised. A random function model is used since this type of model enables the error, its mean value and its variance to be expressed mathematically. The theoretical basis for Kriging prediction is summarised in Annex A.

Carr and Glass (1989) used Kriging to map peak ground horizontal acceleration for the 1971 San Fernando earthquake and 1983 Coalinga earthquake. Comparison of predicted values with those obtained from an attenuation equation were made. In this case Kriging was found to give a reasonable spatial distribution of values, although, comparisons were based on attenuation equations that use epicentral distance as source-to-site factor, which give a concentric distribution of values around the epicentre. No attempt to compare actual data with predictions was presented. The Kriging technique was also used by Yamazaki *et al.* (2000) to find the spatial variation of the PGA, PGV and JMA intensity during the 1995 Hyogoken-Nanbu earthquake. In this case they use an exponential model with a range of 5 km, and the analysis was made for residual values defined as the difference between values at the base rock and the trend component. As trend component they used an attenuation relationship for each parameter.

Exponential and spherical models were used in this study for interpolating and extrapolating strong-motion for the 1980 Irpinia (Italy) earthquake. Figures 2.16-2.20 show contours of predicted PGA values for the N-S component and the corresponding recorded values at seismic stations around the epicentral area. In general, it can be seen that this technique tends to under estimate values around the maximums and over estimate values for distant points from these locations. As a linear model, Kriging is unable to predict values higher than the maximum known value. On the other hand, contours tend to be concentric to the maximum values and due to the lack of stations near the seismic source, values around the epicentral area tend to be very low.

Exponential $a=50$

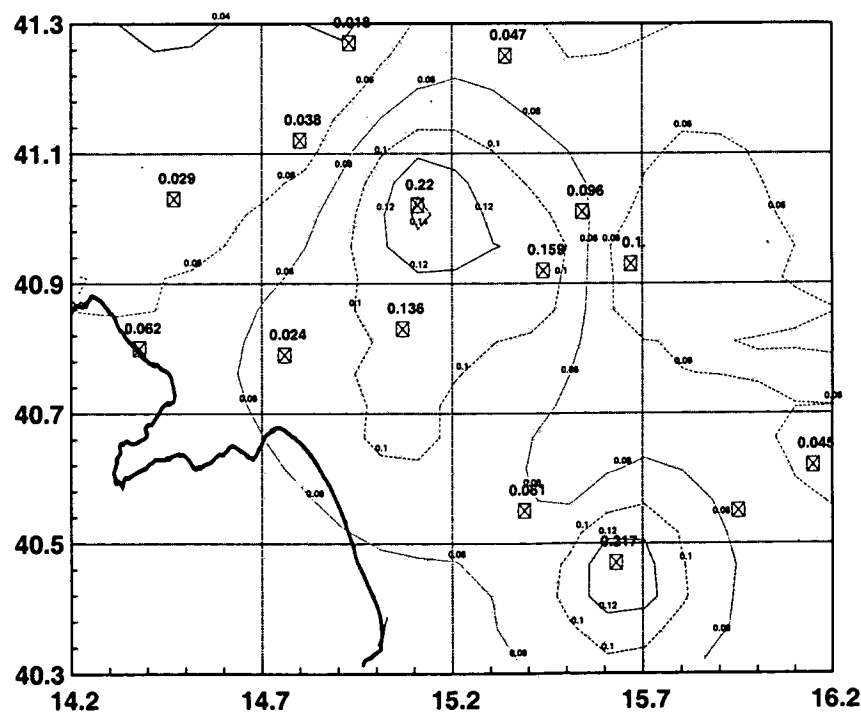


Figure 2.16 PGA distribution for the N-S component during the 1980 Irpinia earthquake. Numbers in large font are the observed values. Contour lines obtained by Kriging of PGA values using an exponential model with $a=50$.

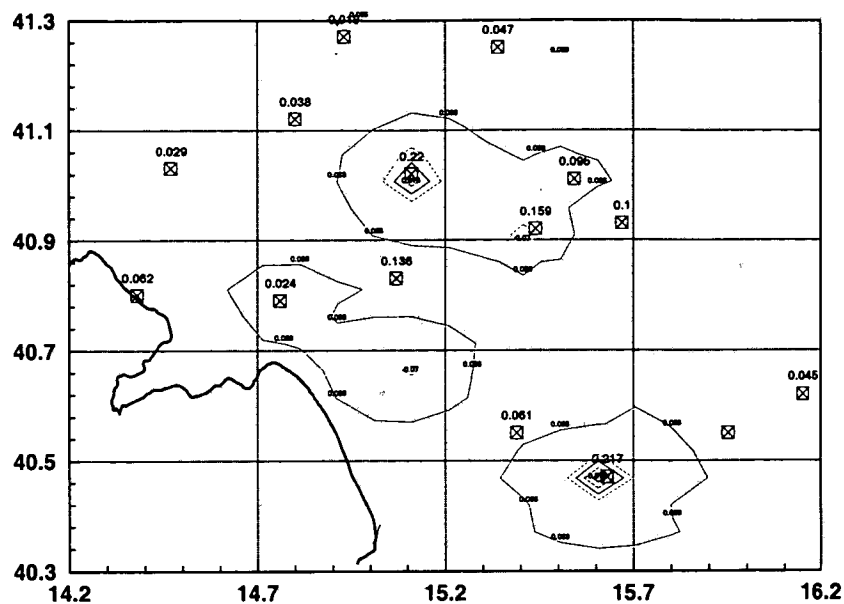
Exponential $a=5$ 

Figure 2.17 PGA distribution for N-S component during the 1980 Irpinia earthquake. Numbers in large font are the observed values. Contour lines obtained by Kriging of PGA values using an exponential model with $a=5$.

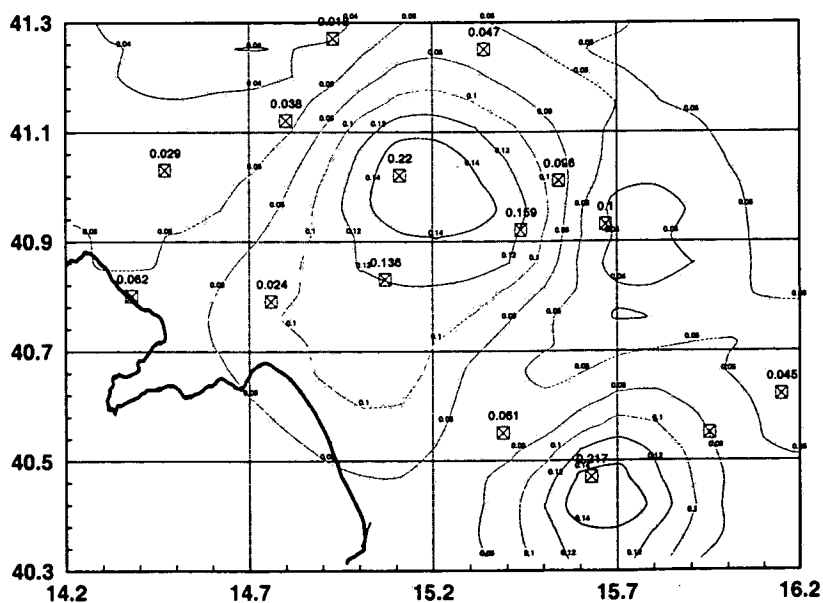
Spherical $a=50$ 

Figure 2.18 PGA distribution for the N-S component during the 1980 Irpinia earthquake. Numbers in large font are the observed values. Contour lines obtained by Kriging of PGA values using a spherical model with $a=50$.

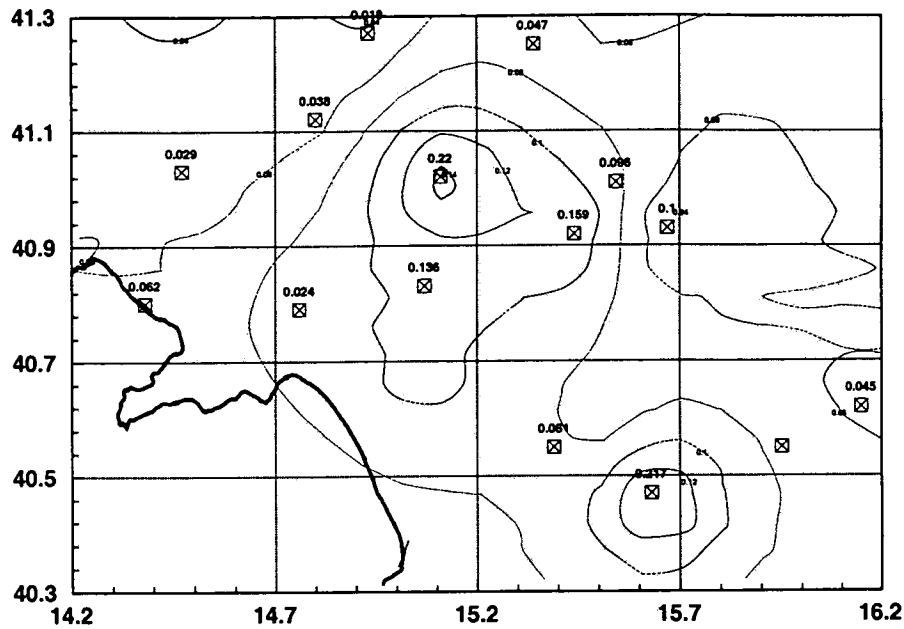
Exponential $a=50C10$ 

Figure 2.19 PGA distribution for the N-S component during the 1980 Irpinia earthquake. Numbers in large font are the observed values. Contour lines obtained by Kriging of PGA values using an exponential model with $a=50$ and $c=10$.

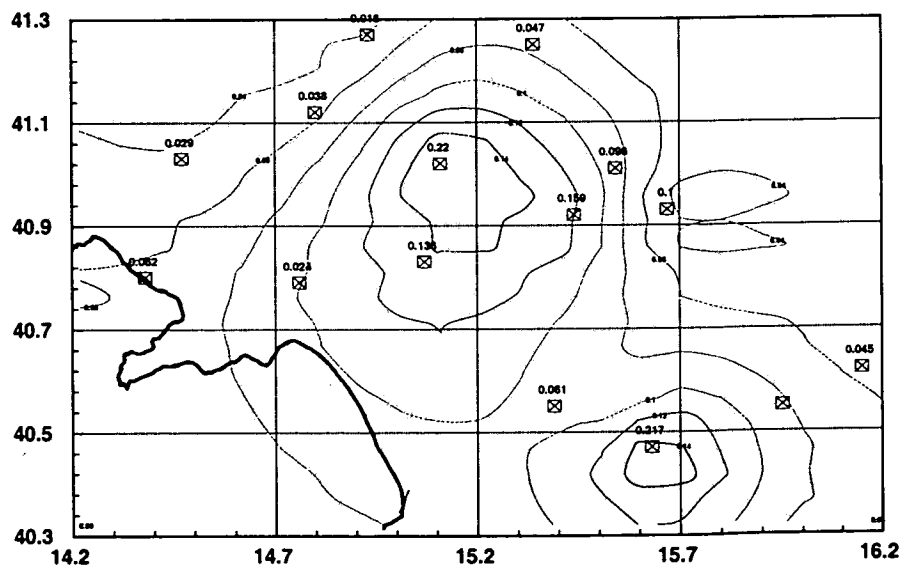
Exponential $a=100$ 

Figure 2.20 PGA distribution for N-S component during the 1980 Irpinia earthquake. Numbers in large font are the observed values. Contour lines obtained by Kriging of PGA values using an exponential model with $a=100$.

Predictions improve as the range of the model is increased, as is shown by Figures 2.16, 2.17 and 2.20; however, since the method has not been widely used in seismic problems, is not possible to define a realistic range value. Based on semivariogram of data Carr and Glass (1989) determined range values of 40 km and 10 km for the San Fernando and Coalinga earthquakes, respectively. As can be seen from Figure 2.17, for very small range values, in this case around 5 km, the prediction tends to be uniform with a value corresponding to the average of the known values. Consequently, in the case presented by Yamazaki *et al.* (2000), it seems that the predicted values are influenced more by the trend component used than by the model itself.

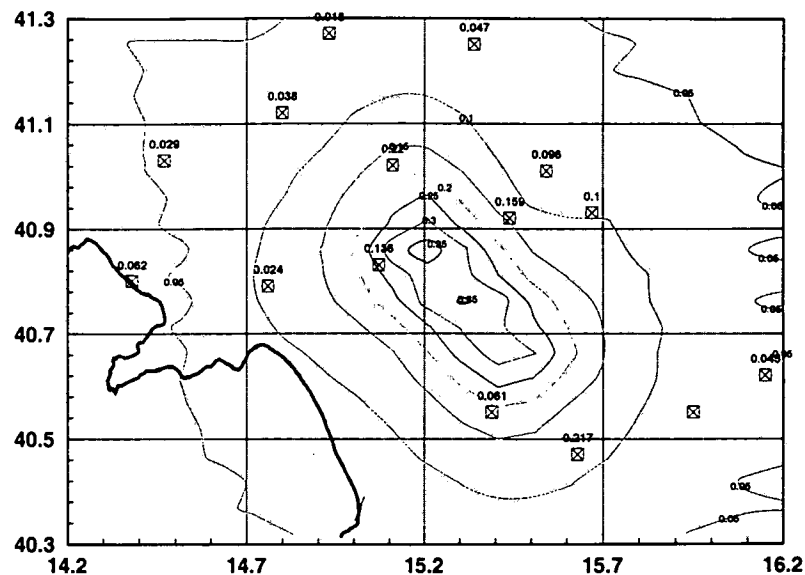
Although Figures 2.16 and 2.18 show that the spherical model predicts better the values than the exponential model with equal range value, when the spherical model was used, some of the weighting factors were found to be negative, which seems to be due to mathematical instability of the model for low range values. For this reason, comparisons of predicted values using Kriging technique are based on the exponential model results.

Similar uncertainty exists in the definition of the Nugget value as for the range value, although the influence of the Nugget value has been shown to be less important in the prediction, as can be seen by comparing Figures 2.16 and 2.19.

Improvement in the prediction due to the use of a trend component is suggested by the results shown by Yamazaki *et al.* (2000). Figure 2.21 shows the prediction of maximum PGA for the Irpinia earthquake, using as trend the Ambraseys and Bommer (1991) attenuation relationship. It can be seen that the use of a trend component improves the prediction and distribution of PGA, particularly for those points that lie along the fault strike direction and near to the fault projection. However, for points located perpendicular to the fault strike and those far from the fault projection, higher values are being predicted. Figure 2.22 shows the distribution of PGA by using the Ambraseys and Bommer (1991) relationship. It is apparent, from comparing Figures 2.21 and 2.22, that use of Kriging in the prediction of strong-motion parameters does not improve the results. However, the use of the attenuation equation alone does not predict appropriately the PGA distribution, over

predicting, in general, PGA values except for near-field stations located along the fault strike direction.

Exponential $a=50$ + attenuation



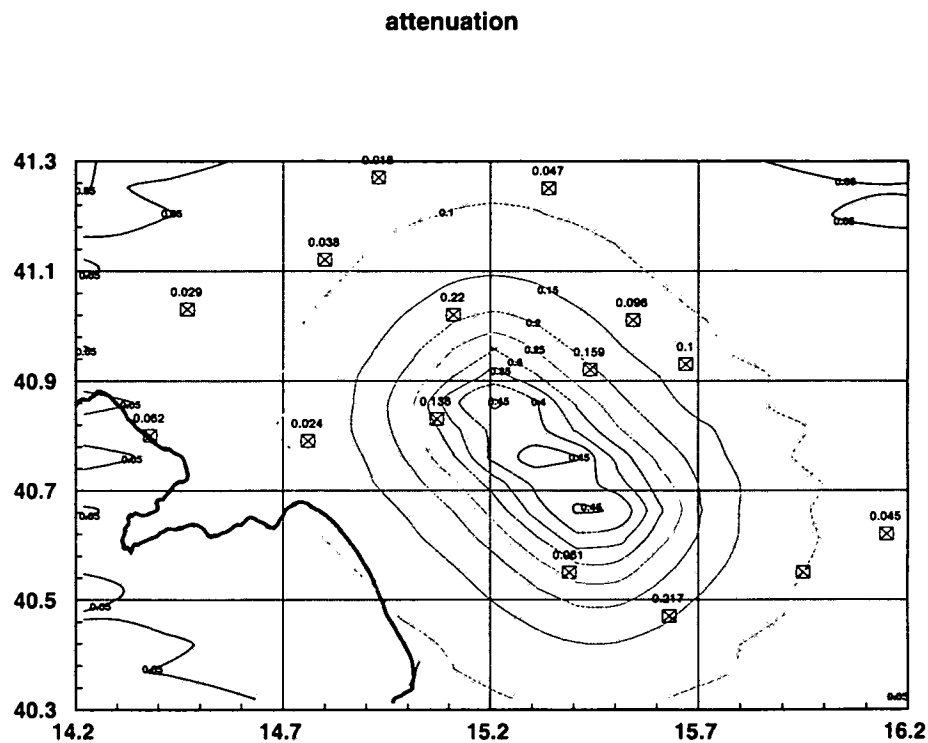


Figure 2.22 PGA distribution for N-S component during the 1980 Irpinia earthquake. Numbers in large font are the observed values. Contour lines obtained by the Ambraseys and Bommer (1991) attenuation relationship, without Kriging.

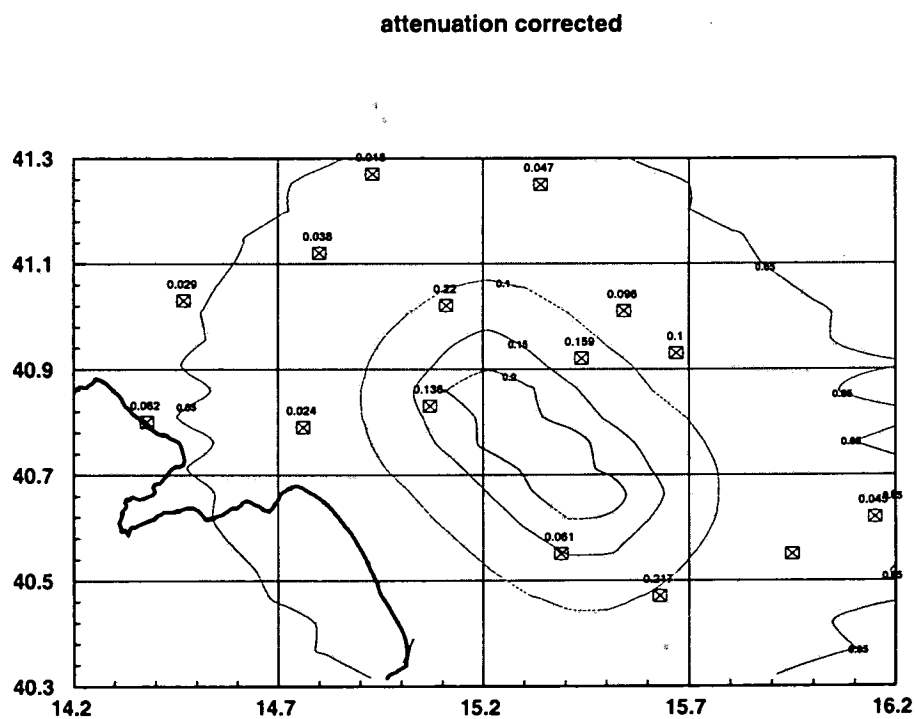


Figure 2.23 PGA distribution for N-S component during the 1980 Irpinia earthquake. Large values are the actual records. Contour lines obtained by a modified version of the Ambraseys and Bommer (1991) attenuation relationship.

2.4.2 Prediction of strong motion by using attenuation relationships

Attenuation curves are a combination of simple physical models of the earthquake and empirical data, which are linked together through statistical analysis. The simplest model used for attenuation relationships is that where the strong-motion parameter is a function of magnitude and distance. Some others include additional variables such as soil characteristics, source mechanism and focal depth. More recently, some others have tried to include directivity, rupture history and radiation pattern effects.

The basic model to represent the influence of magnitude, M , and distance, r , on a strong-motion parameter, y , can be expressed by:

$$y = b_1 e^{b_2 M} (r + b_4)^{-b_3} e^{-b_5 r} \quad (2.6)$$

where b_1 to b_5 are regression coefficients to be determined. It is, however, more common to use the logarithmic form of Equation 2.6, assuming that residuals of the value of the strong-motion parameter follow a log-normal distribution, then the expression becomes:

$$\log(y) = b_1 + b_2 M - b_3 \log(r + b_4) - b_5 r \quad (2.7)$$

The logarithms are actually used in order to avoid the difficulties imposed by solving a non-linear regression model and not to make the resulting predictive equation heavily dependent on large recorded values of the parameter.

The attenuation of strong-motion parameters with distance is taken into account as a combination of mainly two distinct phenomena, as seen in Eq. (2.7). First, the term $\log(r)$ represents the geometrical attenuation, which accounts for the spread of the wave front as it travels from the source. The amplitude of the waves decreases as they move away from the source in proportion to $1/r$, while the amplitude of surface waves, mainly Raleigh waves, decreases according to $1/\sqrt{r}$. The coefficient b_3 accompanying the geometric attenuation term is physically constrained to a

minimum value of 1, which implies spherical radiation of the wave front. Second, the damping of materials through which the waves travel, diminishes the amplitude of ground motion as an exponential function of r , referred as to anelastic attenuation. Normally, the coefficient b_5 , in the term representing the anelastic attenuation in Eq. (2.7) has a value of the order of 0.001.

Since Eq. (2.7) becomes infinity as $(r+b_4)$ approaches 0, b_4 cannot be equal to 0. The coefficient b_4 , therefore, accounts for seismic energy being radiated at longer distances than the closest assumed between the site and the source or the form $R = \sqrt{(r^2 + h_0^2)}$ is used, which also solves the problem provided h_0 is not zero.

As ground motion is affected by different factors, additional terms in prediction equations are found to allow for these effects. Usually, the inclusion of these effects is assumed as a multiplying factor through the use of dummy variables. Some approaches to include source, path and site effects in predictive equations are summarised as follows.

2.4.2.1 Style of faulting

Differences in strong-motion parameters due to style of faulting and earthquake type have been recognised from theoretical studies (Somerville and Sato, 1998) and from strong motion data (McGarr, 1984). Some attempts have been made to include this factor in attenuation relationships. Somerville (1998) presents relations for different types of earthquakes and different rupture mechanisms developed for New Zealand, but no major details are presented. Other relationship include the earthquake style as a dummy variable (Youngs *et al.*, 1997; Sadigh *et al.*, 1997). In these cases, the Eq. (2.7) becomes:

$$\log(y) = b_1 + b_2 M - b_3 \log(r + b_4) - b_5 r + b_6 F \quad (2.8)$$

where F is strictly binary (e.g. 0 for reverse fault and 1 for normal fault). If more than two focal mechanisms or earthquake types are considered in the classification, additional terms have to be included in the form of $b_i F_x$.

2.4.2.2 Magnitude

Different magnitude scales have been used to developed attenuation relationships. It is sometimes stated that because M_L is determined from an instrument with a natural period in the range of greatest engineering interest, M_L should be preferred as the measure of the earthquake size in making ground motion estimates (Joyner and Boore, 1988). However, catalogue values of M_L for large earthquakes are commonly poorly determined, and moment magnitude is a better measure for such use. Some relationships have used M_S for earthquakes with M_L and M_S greater than or equal to 6.0 and M_L for earthquakes with M_L and M_S less than 6.0 (e.g., Campbell, 1989).

2.4.2.3 Depth

Focal depth is sometimes included in the distance parameter, r . However, some attempts have been made to differentiate strong motion from crustal earthquakes and from deep subduction-zone earthquakes (e.g., Dahle *et al.*, 1995; Zhao *et al.*, 1997). In these cases, the Eq. (2.7) becomes:

$$\log(y) = b_1 + b_2 M - b_3 \log(r + b_4) - b_5 r + b_6 D \quad (2.9)$$

where D represents the depth scenario and is strictly binary (e.g., 1 for interface earthquakes and 0 otherwise). There are other subduction zone relationships (e.g. Crouse, 1988), where the depth is included explicitly as a term, with a positive coefficient, to account for the lower levels of attenuation in the more competent, deeper layers.

2.4.2.4 Source-to-site distance

Various measures of source distance have been used in the development of attenuation relationships, as explained previously and shown in Figure 2.12. The most common functional form to define the dependence of strong motion with distance express r as:

$$r = \sqrt{d^2 + h^2} \quad (2.10)$$

where h represents the hypocentral depth or a fictitious depth determined together with the other regression coefficients, and d the distance parameter (e.g., epicentral distance, distance to fault rupture or distance to fault projection).

2.4.2.5 Directivity

Near source recording sites are more likely to be affected by directivity, and the predictive equations must reflect this. Although directivity has not been widely included in predictive equations, Somerville *et al.* (1997) developed an empirical model for specifically incorporating near-fault effects in ground motion response spectra. Based on empirical analysis of near-fault data, they developed modifications (multiplicative factors) to empirical strong ground-motion attenuation relationships to account for this particular effect on amplitudes and duration. Amplitude variations were considered to be dependent on two geometrical factors. First, the smaller the angle between the direction of rupture propagation and the direction of the waves travelling from the fault to the site, the larger the amplitude. Second, the larger the fraction of the fault rupture that lies between the hypocentre and the site, the larger the amplitude. Duration was expected to depend on the same two factors, with an inverse relationship between duration and amplitude.

2.4.2.6 Hanging wall effect

Effects of the hanging wall and footwall on ground motions during the Northridge earthquake were analysed by Abrahamson and Somerville (1996). For this case an empirical model was developed to account for the increase in PGA at hanging wall sites. This model results in up to a 50% increase over a distance range of 10 to 20 km relative to the median attenuation for the Northridge earthquake. These results were implemented in the Abrahamson and Silva (1997) attenuation relationship.

2.4.2.7 Surface geology

Influence of surface geology has been commonly included by dividing recording sites into different categories (e.g., soil deposits: soft or stiff, or rock deposits). Depth of superficial deposit has also been considered in some classifications. Distinction between site categories is normally based on lithologic description, but in recent predictive relations shear wave velocity over the uppermost 30 m has been used as the parameter (e.g., Boore *et al.*, 1993).

Site categories have also been incorporated into predictive equations by using dummy variables (e.g., Dahle *et al.*, 1995; Ambraseys and Bommer, 1991). In this case the Eq. (2.7) becomes:

$$\log(y) = b_1 + b_2 M - b_3 \log(r + b_4) - b_5 r + b_6 S \quad (2.11)$$

where S represents the soil category, in some cases including depth, and is a binary parameter (e.g., 0 for soil deposits and 1 for rock deposits). If more than two categories have been considered, additional factors $b_i S_i$ must be included. Boore *et al.* (1997) include explicitly the shear wave velocity in their relationship.

2.4.3 Methodology used for prediction of strong-motion parameters

In order to develop relationships between seismic parameters and landslide characteristics and their spatial distribution, it is necessary to determine these parameters for each site where individual landslides were observed. The earthquake-induced landslide database is presented as a EESE Research Report from the Imperial College; for each case distance to the epicentre and to horizontal projection of fault rupture were defined for most of the cases. Predicted seismic parameters for each site are also reported in the database.

Although the Kriging approach can be used in order to extrapolate spatial data, it was shown in a previous section that improvement in predictions is only significant when the methods is used with a trend component of the parameter to be predicted. As trend component, attenuation relationships

can be used, however, in this case predictions are defined more by the attenuation relation used than by the Kriging process itself. This leads to conclude that use of Kriging for seismic parameters interpolation is not a significant improvement in predictions compared with those obtained by using the attenuation relations alone. For this reason, for prediction of those parameters that were used in analysis, (PGA, PGV, PGD, AI and duration), appropriate attenuation relationships were used. However, where no attenuation expressions are available for any of these parameters, in some cases attenuation equations from actual data of particular earthquakes were obtained by simple regression analysis.

Actual records for the Irpinia, Loma Prieta and Kobe earthquakes were used to transform the published regional PGA attenuation equations in order to explore the parameters predictions. For each case, coefficients of attenuation equations were changed in a way that the residuals from actual records were reduced to a minimum by getting regression expressions between residual values and distance parameters. Attempts were made to reduce the residuals by using different source-to-distance parameters, as shown in Figures 2.24 to 2.28.

Figure 2.24 shows results for the Irpinia earthquake. It can be seen that transformed equations tend to predict lower values in the range of short distances. This is due to the fact that most of the actual records are for large distances and amplitude values are very small, which tends to move the transformed equation down. In this case, it seems more convenient to predict values based on the Ambraseys and Bommer (1991) equation, at least for short distances, where differences in predicted values may be significant. For large distances, due to values being small, differences in predicted values are not important from an engineering point of view. The same exercise was carried out by fixing the intercept term of regression expressions equal to zero, this is, maintaining the independent term of attenuation equations unchanged. Results are shown in Figure 2.25; in this case, although general prediction based on transformed equations is improved, Ambraseys and Bommer (1991) expression continues being the best way to predict PGA values.

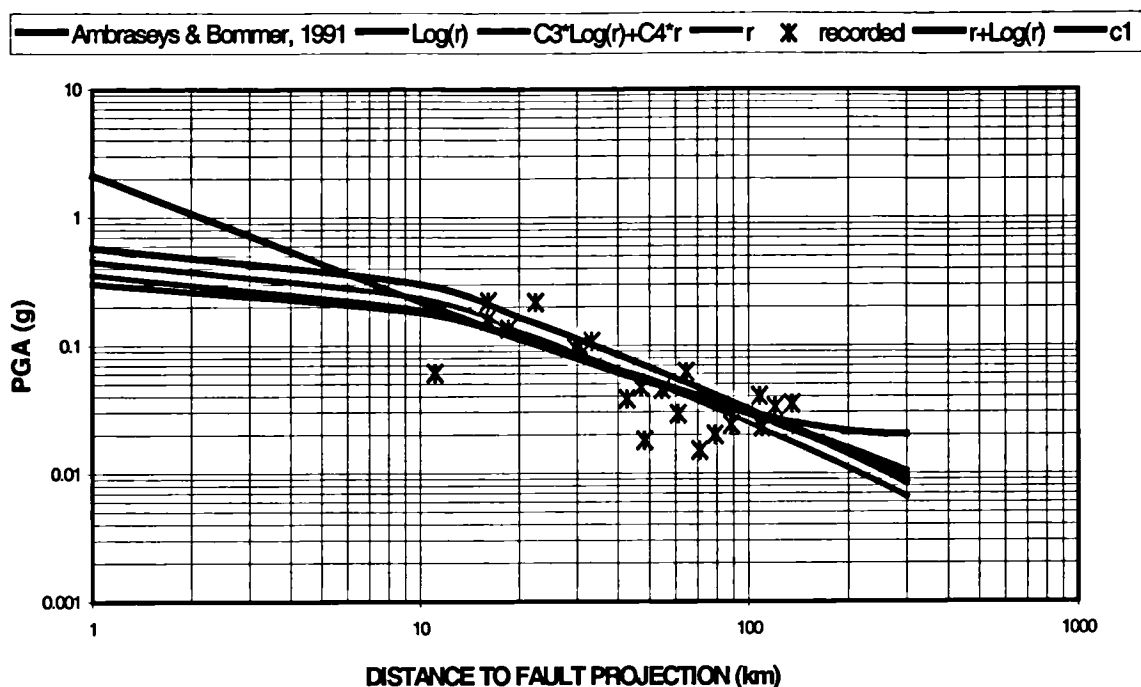


Figure 2.24 PGA attenuation relationships for the 1980 Irpinia earthquake. Solid lines represent different expressions as indicated. Those lines marked by (r), log(r) and $C3 \cdot \log(r) + C4 \cdot r$, are expression obtained by transforming the Ambraseys and Bommer (1991) expression by reducing the residuals of actual data, in each case these different source-to-distance parameters were used. Case marked by C1 represents a transformed expression in which only the intercept term was changed, in this case the line is moved with respect to Ambraseys and Bommer (1991) expression by an amount equal to the mean value of residuals.

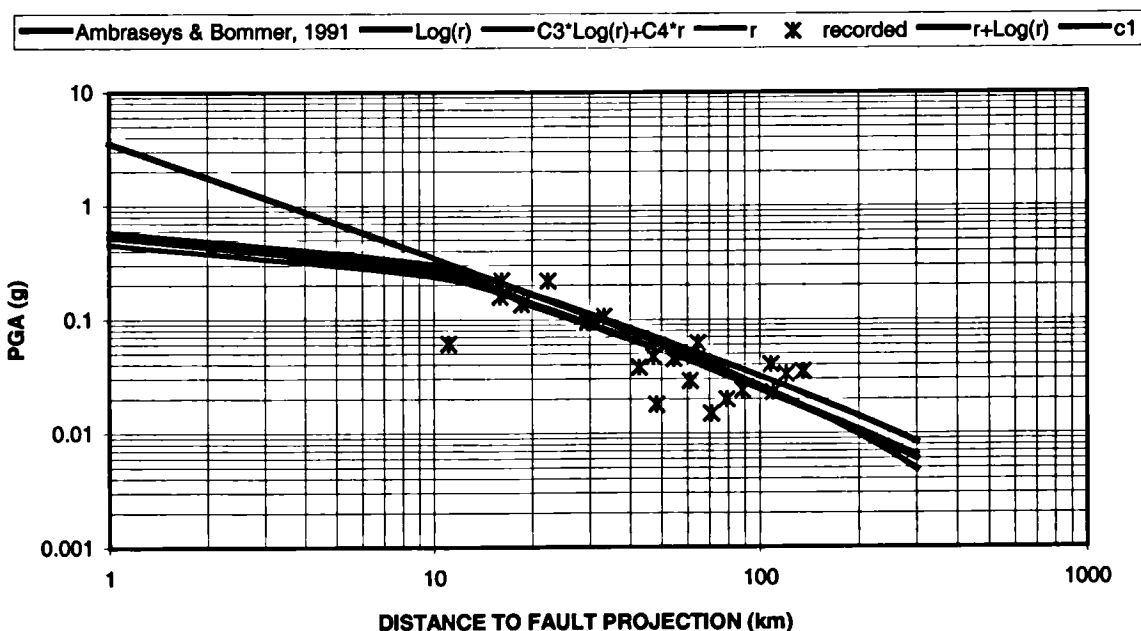


Figure 2.25 PGA attenuation relationships for the 1980 Irpinia earthquake. Solid lines represent different expressions as indicated. Those lines marked by (r), log(r) and $C3 \cdot \log(r) + C4 \cdot r$, are expression obtained by transforming the Ambraseys and Bommer (1991) expression by reducing the residuals of actual data and maintaining the intercept term, of regression expression, constant, in each case these different source-to-distance parameters were used. Case marked by C1 represents a transformed expression in which only the intercept term was changed, in this case the line is moved with respect to Ambraseys and Bommer (1991) expression by an amount equal to the mean value of residuals.

Actual records from Loma Prieta earthquake were compared with the Joyner and Boore (1988) expression, as shown in Figure 2.26. In this case actual values of PGA lie well above the attenuation expression, particularly for large distances. This makes the transformed expressions move up compared to the Joyner and Boore (1988) equation. For this case it is evident that prediction of PGA should be determined by using an attenuation expression based on actual data rather than on the published expression. The attenuation equation was also transformed in a way to maintain the intercept term, of the regression, constant; however, no improvement in the transformed expressions was found. Records from the Loma Prieta earthquake were also compared with the Boore *et al.* (1993) equation, in order to include the influence of local geology. Figure 2.27 shows results for sites of class A, as can be seen, predictions based on the Boore *et al.* (1993) expression underpredict the PGA values. The same results were found for site classes B and C. On the other hand, the transformed expressions tend to over predict PGA values, thus confirming, in this case, the necessity to work with an expression obtained from actual records rather than with published expressions.

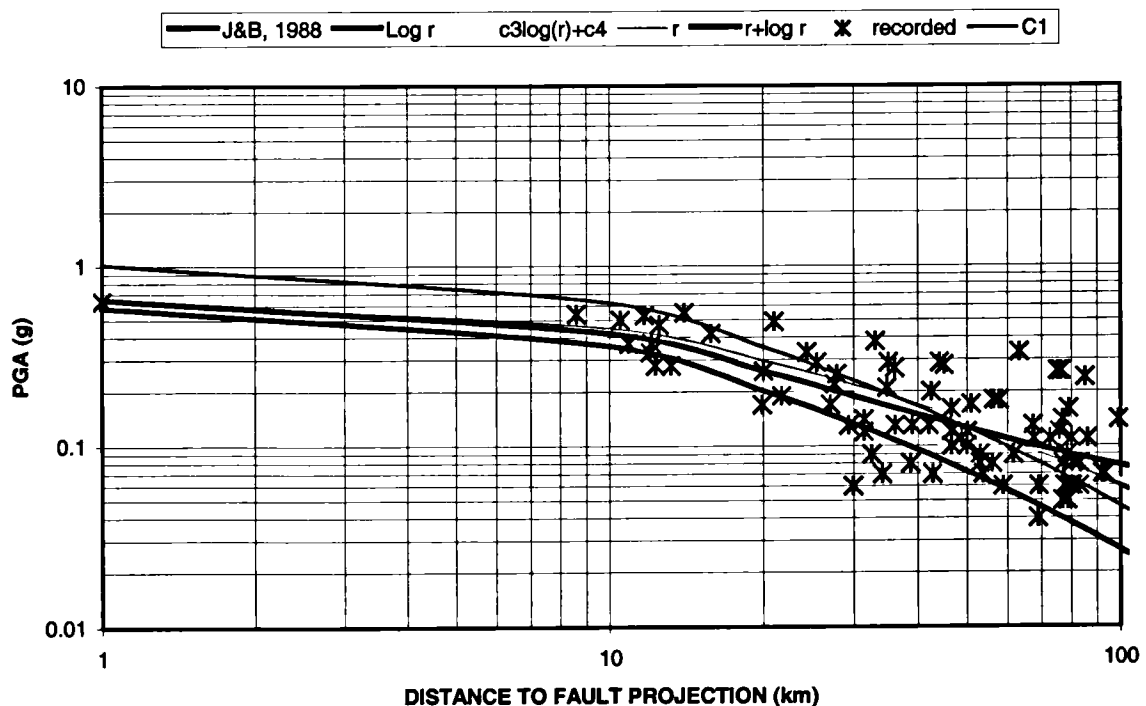


Figure 2.26 PGA attenuation relationships for the 1989 Loma Prieta earthquake. Solid lines represent different expressions as indicated. Those lines marked by (r) , $\log(r)$ and $C3 \cdot \log(r) + C4 \cdot r$, are expression obtained by transforming the Joyner and Boore (1988) expression by reducing the residuals of actual data; in each case these different source-to-distance parameters were used. Case marked by C1 represents a transformed expression in which only the intercept term was changed; in this case the line is moved with respect to Joyner and Boore (1988) expression by an amount equal to the mean value of residuals.

Actual records from the Kobe earthquake are compared with the Fukushima and Tanaka (1990) attenuation expression. Results are shown in Figure 2.28, in this case, transforming the published expression does not improve the predictions, due mainly to actual values follow quite well the Fukushima and Tanaka (1990) relationship. In this case, predictions based directly on published expression are justified.

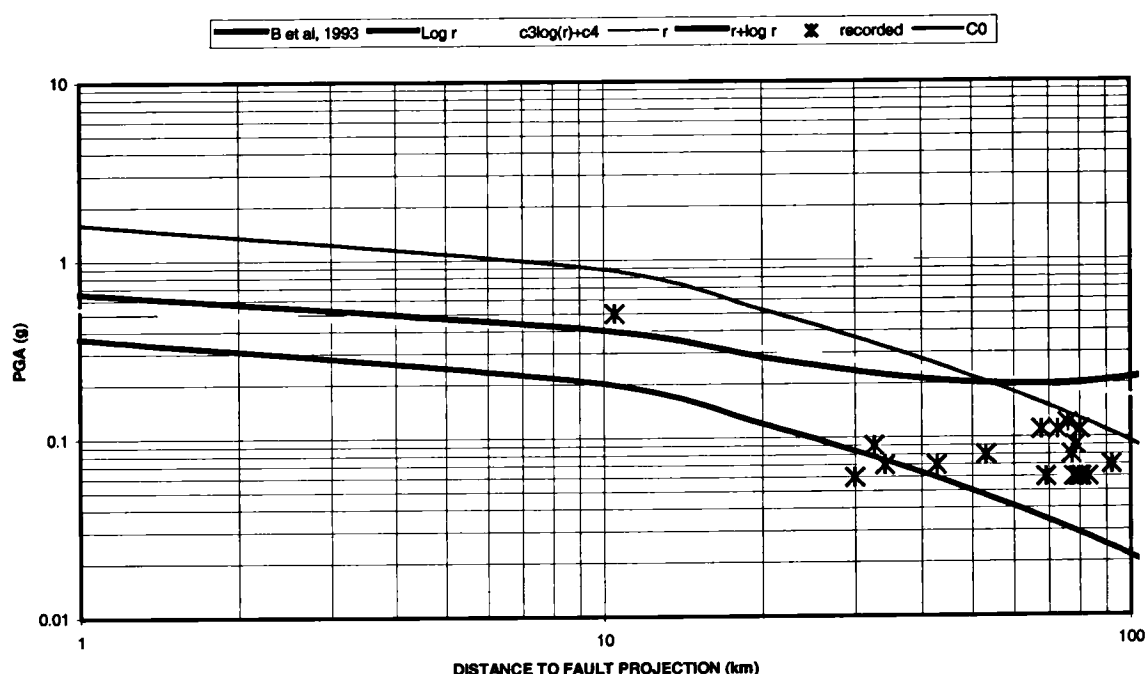


Figure 2.27 PGA attenuation relationships for the 1989 Loma Prieta earthquake. Solid lines represent different expressions as indicated. Those lines marked by (r) , $\log(r)$ and $C3 \cdot \log(r) + C4 \cdot r$, are expression obtained by transforming the Boore et al. (1993) expression, for sites type A (rock), by reducing the residuals of actual data; in each case these different source-to-distance parameters were used. Case marked by C0 represents a transformed expression in which only the intercept term was changed; in this case the line is moved with respect to Boore et al. (1993) expression by an amount equal to the mean value of residuals.

In general, when predictions based on actual records were very similar to values obtained from published attenuation expressions, the latter values were adopted for each site. On the other hand, when values from actual records were very different from those of attenuation relationships, the former values were adopted.

In some cases, as for the Loma Prieta earthquake, attenuation expressions based on actual records needed to be used to predict seismic parameters at particular sites. Figure 2.29 shows the actual data and attenuation expression used in Loma Prieta earthquake for predicting PGA. Similar expressions were obtained, by regression analysis, for other earthquakes and for those parameters

for which no attenuation relationships have been presented. In Annex B, the published expressions used to predict the different values of seismic parameters are summarised.

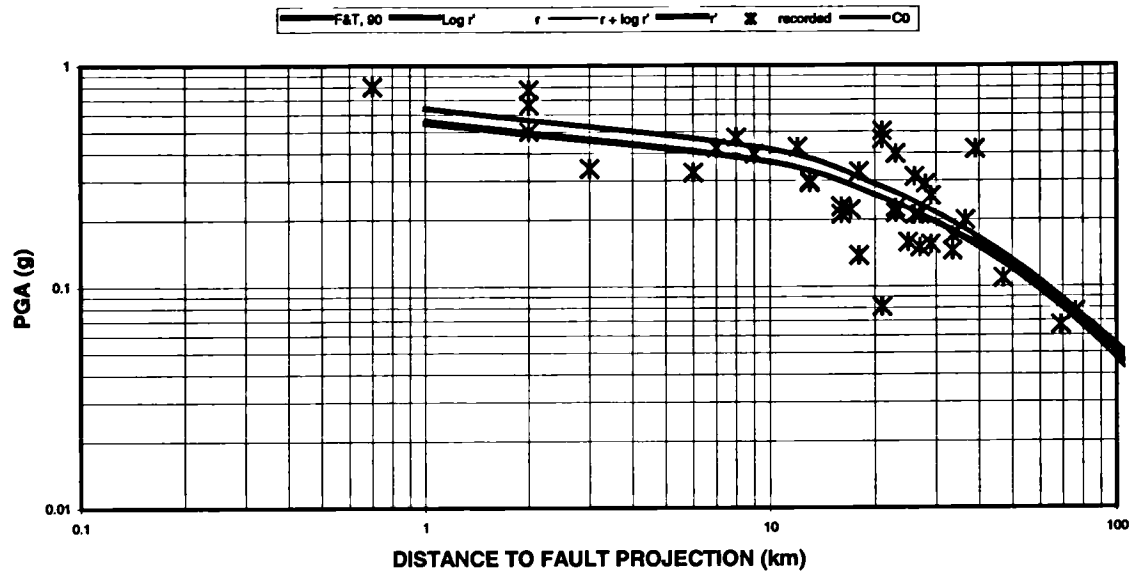


Figure 2.28 PGA attenuation relationships for the 1995 Kobe earthquake. Solid lines represent different expressions as indicated. Those lines marked by (r), log(r) and $C3 \cdot \log(r) + C4 \cdot r$, are expression obtained by transforming the Fukushima and Tanaka (1990) expression by reducing the residuals of actual data; in each case these different source-to-distance parameters were used. Case marked by C0 represents a transformed expression in which only the intercept term was changed; in this case the line is moved with respect to Fukushima and Tanaka (1990) expression by an amount equal to the mean value of residuals.

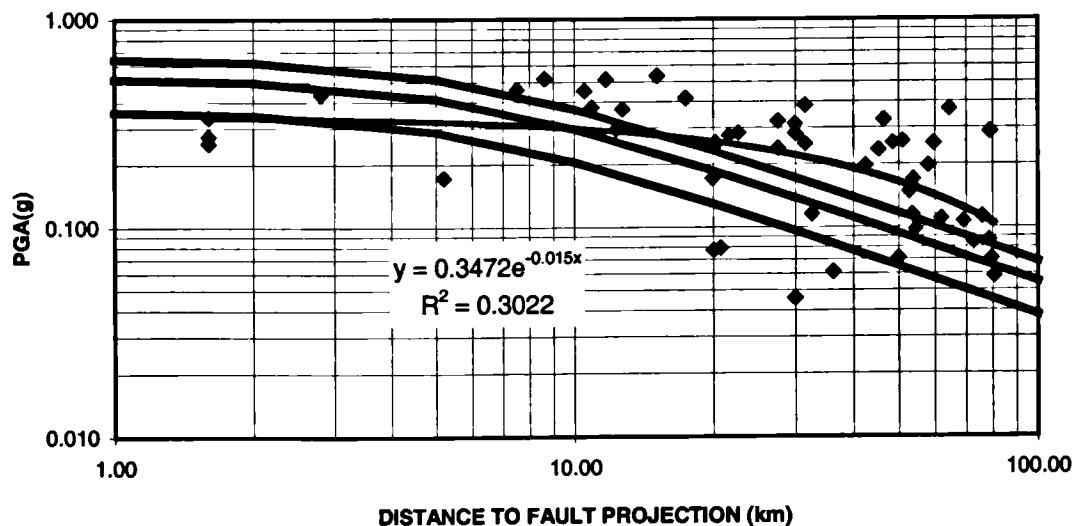


Figure 2.29 PGA attenuation expressions for the 1989 Loma Prieta earthquake. Red lines represent the Boore et al. (1993) expression for sites A, B and C (upper, middle, and lower lines, respectively). Black line represents the expression obtained by regression analysis of actual data.

CHAPTER 3

SLOPE SUSCEPTIBILITY AND SUSCEPTIBILITY FACTORS

Slope susceptibility is a term usually used to express the “landslide potential” of a slope. The most common approach to define the landslide potential is the mapping of information regarding the location and extent of observable landslides together with an assessment of how the factors that influence slope failure may be combined over the area under study. The approach is based on the assumption that the recorded distribution of landslides reflects the occurrence of combinations of factors that resulted in slope failure. If these causal factors can be identified, measured and mapped, then comparison of the distribution of different factors allows the identification of areas with varying potential for landsliding. The process is normally summarised in the following way (Jones, 1993):

1. A detailed landslide inventory map is prepared.
2. The factors potentially capable of contributing to slope failure in the area are identified.
3. Each factor is mapped, thereby creating a series of element maps or factor maps, each of which is subdivided into a number of factor zones with a relatively consistent set of conditions/values.
4. Univariate analysis is undertaken to ascertain the degree of association between the distribution of known landslides, as shown on the landslide inventory, and each factor. This is usually achieved by encoding all data for each slope, and results in a series of ratings or weighting coefficients. High ratings indicate possible associations while low rating values indicate little or no association, and the factors in question are discharged.
5. High rating factors are then combined by a multivariate analysis to create “landslide potential values”. The range of landslide potential values may be quite great depending on the number of relevant factors identified.

The different factors to be considered in the susceptibility function for the present analysis were defined from the study of those factors normally included in landslide hazard analysis under static conditions, from the study of landslide-rainfall relationships, and from landslide-earthquake relations defined for the data compiled during the present study.

3.1 Geology

It has been identified that geological factors contribute to slope susceptibility. On a local scale, where other factors can be considered constant, the geology defines not only the potential of landsliding but also the failure mechanism, landslide extent and geomorphological characteristics.

Different geologic deposits present different mechanical behaviour under external and internal loads. Strength depends not only on the kind of material but also on the process that this material has experienced from its formation, hence two similar deposits, but with different geological history, may show very different susceptibility.

Strength of geological materials and kinematic feasibility of slope failure are influenced by structural features, both on a microscale and a macroscale. Soils in very active seismotectonic environments can present microfissures that control the whole mass behaviour, whereas discontinuities and their orientation control the behaviour of rock blocks.

Groundwater conditions influence not only the pore water pressure within the soil or rock mass, but also can act as a contributor factor of instability by increasing the external loads. Hydrological response of hill slopes is a function not only of internal factors, such as permeability and antecedent groundwater regime, but also of external parameters such as vegetation and human intervention. On the other hand, changes in normal conditions induced by exceptional climatic changes depend strongly on the antecedent groundwater conditions, hence influencing the effect of climatic factors on stability.

Material behaviour is changing continually due to the material degradation caused by weathering processes, which are more effective in certain materials and in certain environmental conditions. Igneous and metamorphic rocks are more susceptible to chemical weathering than other rocks, and humid tropical climates are more effective in degrading geological materials (Varnes, 1984).

Geological factors will be considered by dividing them into four main groups: (a) material type, (b) geological structures, (c) groundwater conditions, and (d) weathering degree. Some of these factors are permanent, such as lithology and macro geological structures, whereas others are transient in nature, such as groundwater regime and weathering. These transient factors make the slope susceptibility a dynamic parameter, hence the spatial and temporal variability of susceptibility factors must be included into the analysis.

3.1.1 Material and lithology

Earthquake-induced landslides have been observed in a wide variety of geological deposits. Some attempts to relate lithology and failure mechanism (Ramanatha, 1972; Keefer, 1984, 1999) and slope susceptibility (Mora and Vahrson, 1994) have been published. These proposals are summarised in Tables 3.1 to 3.3.

Lithologies related to earthquake-induced landslides during the different earthquake included in the database compiled during the present study are reported in the earthquake-induced landslides database (Rodriguez, 2001). Frequency analysis of these data is presented in Figure 3.1. This graph shows that transported deposits under high water levels are the most susceptible to landslides during seismic events. This kind of deposits include alluvial flood plains, alluvial fans, alluvial terraces, lacustrine and deltaic deposits, and coastal deposits like beaches and tidal flats. Volcanic and residual soils have also been shown to be very susceptible compared with other soil deposits.

Landslides in volcanic (including tuffs) and sedimentary rock deposits have shown to be more common, and their frequency seems to be associated with structural controls such as the basal

discontinuity inclination and orientation, these being more critical when discontinuities dip out from the slope face. Many cases of volcanic rock deposits in Figure 3.1 are volcanic ashes, which have been shown to be very susceptible to rock and soil falls. Rarity of landslides in metamorphic and other igneous rocks seems to be due to these rocks are more susceptible to weathering, and landslides in these deposits occur, generally, along the interface between the intact rock and residual soil.

Table 3.1 Failure mechanism and susceptible lithologies to earthquake-induced landslides
(Adapted from Ramanatha, 1972)

TYPE OF FAILURE	LITHOLOGY	FEATURES OF EARTHQUAKE	DETAILS OF GROUND DAMAGE
1. Failure of slopes at limit equilibrium prior to earthquake	Fissured clays. Laminated clays. Clays with sand seams.	Just sufficient to trigger movement High rain build up of pore pressure prior to failure. Factor of safety unity prior to earthquake.	Landslides Ground cracks Damage continues after shock is over
2. Lurching and faulting leading to slides. Coseismic slides	Fault in bedrock. Stiff hard overburden soil	High magnitude earthquake. Large ground acceleration.	Landslides Ground displacement along faults. Gravity grabens
3. Liquefaction and slumping failure	Fine loose sand and silt or quick clay loess. Sufficiently thick deposit	Intensity and duration of earthquake influence extent of damage	Slumping and spreading failure, flows, vertical shortening of embankments. Additional pore pressures on liquefaction of cores of dams
4. Liquefaction and upward seepage and fissuring due to reduction of effective stress.	Thick stiff overburden underlain by fine sand or silt capable of liquefaction	Higher intensity and duration required	Fissures. Cracks in the ground, sand blows, ejection of mud
5. Loss of strength due to ground shaking	Sensitive clays, or materials susceptible to weakening by cyclic stresses	High intensity required. Long time required	Landslides. Removal of toe by wave action leads to progressive failure. Movement starts after earthquake shock
6. Effect of earthquake acceleration as inertial force	Stiff ground not susceptible to collapse	High intensity. High acceleration	Non-circular surface of failure. Large horizontal movements Movement stops with termination of shock. Ground cracks perpendicular to earthquake acceleration.

Sensitive clays have been shown to be especially susceptible to landsliding during earthquakes. During the 1988 Saguenay earthquake (Canada), landslides affected quick clay slopes very far from the earthquake source. In this case, gentle slopes were mobilised even though the seismic energy that reached these localities was very low.

Wet deposits, such as marshes and swamps, are also very susceptible. Although under static conditions deposit of previous landslides are very susceptible, movements in these kind of deposits have been shown to be not very frequent during seismic events, probably due to the rapid shear strength along precedent slip surfaces being higher than static strength (Ishihara, 1985; Lemos *et al.*, 1985).

Although few cases of landslides in colluvial and talus deposits have been compiled, these kind of geology are very susceptible not only under static conditions but also dynamically as is suggested by Tables 3.2 and 3.3.

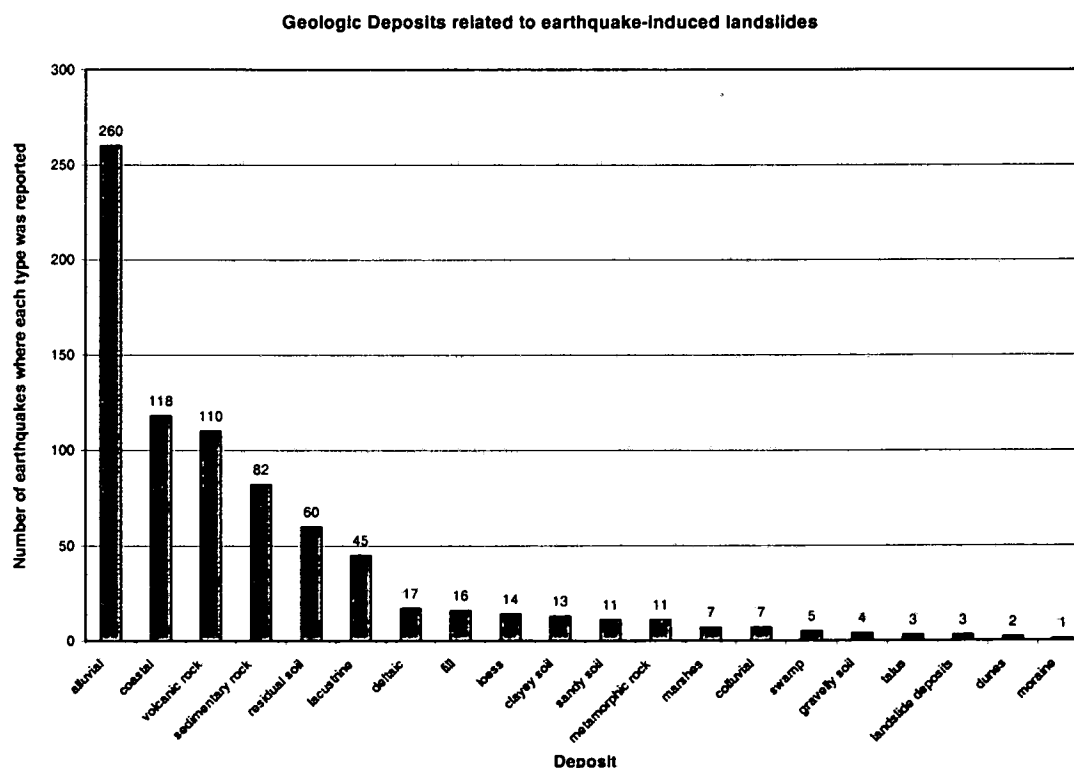


Figure 3.1 Geologic deposits related to earthquake-induced landslides for cases included in the new dataset. Number refers to earthquakes where each type of deposit was reported to be related to landslides; total number of earthquakes in dataset is 1276.

Table 3.2 Classification of different lithologies and their susceptibility. Based on cases from Costa Rica and Central America (from Mora and Vahrson, 1994)

LITHOLOGY	SUSCEPTIBILITY
Permeable compact alluvium; permeable limestone, slightly fissured intrusions, basalt, ignimbrite, gneiss, hornfels; low degree of weathering, low water table, clean-rugose fractures, high shear resistance	Low
Higher degree of weathering of above mentioned lithologies and hard massive sedimentary rocks; lower shear resistance and shearable fractures	Moderate
Considerably weathered sedimentary, intrusive, metamorphic, volcanic rocks, compacted sandy regolithic soil, considerable fracturing, fluctuating water tables	Medium
Considerably weathered, hydrothermally altered rocks of any kind, strongly fractured and fissured, clay filled; poorly compacted pyroclastic and fluvio-lacustrine soils, shallow water tables	High
Extremely altered rocks, low shear resistant alluvial, coluvial and residual soils, shallow water tables	Very high

3.1.2 Geological structures

Structures includes features such as heterogeneity and discontinuity in rocks and soils, including stratigraphic sequence, attitude of layering, gross changes in lithology, bedding planes, joints, faults and folds. Few rocks and soils are homogeneous and unfractured, and gravitational failures habitually make use of any weak zones or surfaces that may be oriented unfavourably for stability.

Based on the characteristics of earthquake-induced landslides compiled in the dataset, it may be concluded that the degree of fracturing and shearing plays an important role in determining slope stability not only for rock deposits but also in soils. Earthquake-induced landslides in fractured rocks are very common in any kind of rock, and they are mainly rock falls, in which the size of blocks depends on the discontinuity spacing. Volume and failure mechanism of exceptionally large rock avalanches, like those induced by the 1970 Perú earthquake, the 1984 Nagano (Japan) earthquake and the 1985 Nahanni (Canada) earthquake, have been shown to be controlled extensively by structural features like bedding, folding and faulting planes.

Table 3.3 Slope failure mechanism and susceptible lithologies (based on Keefer, 1984a, 1999)

FAILURE MECHANISM	SUSCEPTIBLE LITHOLOGIES
Rock falls	Have occurred in virtually all types of rocks, most were in closely jointed or weakly cemented materials such as pumice, tuff, shale, siltstone, sandstone, and conglomerate. Boulders in moraine and in residual soils. Sheared and weathered rocks. Well cemented rocks broken by joints spaced a few decimeters apart. Joints usually opened by physical weathering or stress relief or filled with weak chemical-weathering products.
Rock Slides	This mechanism involved the same lithologies than rock falls, or, occasionally, older rock-slide deposits
Rock avalanches	Most of these reported on slopes undercut by active fluvial erosion or by active, Holocene, or late Pleistocene glacial erosion. Slopes of intense fractured rock deposits broken by several sets of fractures spaced a few centimeters to a few decimeters apart. Most of these slopes also exhibit at least one of the following signs: (1) conspicuous planes of weakness-faults, master joints, bedding planes, or foliation surfaces- dipping out of the slope, (2) significant weathering, (3) weak cementation of the rock, or (4) geologic or historical evidence of landsliding.
Rock slumps	Basalt with interbedded ash and breccia, pumice, andesite, granite, greenstone, slate, schist, amphibolite, shale, siltstone, and sandstone. Most of these were weak, either because they were poorly cemented, closely jointed, weathered, or sheared. Also older rock slump deposits have been reactivated.
Rock block slides	Tuff, andesite, weak cemented pumice, closely jointed shale, mudstone, siltstone and sandstone. Basal shear surfaces are bedding planes or other discontinuities that dip out of slopes
Soil falls	Coastal bluffs, canyon walls, stream banks, terrace faces, and cut slopes
Disrupted soil slides	Loose, unsaturated, residual or colluvial sand with little or no clays. Saturated volcanic soils consisting of alternating layers of scoriaceous gravel and sensitive clay produced by weathering of fine grained ash. Saturated deposits of fluvio-glacial and landslide materials. Tills and colluvial deposits. Sandy or silty man-made fills, fault gouge, flood plain alluvium, terrace deposits, and cemented sand, silt and clay. Most of slides presented basal shear surface along soil-bedrock contacts or at boundaries between different soil layers.
Soil avalanches	Unsaturated sand, older soil avalanche deposits, and volcanic soil deposits
Soil slumps	Man-made fills usually poorly compacted sand and silt in hillslopes and coastal flood plains, particularly susceptible embankments over marshes and filled channels. Flood plain alluvium particularly in sandy materials. Other deposits with this mechanism reported are: Old slump deposits, colluvium deposits, till, alluvial fan deposits, sand dunes, terrace deposits, cemented sand and gravel, volcanic ash, pyroclastic deposits, sensitive clay, fluvio-glacial deposits, moraine, playa deposits, lacustrine deposits, deltaic deposits and barrier island deposits.
Soil block slides	Periglacial deposits consisting in interbedded sand, silt and clays. Coastal bluffs. Sensitive clays and liquefiable sand and silts. Lacustrine silts. Older landslide materials. Sandy or silty flood plain alluvium or man-made fills. Most fills on flood plains. Other materials producing this mechanism are: till, volcanic ash, colluvium, clayey playa sediments, lacustrine sediments, terrace gravel, sandy eolian sediments, sandy alluvial-fan sediments, and periglacial sediments. Basal shear surface commonly saturated
Slow earth flows	Clayey residual soils, clayey loam, till, volcanic ash, colluvium, and older earth-flow deposits. Most of these deposits bounded by shear surfaces.
Soil lateral spreads	Liquefiable gravel, sand and silts. Occasionally on sensitive clays. Granular man-made fills and flood plain alluvium. Most along abandoned river channels or in marshes. Most alluvial deposits were Holocene, and a few Pleistocene. Basal zone commonly saturated. Other deposits presenting this mechanism are: deltaic deposits, sand dunes, coastal sand spits, alluvial fan deposits, lacustrine sediments, beach deposits, beach bar deposits, estuarine deposits, periglacial sediments, barrier islands, tidal flats, terraces not in flood plains, glacial outwash, glacio-lacustrine deposits, playa sediments, organic marine mud, carbonate silt in thermal springs, and sensitive clays.
Rapid soil flows	Loess in terraced hillsides and walls of U-shaped valleys. Saturated cracked loess deposits. Volcanic soil consisting of interbedded layers of scoriaceous gravel and ash containing sensitive clays. Dune sand, sandy residual and colluvial soils, silty and sandy alluvial fan deposits, flood plain alluvium, and, man-made fills. Mine tailings composed of silt and sandy silt, moderately loose sandy gravel, and reclaimed land on flood plains. Most flows in saturated deposits but not necessarily.
Subaqueous landslides	Holocene deltaic sediments, mainly composed of sand and gravel from glacial outwash. Coarse-grained till, sand spits, tidal flats, beaches, alluvial fans, and, most commonly, Holocene deltaic sediments composed of sand, gravelly sand, sandy gravel or gravel.

The attitude of bedding or jointing relative to the adjacent slope is an important factor in some geologic situations, especially when strata dip towards the slope face. Stratigraphic sequence, particularly if beds are slightly dipping, often leads to widespread instability. Interbedded shales and sandstones are commonly more susceptible to failure than either type alone, mainly because the coarser units may conduct water readily to the weaker fine-grained units, altering their pore water pressures.

In residual soils, landslides have been shown to occur along the soil and bedrock interface. In this case, the discontinuity represents a sudden change in the mechanical properties leading to clearly defined slip planes along which seepage also tends to be concentrated after rainfalls. Fissured and laminated clays have been shown to be more susceptible to landsliding than homogeneous clay deposits. As can be seen in Table 3.1, the seismic energy needed to trigger structurally-controlled landslides in these deposits is lower than that required for a homogeneous material.

Failure mechanisms in fissured residual and clay deposits have been observed to be different than failures in more homogeneous deposits of similar origin. Shallow residual soils and fissured clays tend to move as translational landslides along a planar surface, which commonly is defined by the discontinuity plane. Deeper residuals soils and homogeneous clays tend to move as rotational slides (Nishida, 1989; Ramanatha, 1972).

Lateral spreading is a very common movement mechanism during earthquakes; however, the extent and severity of damage increase when failure is structurally-controlled by weak layers. Lateral spreads during the 1964 Alaska earthquake were shown to be controlled by layers of liquefiable materials along which failure surfaces propagated to generate the immense failures reported along the coast around the epicentral area (Seed, 1968).

Near vertical slopes in brittle materials, such as volcanic tuff, present tension cracks behind the slope crown, which propagate vertically due to seismic wave action, producing soil falls as suggested by Harp *et al.* (1981) for the 1976 Guatemala earthquake. This mechanism has commonly been observed, amount other regions, in volcanic deposits in Central America, Japan,

and South America. Similar behaviour has been observed for weakly cemented loess deposits, particularly in China (Feng and Guo, 1985, 1986).

3.1.3 Groundwater conditions

Water is one of the most important factors in slope instability. Therefore, identification of the source, movement, amount of water and water pressures is very important. Types and severity of slope failures vary markedly from region to region depending on the climatic patterns of temperature and precipitation. Groundwater condition in a slope depends on the hydrological response of the slope, which in turn depends on internal factors such as permeability, and external factors such as vegetation cover. External factors change seasonally hence groundwater condition is a dynamic parameter.

Groundwater condition as a geologic factor, as presented in the earthquake-induced landslides dataset, was considered for historical cases, as simply the relative position of the water level related to slope height. Although it was not possible to determine this factor for all cases, available data show that most earthquake-induced landslides are associated with high water levels, at least above the mid-height of the slope. However, this is not a necessary condition for landslides to occur. Extensive landsliding, due to earthquakes, has been reported not only for arid zones like Perú and Iran but also for some earthquakes that occur during or after very dry seasons like the 1976 Guatemala earthquake, or the more recent 2001 El Salvador earthquakes.

Patton (1984) stated that there are three different magnitudes of the piezometric levels acting on the failure plane of a potential slide, which should be considered in stability analysis. These should also be considered in earthquake-induced landslides. These levels are:

1. The maximum short-term levels (mean of monthly readings).
2. The maximum seasonal levels (mean of annual seasonal readings).
3. The maximum long-term levels.

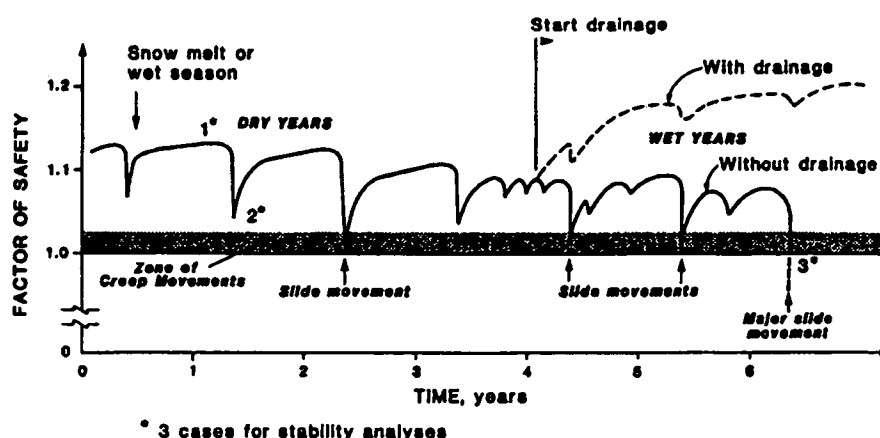


Figure 3.2 Variation of factor of safety with time for a hypothetical slide, with and without drainage (Patton, 1984)

For each of these three cases, there would be a different factor of safety, which can be suddenly changed by the influence of a triggering event as an earthquake. Fluctuations of this factor of safety with the groundwater level variation are shown schematically in Figure 3.2. Hence the slope susceptibility changes with time, and a slope may require different seismic intensities depending on the period of the year when the earthquake occurs. On the other hand, for a given earthquake the extension of damage due to landsliding has been found to depend on the time when the earthquake occurs, this is shown in Chapter 7, where models for predicting area affected by landslides and maximum distance to fault projection or to epicentre have been found to depend on precedent short and long term climatic conditions.

3.1.4 Weathering

Changes in material properties are due mainly to weathering from atmospheric agents. In the field, formation of the weathering profile depends on the parent rock, geomorphology and geological history, as well as on human activities. Interface between residual soils and fresh rock has shown to be a predominant slip surface for both rainfall-induced and earthquake-induced landslides, hence geometric characteristics and areal landsliding extension may be controlled by the spatial variation of the weathering profile. Some attempts have been made to relate failure mechanisms with weathering degree of the slope material (Massey and Pang, 1988; Nishida, 1989). Figure 3.3 shows the typical failure mechanism for residual soil proposed by Nishida (1989).

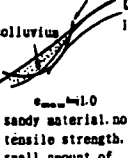
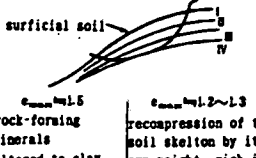
classification of soils		little-weathered		much-weathered	
		T _I	T _{II}	T _{III}	T _{IV}
soil composition	soil type and geomorphologic features	steep slope severe erosion	somewhat gentle slope	gentle to flat plateau	
	schematic expression				
	physical properties	$e_{max} \approx 0.5$ very little clay materials	$e_{max} \approx 1.0$ sandy material, no tensile strength, small amount of clay	$e_{max} \approx 1.5$ rock-forming minerals altered to clay minerals, cohesion	$e_{max} \approx 1.2 \sim 1.3$ recompression of the soil skeleton by its own weight, rich in clay minerals
failure pattern	frequency	frequent		occasionally	
	place	around summit - down to middle		near summit	
	scale	small shallow surficial slip		somewhat larger than T _I and T _{II}	
	gradient	45° ~ 50°		35° ~ 40°	
	slip surface	within colluvium between the loosening zone and colluvium		within in-situ weathered soil	
	depth of slip surface	± 50 cm		(e ≈ 1.0)	
	failure pattern	surficial slip failure		1 ~ 2m occasionally 20m	
				circular slip failure	

Figure 3.3 Slope failure patterns for residual soils (Nishida, 1989)

Earthquakes in regions dominated by residual soils have been shown to produce a large number of shallow slip-surface failures. Although the size of individual slides is very small, the extent of the area affected by them is large, and some times, particularly after intense rainfalls, the confluence of sediments produced by these slides has produced huge mud and earth avalanches that have been very destructive. This kind of process occurred during the 1935 and 1970 Papua New Guinea, the 1987 El Napo (Ecuador), the 1990 Luzón (Philippines), the 1991 Valle de La Estrella (Costa Rica), and the 1994 Páez (Colombia) earthquakes. Deep failures in residual soils, produced by earthquakes, are not common.

In rock slopes, weathering along discontinuities alters the mechanical properties of the rock mass, increasing its susceptibility to landsliding. When weathering has penetrated an appreciable distance into the rock material, mechanical behaviour could be controlled by the fill material behaviour only, and as the discontinuity becomes wider, the rock mass becomes looser decreasing its general stability. In the earthquake-induced landslides database many cases of rock failures related to weathering along discontinuity surfaces can be found.

Rock failures have also been found to have involved weathered rock masses moving over deposits of fresh or less weathered masses. This was the case of the Bairaman rock avalanche induced by the 1985 Papua New Guinea earthquake, which moved around $180 \times 10^6 \text{ m}^3$ of weathered biosparrodite over a much less weathered biomicrite along the interface between both deposits (King *et al.*, 1989).

3.2 Mechanical behaviour of the slope material

Slope stability is controlled by the mechanical properties of the material not only before the slope failure but also after the movement. As pointed by Leroueil *et al.* (1996), four different stages of slope movement can be considered:

1. the pre-failure stage;
2. the failure stage characterised by the material failure creating discontinuities along which material can slip, rotate or fall;
3. the post-failure stage, which includes movement of the material in the landslide, from just after failure and until it essentially stops; and
4. the reactivation stage, when the slipped mass moves continuously or occasionally.

Each of these stages involves mechanical phenomena, controlling laws and parameters which are very different, so it is necessary to separate these four stages to understand, analyse and characterise the movement of slopes.

During the pre-failure stage, slope stability is controlled by, amongst others, factors such as the *in situ* stress state, the stress history of the material and the rate of strain. An important feature of pre-failure movements is that they develop in the mass and not only in a shear zone. The general failure of a slope is the result of a combination of several processes. First, local failure develops under conditions that depend on the strength parameters and strain rate. Then, local stress conditions change depending on the deformation magnitude; this stress redistribution can lead to progressive

failure of the mass, which can remain limited or can extend along a shear zone through the entire mass and generate a general failure.

Post-failure behaviour depends on the potential energy (E_p) available at failure, which is closely related to the shear strength, as illustrated in Figure 3.4. As post-failure movements progress, potential energy progressively dissipates into several components:

1. Frictional energy (E_f): necessary for moving the mass over a sliding surface.
2. Breaking up and remoulding energy (E_D): dissipated by deforming the internal body of the mass.
3. Kinematic energy (E_K): accelerates the material to a certain velocity.

Dissipation of potential energy in these components depends on the failure mechanism and controls the post-failure behaviour, which depends in turn on the stress-strain characteristics of the material.

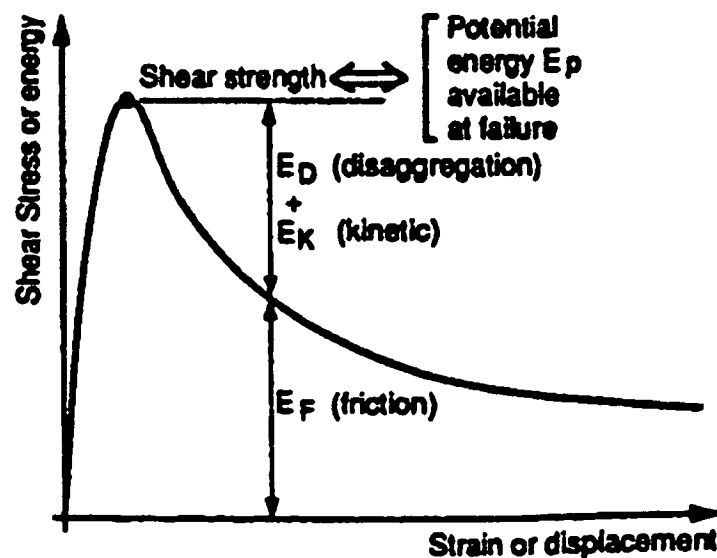


Figure 3.4 Redistribution of the potential energy after failure (Leroueil et al., 1996)

Reactivation of slides may occur when the initial failure is located along one or several shear surfaces. Strength parameters in this case are different from those of a new slide, and the mechanism and movement rate are controlled by post-failure stress-strain characteristics.

3.2.1 Rock slopes

Failure of rock under seismic loading usually involves the relative movement of blocks along discontinuities, hence strength along these discontinuities controls the seismic slope behaviour. However, soft rocks and very disaggregated deposits present failure mechanism similar to soils where failure propagation along intact material controls the slope stability. In these cases, tension strength and /or shear strength control the process depending on the failure mechanism.

In fractured rock masses, rock falls may be triggered by relatively small vibrations depending on the dip angle of the basal plane, slenderness of blocks and on the residual strength of discontinuities, as can be observed from the relations proposed by Dowding and Gilbert (1988). Rock falls have also been triggered in soft rocks like volcanic tuffs; in this case failure mechanism is related to tension cracks propagation vertically behind the slope crown produced by tension stresses induced during seismic loading. These tension stresses may be related to seismic wave reflection at free surfaces such as slope faces or open cracks (Hudson and Harrison, 1995), or to direct tension stress field generated by surface waves, particularly Raleigh waves (Noda *et al*, 1993).

Rock block slides generally move along discontinuity surfaces, whose strength controls the slope behaviour. Displacement of these slides is a function of critical acceleration, which in turn depends on the discontinuity shear strength (Dowding and Gilbert, 1988). Rock slumps are caused by shear failure propagation along deep homogeneous material, which has been shown to be very weak due to being poorly cemented, closely jointed, weathered or sheared (Keefer, 1984). A mechanism of failure propagation in rock slopes due to dynamic loading was presented by Korotkov (1990); his model is based on an elastic-plastic yield surface for shear strains.

3.2.1.1 Intact rock behaviour

Stress-strain behaviour of intact rock depends on the rock type, origin, structure, water presence, initial stress state and geologic history. Changes in physical and chemical properties of rock material

due to weathering alters the material behaviour, whereas the effectiveness of weathering agents in producing material alteration depends on material genesis and structure. Loading conditions also plays an important role in material behaviour: shear or tension stresses may produce rock failure, whereas strain rate and cyclic loading have been shown to alter the stress-strain response.

Based on lab tests, Deere and Miller (Deere, 1968) proposed an engineering classification system for rocks based on material strength and stiffness. Results show how mechanical properties of intact rock depends on rock type, rock density, cementation, fissuring, type and degree of metamorphism, and durability. The combined effect of these properties may lead to sedimentary rocks like limestone can be very strong, whereas on the other hand, some igneous rocks like tuff are very weak.

Some rocks are weakened by the addition of water, the effect being a chemical deterioration of the cement or clay binder, and/or the increase in pore pressures under undrained conditions. Pore water pressure not only reduces the rock strength but also reduces the material brittleness (Goodman, 1989). Effects of these changes are the increase of slope susceptibility, change in the failure mechanism and post-failure behaviour.

Most rocks are strengthened by confinement. As pressure increases material brittleness decreases and for large confining pressures the material becomes ductile. This brittle-to-ductile transition occurs at pressures far beyond the level of engineering interest, although in some cases like evaporite rocks and soft clay shales, plastic behaviour can be exhibited at engineering service loads (Goodman, 1989).

In situ stress state in slopes has been found to be dependent of the shape and depth of the valley, and the heterogeneity and anisotropy of the slope. Initial stress state for an isotropic slope is shown in Figure 3.5. Compared with a continuous media, the principal effect of slope presence is the reduction of horizontal normal stress and generation of shear stresses. Near the face and at the toe of the slope, vertical stresses are also reduced and shear stresses are maximum. For horizontally bedded slopes, the initial vertical stress state is similar to the isotropic condition, whereas vertical

stresses increase by a factor of three and shear stresses decrease compared with isotropic conditions (Kalinin and Sheshenin, 1990). For vertically bedded slopes, horizontal stresses are similar to isotropic conditions, whereas vertical and shear stresses decrease.

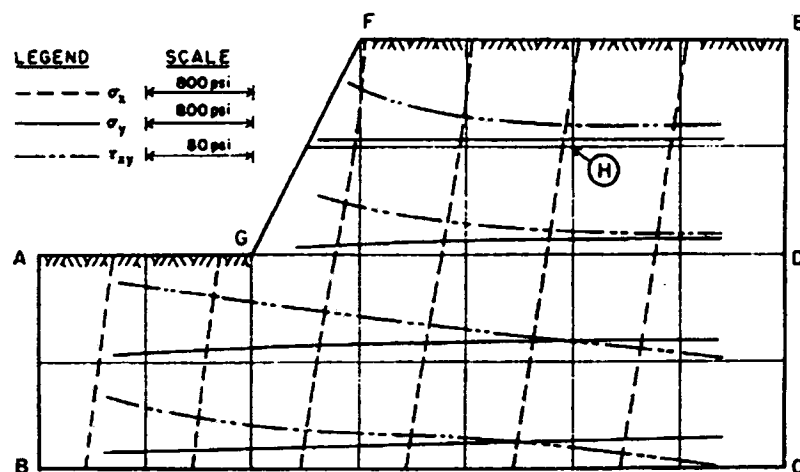


Figure 3.5 Total stresses in isotropic slope (Finn, 1965)

Kalinin and Sheshenin (1990) also found that the stress field of inclined bedding slopes depends markedly on the relation between the dip angle of bed layers with respect to the slope face; in this condition stress states show, in general, higher stress values than in the isotropic case. Vertical stresses have the greatest values on the slope, where bedded layers dip toward slope face, as shown in Figure 3.6a. At the toe of the slope, the increase in vertical stress is highest reaching up to four times the value of the isotropic case. Similar changes in stress field are observed for horizontal stresses in Figure 3.6b. Shear stresses increase by factors of four and two with respect to isotropic for bedding dipping out and into the slope face, respectively. Shear stresses have the maximum at the base of the slope as, seen in Figure 3.6c.

Valleys often coincide with the axes of anticlinal or synclinal folds. In the case of valley along the anticlinal fold, the distribution of horizontal stresses is similar to the isotropic case, but values are on average 1.5 times higher. Vertical and shear stresses have similar distributions to stress fields shown for beds dipping into the slope face, but values are on average 25 percent higher.

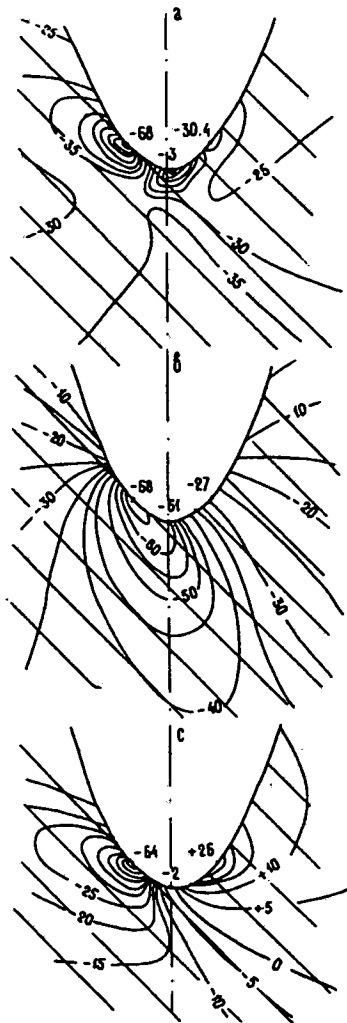


Figure 3.6 Distribution of a) vertical, b) horizontal, and c) shear stresses (MPa) on both sides and base of a valley in anisotropic rocks (Kalinin and Sheshenin, 1990).

When the valley is along a synclinal fold, the horizontal stress field is similar to that of horizontally bedded slope, but values in this case are 1.5 to 2 times lower than those for a anticlinal, but 2 times higher than the isotropic case. Vertical and shear stresses show similar distribution to the case of beds dipping out of the slope, but values are 2.5 and 3 times lower, respectively. When deposit consists of interbedded soft and stiff layers, it was found that stresses in stiff beds are higher than in soft beds. Distribution and values of stresses in stiff layers are similar to those of the anisotropic case shown in Figure 3.6, whereas for soft layers the distribution and values change considerably as can be seen in Figure 3.7.

Differences in stress states, presented in this section, may explain the different behaviour, observed on slopes at both sides of a mountainous ridge, during past earthquakes, such as the 1987 El Napo earthquake, which shown to affect in different way the slopes, of the Cayambe volcano, facing toward ESE compared with the opposite slopes (Tibaldi *et al.*, 1995).

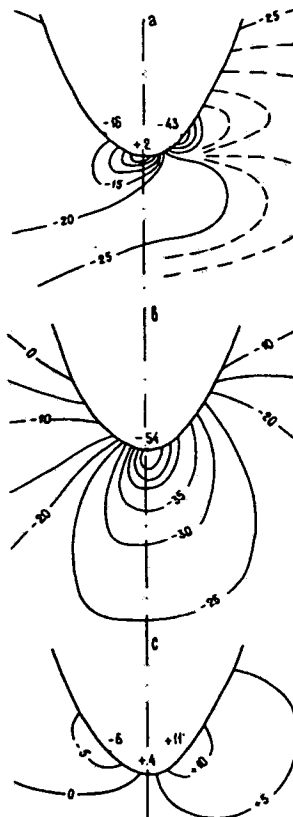


Figure 3.7 Distribution of a) vertical, b) horizontal, and c) shear stresses (MPa) on both sides and base of a valley in soft beds for a two-bedded slope (Kalinin and Sheshenin, 1990).

In slopes, weak materials may have been washed out by erosion more easily than stronger materials, and when these weak materials remain, they may present induced discontinuities caused by the field stresses. Weathering reduces the intact rock strength and increases its deformability. Stiff materials may contain higher stresses and strong rocks permit higher and steeper slopes than weak rocks. In fact topographic features, mainly slope inclination and slope height, are indicators of the intact rock strength.

Susceptibility of rock materials to weathering is a function of the formation energy of the rock material and particularly of the difference between the energy during the formation process and the

environmental energy. This difference is the available energy to be transformed by weathering processes. The energy during the formation process is a function of the material type and genesis process. Igneous rocks are related to higher energies, whereas sedimentary rocks are related to lower energies. Mineralogical components of the rock also control the susceptibility to weathering. Minerals containing ferritic and magnesian elements are more susceptible to change by weathering, whereas those with silica minerals are more stable, as is the case of quartz. Effectiveness of a weathering agent in altering material properties is also a function of the specific area available for the agent to act; this area increases with fracturing of rock masses and fissuring of rock material. As strength and deformability of intact rock depend on the degree of weathering, these consequently depend on the rock type, mineralogy and structure.

Tensile strength of intact rock has been proposed to be evaluated as 5-10% of the uniaxial compressive strength (Hendron, 1968). However, tensile strength is strongly dependent on material microstructure, anisotropy and heterogeneity. Fissuring creates weakness planes along which tension crack propagates, whereas for an anisotropic material, Amadei *et al.* (1983) found that tensile strength depends on the orientation of the anisotropy with respect to direction of loading. The minimum tensile strength occurs when the planar anisotropy is perpendicular to the induced tensile stress, while the maximum occurs when the anisotropy is parallel to that tensile stress.

Failure of rock material can also be the result of *creep*. In this case, failure is the result of accumulative strains until the stress path intersect the monotonic stress-strain envelope. A similar concept applies to cyclic loading (Goodman, 1989). Cycles of loading and unloading produce hysteresis loops as energy is consumed in sliding on cracks and fissures inside the rock volume, and the stress path migrates toward the monotonic envelope.

Based on cyclic compression tests, Homand-Etienne *et al.* (1983) show how compressive strength decreases with increasing number of load cycles at constant frequency and load amplitude. A reduction to 85% and 83% of monotonic strength was found for granite and marble, respectively. They also found that fracture propagation in static and dynamic conditions are different; for dynamic loading three phases may be distinguished: 1) microcrack propagation, which increases with

duration and loading magnitude; 2) stabilisation of cracks and deformation under constant rate, and 3) beyond a critical stress level, an accelerated crack propagation until collapse is reached.

In situ dynamic loading plate tests presented by Ishikawa *et al.* (1983) show the variation of rock properties under dynamic conditions. Dynamic modulus of elasticity and damping ratio have been found to decrease with increasing cyclic load magnitude. For low frequency values modulus shows to increase although for larger values than 1 Hz this trend is not well defined. Damping ratio has been found to increase with frequency. Similar to dynamic soil behaviour, dynamic rock properties have been shown to depend on strain level, with elastic modulus decreasing and damping ratio increasing with increasing strains. On the other hand, the modulus increases with loading rate, whereas damping ratio decreases. Similar results were found by Akai and Ohnishi (1983) for a soft sedimentary rock tested under dynamic compression.

3.2.1.2 Discontinuity behaviour

Discontinuities are the single most important characteristic of a rock mass because they dominate the deformability, strength and permeability of the whole rock mass. Discontinuities vary in their geometry and mechanical characteristics. For engineering purposes it is necessary understand these characteristics and their influence on rock mass behaviour.

Geometric characteristics common in discontinuity description are orientation, spacing, persistence, roughness, aperture, filling, number of sets and block size. Discontinuity orientation controls the kinematic feasibility of the slope and an increase in the number of unfavourable joint sets reduces the rock mass stability, constraining slope height and inclination. The magnitude and direction of *in situ* stresses, on the other hand, are dependent on the discontinuity orientation. Slope stability will depend on the relation between discontinuity orientation and slope face orientation.

Spacing and persistence determine the block size in a rock mass, which in turn controls the rock mass stability. Stability decreases with the increase of the ratio of engineering work dimension to mean discontinuity spacing. Intact rock properties may be treated as being homogeneous and

isotropic. However, due to discontinuities rock mass becomes highly heterogeneous and anisotropic, although, with relatively frequent discontinuities it again becomes more homogeneous and isotropic.

Aperture and filling establish the intercourse of adjacent blocks, which controls the rock mass stability. The wider the aperture, the lower the influence of roughness of the discontinuity wall on shear strength and the higher the influence of mechanical properties of the filling on rock mass behaviour. Aperture depends on the *in situ* stress state, whereas filling is commonly a result of weathering of rock material. Weathering susceptibility increases with a reduction in block size by increasing the specific area over which weathering agents can act.

An open discontinuity causes the rock to have zero tensile strength at that location, and similarly the presence of discontinuities significantly reduces both the compressive and shear stiffness of a rock mass. The rock mass deformability increases as the discontinuities become more frequent.

Shear strength along discontinuities depends on the roughness and filling. Roughness controls the friction angle, whereas filling controls the cohesion for narrow discontinuities or the whole strength for wider discontinuities. The higher the roughness and the closer the discontinuity, the higher the frictional strength, whereas thicker filling tends to reduce the shear strength. A decrease in friction, due to these effects, causes a reduction in safe slope height and slope angle. Drainage conditions, and consequently pore pressure changes, in discontinuities are also controlled by the nature of filling.

Strength parameters can be altered due to the presence of water within the discontinuity. Firstly, pore water pressure reduces the normal effective stress. Secondly, filling properties depend on the water content, especially if filling is a clayey material produced by weathering of rock walls.

Flow patterns in rock masses depend mainly on the discontinuity properties more than on the intact rock properties; permeability due to discontinuities is much higher than permeability of intact rock, which depends on material porosity. Local permeability can increase due to stress relief that opens

existing discontinuities or generates new discontinuities in low strength materials. Water flow through discontinuities (rather than through intact material) leads to large variations in the hydraulic conductivity values, depending on the scale of rock mass considered, and its relation to engineering works. For a small volume of the whole rock mass, permeability is higher in the discontinuity direction, whereas for larger rock mass volumes, permeability tends to be more isotropic. This effect is also applicable to larger rock mass volumes of stronger material, which presents less discontinuities compared with similar rock mass volumes of brittle materials. Flowing water can also wash out filling and thus alter the discontinuity conditions. Increase in flow through the rock mass reduces the safe slope height and slope inclination.

Cyclic load tests on shear discontinuities reported by Gillet *et al.* (1983) have shown that the strength of smooth discontinuities increases with shear strain velocity, and although the results are not conclusive, it seems that this velocity effect is independent of the normal stress level. The behaviour of rough joints appears to be very unpredictable. Geometry of the joint, mating of the surfaces, and localised degradation all contribute to the observed behaviour. Crawford and Curran (1981), referred to by Gillet *et al.*, concluded that the shear resistance of harder rocks decreases with increasing velocity greater than a variable critical velocity, and the shear resistance of softer rocks increases with increasing velocity up to a critical velocity.

Gillette *et al.* (1983) also show that the interstitial water pressure in a discontinuity subjected to dynamic displacements stabilizes early in the process and does not continue to increase with increasing displacement cycles. However, the pressure fluctuates in a manner closely related to joint dilation or contraction. The strength of the joint closely follows the normal effective stress variation.

3.2.2 Soil slopes

Slopes behaviour has been found to depend on parameters such as the *in situ* stress state, stress path and mechanical material behaviour. The Initial stress state is strongly influenced by the deposit genesis, geological history, and ground water conditions. Stress path and stress-strain

behaviour are a function of the loading process including load characteristics and drainage conditions.

3.2.2.1 Initial stress state and stress path

The initial stress condition in soil slopes depends on the slope formation process and ground water regime. Figure 3.8 shows that if an artificial cutting formed the slope, stresses moved from point O to a point like U_0 , whereas if it was naturally created by an erosion process, it will be moved from O to D. The shear strength, which governs the initiation of local failure at any point in the slope, corresponds to an effective stress condition on the limit state curve, hence the closer the initial stress condition to the limit stress line, the lower the stress increments necessary to produce the local failure.

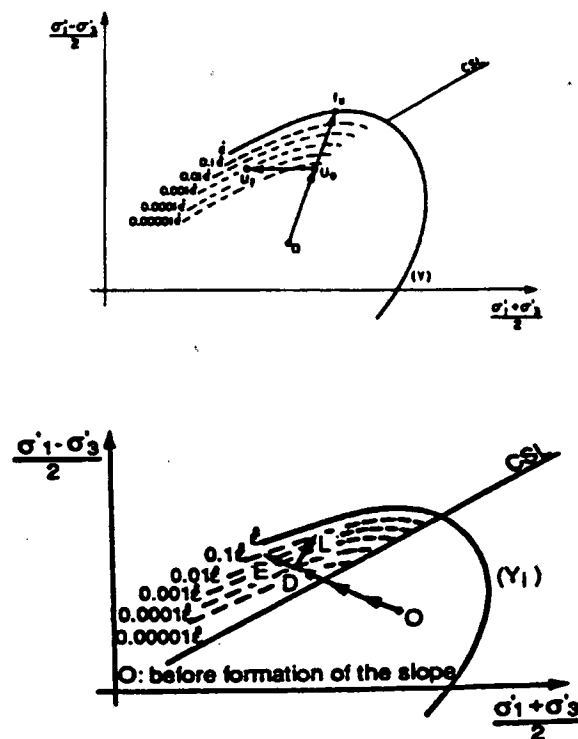


Figure 3.8 Schematic effective stress path for slopes created by cuts (top), and natural erosion process (bottom)(Leroueil et al., 1996)

On a completely dry, infinite and planar slope, the *in situ* stresses are gravity-induced and related only to slope angle, soil density and depth. The principal stress orientation can be determined by invoking assumptions on strain direction and stress rotation as suggested by Anderson and Sitar (1995). When the soil is unsaturated, capillary action provides an isotropic confining pressure that decreases the *in situ* principal stress ratio $K_C = \sigma'_1 / \sigma'_3$. However, due to the groundwater influence, the actual value of K_C changes depending on several factors related to seepage, including the depth of saturation, the seepage orientation and the hydraulic gradient. Effects due to these factors are shown in Figure 3.9.

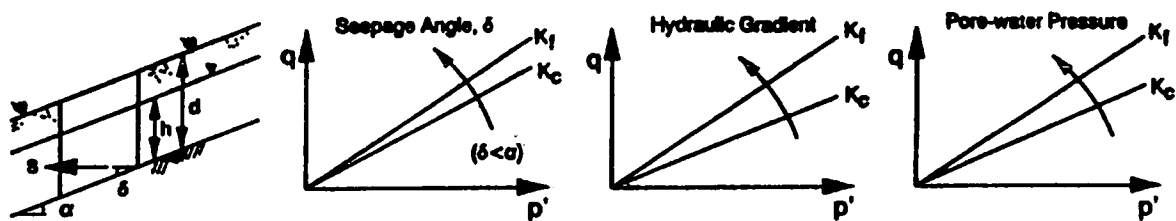


Figure 3.9 Influence of individual hydraulic parameters on K_C . From left to right: schematic planar failure model, seepage angle influence, hydraulic gradient influence, and pore pressure influence (Anderson and Sitar, 1995).

The negative pore pressure generated during the excavation of an artificial cut dissipates with time as a result of consolidation. As a result of the dissipation of negative pore pressures the effective stress condition at any point in the slope progressively moves towards the limit stress line along U_0 - U_1 path, as shown in Figure 3.10a.

Variations in pore water pressure in the slope due to a triggering factor, such as rainfall or earthquake, produce changes in the effective stress condition. For yearly or seasonal variations due to rainfall, the effective stress condition fluctuates between limits such as HW and LW, as shown in Figure 3.10b. During the periods of high pore water pressure, the effective stresses are reduced towards HW and get closer to the limit state condition. On the other hand, during dry spells or when infiltration is cut off, the effective stresses increase towards LW, away from the limit state. During cyclic loading due to seismic events, depending on the permeability, an increase in pore water pressure is produced, and effective stresses moved towards the limit state condition as shown in Figure 3.10c.

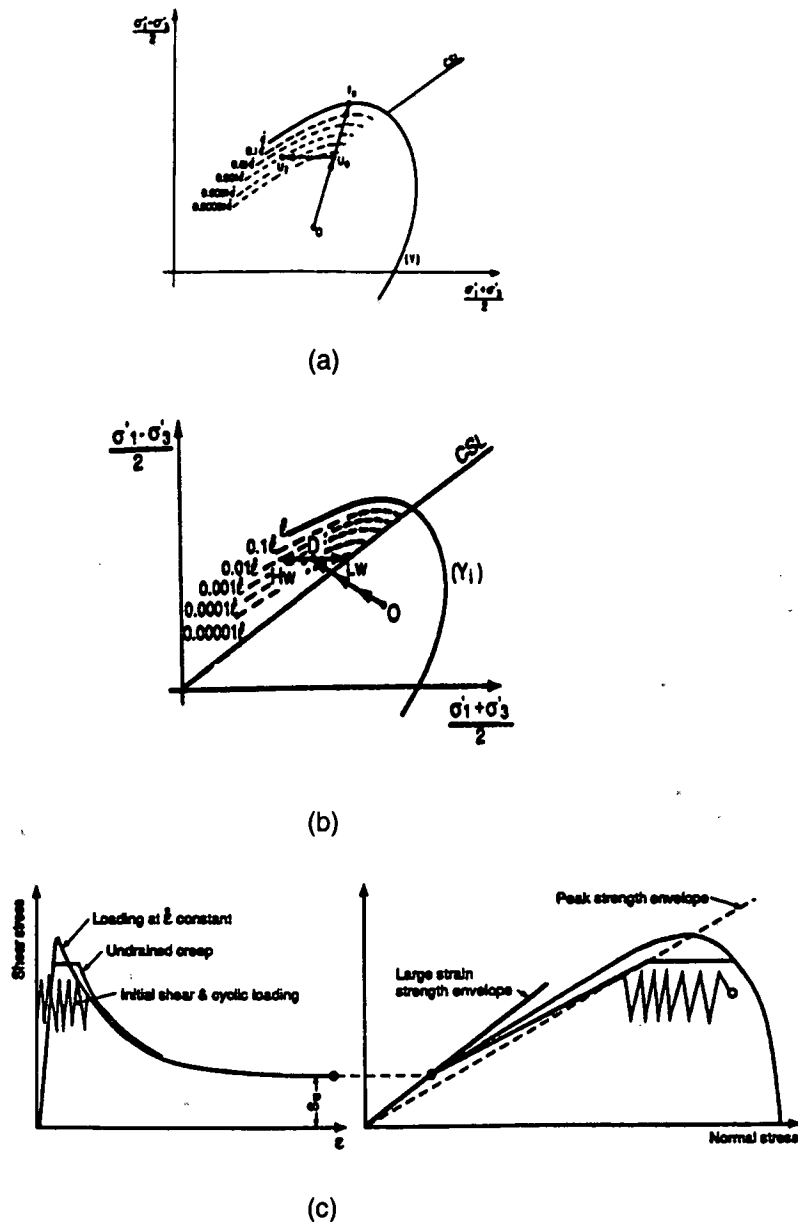


Figure 3.10 Stress paths in slopes due to different processes: a) dissipation of negative pore pressures due to excavation, b) changes in effective stresses due to fluctuation of water level, and c) stress path due to dynamic loads (Leroueil et al., 1996)

The creep behaviour of over-consolidated natural clays can be visualised as a series of lines of equal creep strain rates ϵ , more or less parallel to the limit state curve (Y), with increasing strain rates closer to (Y), as shown in Figure 3.8. Therefore, as the effective stresses move towards the limit state, creep deformations develop in the clay behind the slope at increasing strain rates, in a uniformly accelerated process. A similar time-dependent deformation behaviour may result from the continuous excavation up to failure in an undrained condition.

As duration of stress application is increased, the limit state curve moves towards the critical state line, which represents a lower limit for an infinite duration of load application. The time-dependent displacement of the limit combines with the time-dependent displacement modification of effective stresses in the slope to result in a delayed failure.

If a slope is excavated rapidly in undrained conditions up to failure, the effective stress path would be such as $O-f_u$ in Figure 3.11, and local failure would be initiated at f_u on the limit state line corresponding to the excavation rate. If the slope is excavated to such a depth that no short-term failure occurs, the effective stress condition at the end of excavation may be U_1 or U_2 , and the magnitude of shear stresses being directly related to the height and inclination of the slope.

The two time-dependent processes noted above are then initiated: the dissipation of the negative pore pressures results in a decrease of the mean effective stress along U_1D_1 or U_2D_2 , and at the same time the creep deformations of the clay result in the progressive decrease of the limit state curve and of the corresponding shear strength. In a steeper slope, with a high initial shear stress U_1 , a condition is rapidly reached where the effective stresses touch the reduced limit state curve. In this case failure is established after a relatively limited pore pressure dissipation. In a flatter slope with a low initial shear stress U_2 , a more significant pore pressure dissipation and a longer period of time is required before the local failure condition D_2 is reached.

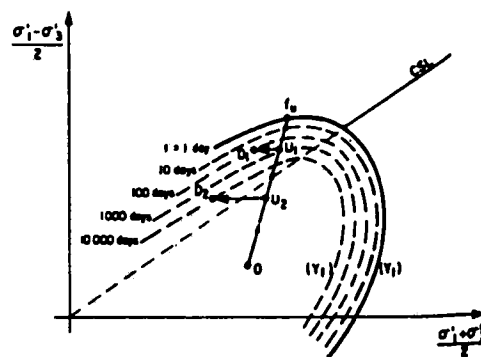


Figure 3.11 Schematic effective stress paths and development of failure in cut slopes (Tavenas and Leroueil, 1991)

3.2.2.2 Dynamic soil behaviour

Under cyclic loading progressive degradation of deformation properties of soil is observed by the reduction in shear modulus and increase of the damping ratio. Variation of these parameters has been extensively studied for different soils, and it has been found to be non-linearly dependent on the strain, and is influenced by factors such as soil type, homogeneity, anisotropy, relative density, confining pressure, stress history, characteristics of cyclic load (magnitude, duration and frequency), and drainage conditions (Ishihara, 1985, 1996; Seed *et al.*, 1986; Alarcón *et al.*, 1989; Stokoe *et al.*, 1992; Prakash and Dokoulas, 1994; Keefer and Ho, 1995; Yoshida and Iai, 1998; Pak and Yamamura, 2000). As pointed by Lee and Focht (1976), the main consequence of modulus degradation is an increase in soil displacements with cycles or loading duration, whereas increase in damping ratio represents the increase in the ductility of soil.

When a soil deposit is subjected to cyclic loading, generation of positive pore pressure is controlled by material properties and drainage conditions. Under undrained conditions a progressive increase in shear strains produces a progressive increase in pore pressures, and reduction in effective stresses, therefore the stress path moves toward the failure envelope. Cyclic failure, as defined by Yasuhara *et al.* (1992), is reached when effective stress path comes onto the failure line. The increase in pore pressure, and number of cycles needed to reach this failure, are a function of the initial stress state, the soil dynamic response, and magnitude magnitude of shear stresses induced in the soil.

After cyclic loading, soil may have been softened and weakened so that the static stress-strain properties are changed (Castro and Christian, 1976; Lee and Focht, 1976; Yasuhara *et al.*, 1992; Matsui *et al.*, 1992). This change in stress-strain properties increases the soil susceptibility for post-earthquake events, such as rainfalls or further earthquakes, as it has been reported for many earthquakes included in the present dataset. However, Yasuhara *et al.* (1992) show how in a normally consolidated clay, the effect of cyclic loading is similar to an over-consolidation produced by unloading, hence under drained conditions after cyclic loading an increase in static post-cyclic

strength must be expected due to an increase in effective stresses. The increase in undrained strength in this case has been found to be dependent on the increment in pore pressure induced by cyclic load. This implies that soil behaviour will depend on the sequence of events and the time lapse between them, which makes the analysis of earthquake influence on behaviour of slopes failed by post-earthquake events very complex.

3.2.2.3 Pore water pressures

Stability of slopes are markedly controlled by the pore pressure distribution along the potential shear surface. These pore water pressures (positive or negative) depend on many factors and are difficult to predict.

Under normal conditions, pore water pressure response to rainfall depends on factors such as vegetal cover, topography, soil stress-strain behaviour, soil fabric, soil heterogeneity, antecedent moisture condition, seasonal and yearly rainfall patterns, and position of the water table. These factors will be discussed in Section 3.4.

In a uniform slope, a saturated zone first develops at the base of the slope and progresses upwards with continuing rainfall. This suggests that downhill slopes would be most susceptible to slope failures. However, due to convergence of water flow, pore pressures in convex-concave slope transitions are significant, and slopes become more susceptible. On the other hand, in these zones antecedent moisture condition tends to be higher due to delays in water migration. However, earthquake-induced landslides are not confined to these locations and they have been seen at all parts of hillslopes. As was suggested by Kobayashi *et al.* (1998) based on data of earthquake-induced landslides during the 1995 Kobe earthquake, in convex slopes, ground shaking effect is stronger, and for concave the slopes ground water effect is more pronounced.

Pore pressures are strain dependent and are a function of the material properties and of the strain rate. The two extreme slope failure situations that may occur are rapid failure, which takes place

under constant water content, and slow failure, which takes place under approximately constant suction conditions (Anderson and Sitar, 1995).

Suction during constant water content shearing depends on the mode of shearing. A dilatant mode causes suction to increase throughout shearing, while a compressive mode results in a suction decrease. This leads to the following hypotheses: 1) If the soil in a potential failure surface will fail in a compressive mode, the suction will decrease rapidly, and the suction loss due to infiltration alone is sufficient to reduce the strength to a level where significant strain will occur. Rapid collapse will result, therefore, because the suction will decrease rapidly throughout the shearing process; and 2) If shear along the potential failure surface will take place in a dilatant mode, the strain-induced suction will increase as shear occurs and this will tend to counteract the losses in suction due to infiltration. These two situations can be represented by the same field stress path, but the rate at which a compressive soil moves along this path is much faster than the rate in a dilatant soil.

Joints of residual soils or fissured clays can control infiltration rate, hence in some cases macro fabric can control the pore water pressure increase. In this case, water flow may be dominated by flow through a few large openings or voids. These paths control the short-term pore pressure response, whereas matrix flow will control pore pressures in the long-term.

Pore water pressure of a slope is influenced by the initial water content, which during rainfall events controls the infiltration rate and the wetting front advance rate. During the rainfall recession, saturated zones become unsaturated at a rate that depends on the flow convergence, being quicker in divergent slopes. Antecedent moisture content is a function of rainfall pattern, hence pore water pressure is also controlled by this pattern. The length of precedent time related to pore water pressure changes is a function of the size and shape of recharge area, or aquifer geometry, and also of the hydraulic conductivity of the material.

Pore water pressures necessary to develop soil failure depend on the initial principal stress ratio, as was explained previously. This increment in pore water pressure can be produced by cyclic loads of different characteristics. Principal features that define a cyclic load are amplitude, duration and

frequency. In general, under undrained conditions, the increment in pore water pressure necessary to produce a slope failure can be achieved by a high monotonic load applied during a long period or by a smaller load applied cyclically. In the latter case, the duration required to induce the failure depends on the dynamic material response, the initial stress state, the cyclic load characteristics, and the drainage conditions. Few actual records of pore pressure increments induced during earthquakes have been reported. Harp *et al.*, (1984) and Mavko and Harp (1984) presented pore water pressures recorded during aftershocks of the 1980 Mammoth Lake earthquake from lake shore sediments at Convit Lake that had undergone liquefaction and lateral spreading failure. They show how pore pressure records have essentially the same frequency content as the acceleration record, especially in the P-wave portions of the record, and most records show the pore pressure to be in phase with the vertical component of P-waves. The S-wave portion of the pore pressure records shows the greatest similarities to and are influenced most by horizontal accelerations. A critical value of horizontal accelerations seems to be needed to produce excess pore pressure depending on the level of strains induced by seismic waves and/or drainage conditions. For this case a critical value around 0.15g was reported.

Ishihara *et al.* (1981) show pore pressure and horizontal acceleration records triggered during the 1980 Chiba earthquake from a loose sand deposit on Owi Island in Tokyo Bay. Data show that pore pressure started to increase when the horizontal acceleration at the ground surface reached 0.06 g. Increase in pore pressure, and its subsequent dissipation, seems to be dependent on depth, soil type and drained conditions. Records for the Wildlife liquefaction array in the flood plain of the Alamo River in the Imperial Valley, California, triggered by the 1987 Superstition Hills earthquake, were presented by Holzer *et al.* (1989). In this case pore pressure started to develop after the horizontal surface acceleration reached 0.21 g, and almost half of the excess pore pressure developed after the 90% of the seismic energy had propagated through the site. The delay in pore pressure increase and the subsequent liquefaction was considered to be due to a redistribution process of pore pressures.

In some circumstances, important increases in pore water pressures are dependent on the increase of the water table due to infiltration of surface or subsurface water. This build up of pore pressure

depends on the rate of water table increase, which in turn depends on the initial water table depth. For an infiltration process, the deeper the water table, the higher the delay in pore water pressure response. During seismic events, the increase of water table as a response to coseismic release of strains after fault rupture has been frequently identified (Rexin et al., 1962; Cottechia *et al.*, 1986; Muir-Wood, 1993). This increase in water table has shown to take some time, usually months, after the earthquake to be completed, hence delay in slope response in these cases has been observed by post-earthquake landslides (Cottechia *et al.*, 1986).

3.3 Topography

The origin and dynamics of the slope movements are influenced by slope geometry. Slopes may be classified based on their slope inclination, profile and plan shape (Small and Clark, 1982).

3.3.1 Slope inclination

A rapid evaluation of slope stability can be made on the basis of geologic units and slope inclination and their relation with previous landslides. Slope angle controls the initial stress condition and the driving force that induces the movement. Earthquake-induced landslides have also been shown to exhibit some relationship between slope inclination and failure mechanism.

Rock falls have been shown to occur in very high and steep slopes, generally steeper than 35°; however, the acceleration needed to induce rock fall of an individual block depends not only on the slope inclination but also on basal discontinuity inclination and on block slenderness. The minimum slope angle related to rock falls for cases compiled in this study was 26°, in the Rio Blanco rock fall and subsequent avalanche triggered by the 1976 Guatemala earthquake.

Disrupted slides of rock deposits including rock avalanches have shown to occur for very steep slopes, usually steeper than 40°; however, some cases in the epicentral area, where shaking severity is very high, have occurred in flatter slopes. This is the case of the Mt. Ontake avalanche generated during the 1984 Nagoken-Seibu earthquake. The source of this avalanche was a

disrupted slide on a slope of 30°, and it seems that a combination of unfavourable height and inclination were the failure controls.

For coherent slides in rock deposits, it has been found that the minimum slope angle in which they may occur is 15°, as was the case of the Rall Drive block slide triggered by the 1989 Loma Prieta earthquake. Most of the cases have an angle steeper than 30°.

Disrupted slides in soil deposits have been triggered in slopes with a very wide range of slope inclination (8°-60°); however, an inclination between 20 and 30° has been shown to be the most common. Coherent slides in soils seem to be associated with flatter slopes; some cases have been reported in slopes with inclinations between 8° and 30°, being common in slopes as flat as 10°. Some cases have been reported for very flat slopes (<5°); these cases are usually lateral spreads related to liquefaction, which can even mobilise level ground deposits.

Shallow flows have been triggered in steep slopes, usually steeper than 30°, covered by residual soil layers over the bedrock, and it usually is associated with high saturation in wet seasons. Spreads and flows have occurred in slopes as gentle as 7.5° and most of the cases occurred in slopes flatter than 35°.

3.3.2 Profile shape

A slope can be classified according to its profile shape into three main groups: rectilinear, concave, and convex. Natural slopes can present along their profile segments of different shape, in which case a complex description must be used.

Profile shape can influence the groundwater flow regime and the seismic site response. Convex and concave profiles tend to present springs of groundwater flowing out of the slope face, whereas in rectilinear profiles flow tends to be parallel to the slope surface. Profile shape influences the flow paths and consequently the hydraulic gradient of flow. On the other hand, profile shape may

produce a greater or lesser interaction of refraction phases of seismic waves arriving at the slope, altering the dynamic shaking in the slope.

Data collected for earthquake-induced landslides are not sufficient to establish a general relationship between slope susceptibility and profile shape. However, from the present dataset it can be seen that most failed natural slopes have a complex shape generally including convex-concave transitions. Rectilinear and convex shapes have shown to be the most common when a simple shape were defined; this is in accord with results obtained for the Kanagawa prefecture (Japan), where a complex profile composed of a concave segment above a convex segment has been shown to be the most susceptible. However, for that area it was found that individual concave slopes are more susceptible than convex slopes (ISSMFE, 1993).

During the study of landslides triggered by the 1995 Hyogo-Ken Nambu earthquake in Japan, Ishikawa *et al.* (1997) considered the profile shape as an important factor. From 71 slopes in the Hyogo prefecture, 47 were convex in shape and these cases were divided into groups depending on the slope inclination and the knick angle (internal angle of the convex segment). They found that the steeper the lower slope angle, the greater the incidence of the knick angle on stability. In hillsides with the same lower slope angle, on Okushi Island, the tendency found was that the smaller the knick angle the greater its incidence in collapse, although this tendency cannot be generalised.

3.3.3 Plan shape

The plan shape for a slope may be classified, based on topographical curvature, into uniform, convergent or divergent. Convergent shapes refer to basins, whereas divergent shapes refer to spurs. The incidence of this parameter in hydrological slope response has been recognised (O'Loughlin, 1986; Anderson and Burt, 1977), and consequently its effect on slope susceptibility may be inferred.

Topographical convergence produces higher precedent moisture contents, with higher and quicker slope response to rainfalls. These factors, in turn, control the increment in pore water pressure and

subsequently the slope stability. On the other hand, this convergence also controls the deposit thickness and vegetation growth, which also influence slope stability.

Ishikawa *et al.* (1997) stated that due to divergent topography, the slope vibrates as a single unit at the time of an earthquake, and this may be an important factor in analysing slope susceptibility under seismic loads. Only for a few cases, in the present dataset, could plan shape be obtained; from these historical cases it can be seen that all three topographic shapes have been related to earthquake-induced landslides. However, uniform slopes seem to be more common than divergent and convergent slopes. Available data are not sufficient to conclude about which kind of plan shape, convergent or divergent, is more susceptible during seismic loading. Convergent slopes tend to concentrate flow and to be wetter, whereas divergent slopes tend to focus the incident seismic waves.

3.4 Land use

Natural conditions control slope susceptibility, but human activity may alter the effect of natural factors. Land use may alter the characteristics of the slope, particularly groundwater flow. On the other hand, artificial cuts and fills tend to be more susceptible to landsliding depending on the construction techniques and controls.

Land use on slopes is usually referred to the land cover and its influence on stability. Changes in slope cover may alter the hydrological response of the slope. Land cover directly affects infiltration and run-off processes. Changes in run-off volumes and flow velocity may cause erosion depending on the cover and soil behaviour. When severe erosion processes take place, they may induce slope instability by changing the geometry of the slope and thus changing its susceptibility. On the other hand, general groundwater flow regime can be changed due to erosion on the surface.

Although vegetation has long been regarded as effectively retarding hillside erosion, grass and chaparral seem to be much less effective in preventing slabs of soil from sliding than in retarding grain-by-grain erosion. The vegetal canopy and litter reduce the susceptibility to grain-by-grain

erosion due to raindrop impact and sheet-wash, but the ratio of infiltration to run-off is thereby increased, leading to a more rapid and through saturation of the soil mantle (Campbell, 1975). Roots increase the cohesion of the soil, hence vegetation removal reduces the soil strength, increasing the susceptibility (Bechini, 1993). Tropical rain forest seems to be particularly susceptible to earthquake-induced landslides as stated by Garwood *et al.* (1979), this may be a result of a critical combination of intensive rainfalls, shallow residual soils and cover overburden.

Depth and area of slips seem to be associated with vegetal cover type and rooting distribution. Vegetation has also been associated with slope angle for slips to occur. The density and variety of vegetation may also affect the width and shape of slab failures. During the January 1969 rainstorm in Southern California, for slopes covered with grass, mustard and sage, scars were as much as several metres wide, whereas in areas covered by the strongest chaparral, the scars were generally only a few metres wide and relatively longer (Campbell, 1975).

Organic coating of soil particles and vegetation burning may produce hydrophobic properties on surface layers. This effect reduces the infiltration rate and increases the run-off inducing erosion process, which lead to reduced slope stability.

No attempts have been made to relate the incidence of land use in earthquake-induced landslides, although many cases have been related to construction activities, particularly road cuts and fills. During the present study it was impossible to define land use for most of the cases, hence no extra conclusions can be made about this subject.

3.5. Climatic factors

Types and severity of slope failures vary markedly from region to region depending on the climatic patterns of temperature and precipitation, as well as on the soils and weathering products characteristics of each climatic region. In arctic regions, frost action and extreme ranges in temperature may induce rock falls. In temperate regions having moderate rainfall, seasonal fluctuations in groundwater flow and pressures may lead to periodic activation of deeper slides. In

semi-arid regions, subjected to infrequent but heavy rainfall or high temperature, and in tropical regions, in which severe rainfall events due to monsoons or cyclonic storms occur, damaging debris slides may be triggered over considerable areas in a short time (Varnes, 1984).

No specific study about the climatic incidence of rainfall patterns on earthquake-induced landslides has been carried out. However, some of available hazard assessment methodologies have included this factor in the analysis, as will be shown in Chapter 4. In slope instability zonation, it is usual to consider climatic factors as a triggering factor, but in this case, where seismic loads are considered as triggering factor, rainfalls can be taken as a factor that modifies the slope susceptibility.

Mora and Vahrson (1994) presented a model in which they consider the precipitation as a triggering factor, which is added to the seismic factor to define a combined triggering factor. In this case, monthly precipitation and evapotranspiration were taken as relevant climatic parameters, in order to define the soil moisture antecedent condition. However, the model was established for Costa Rica, which is a very rainy country, therefore an extension of this model to other regions require modification of this parameter.

For rainfall-induced landslides, it has been identified that a combination of precedent rainfall (long-term rainfall) and event rainfall (short-term rainfall) is necessary to trigger landslides. Antecedent rainfall at the time of the storm has to exceed a minimum threshold; furthermore, duration and amount or intensity of the event rainfall have to exceed certain levels to be able to trigger landslides. In some cases, these threshold values are related to the landsliding severity in terms of landslide morphometric characteristics and areal extension (Michiue, 1985; Wieczorek, 1987; Bechini, 1993).

The significant period of antecedent rainfall varies from days to months, depending on local site conditions, particularly soil permeability and thickness. In the case of highly permeable soils, the period of necessary antecedent rainfall may be very short, possibly hours, or the required amount of rainfall may be supplied by the early part of the storm.

Many attempts have been made to relate landslides to rainfall, some of which are based only on precedent rainfall characteristics, other are based only on event rainfall characteristics, and finally a third group include both event and precedent rainfall conditions to define slope hazard. Tables 3.4 to 3.6 summarises some of these approaches.

Table 3.4 Antecedent rainfall parameters used in rainfall-induced landslides analysis

PARAMETER	EXAMPLE	REFERENCE
1. Antecedent seasonal rainfall	La Honda (California)>279 mm	Wieczorek (1987)
2. Cycle Coefficient (CC)		Guidicini and Iwasa (1977)
3. Precipitation Coefficient (PC)	Tuscany (Italy): for 15-30 days	Canuti <i>et al.</i> , (1985)
4. Antecedent Soil Water Status (SWSa)		
5. Antecedent Precipitation Index (Pa)	(for ten days rain)	Crozier (1986)
6. Antecedent Precipitation 15 to 30 days two weeks before two days 7 days to two months 3 or 4 days	Tuscany area, Italy Hiroshima and Kure, Japan Coastal Alaska La Honda area (California)	Canuti <i>et al.</i> (1985) Abushi (1979)/Michiue (1985) Sidle (1992) Wieczorek (1987) Brand <i>et al.</i> (1984)
7. Annual PTn (Minimum probability threshold)	Durban, South Africa: 860 mm	Garland and Oliver (1993)
8. Annual PTx (Maximum probability threshold)	Durban, South Africa: 1225 mm	Garland and Oliver (1993)
9. Seasonal PTn	Durban, South Africa: 456 mm	Garland and Oliver (1993)
10. Seasonal PTx	Durban, South Africa: 798 mm	Garland and Oliver (1993)
11. Multi year pattern 5 years	Northwestern Washington/Hanzel Landslide	Miller and Sras (1998)
12. Mean annual rainfall	Contra Costa County, California:>56-74 cm	Nilsen and Turner (1975)
13. Maximum annual rainfall		Nilsen and Turner (1975)

Due to lack of rainfall data, particularly for short-term periods, it was very difficult to define short and long terms for earthquake-induced landslides analysed in this study. Rainfall records for the 1980 Irpinia earthquake were used to define these terms, as will explained in Chapter 7. Based on those data a period of one month previous to the earthquake shock was defined as short term, and previous annual and six months rainfalls were selected as long-term parameters. For cases in which average values of these rainfalls were obtained, excess or deficit of rainfall compared to average values were also calculated for the same periods. Finally, with monthly available data, Event

Coefficient and Cyclic Coefficient as defined by Guidicini and Iwasa (1977) were also calculated; in this case Cyclic Coefficient was determined for six months rainfalls. Cyclic and event coefficients represent the percentage of annual rainfall fell during the rainy season and the rainy event, respectively.

Table 3.5 Event parameters used in rainfall-induced landslides analysis

PARAMETER	EXAMPLE	REFERENCE
1. ID Parameter	La Honda, California: $ID_{5,1(mm/h)} > 3h$	Wieczorek (1987)
2. Event Coefficient		Guidicini and Iwasa (1977)
3. Exceptional events	Recurrence periods Tuscany area, Italy: 10 to 20 years Coast Ranges, California: 10 to 25 years Southern California: 75 to 150 years Norway (Debris Flows): 50 to 150 years	Canuti <i>et al.</i> (1985) Campbell (1975) Campbell, 1975 Sendersen <i>et al.</i> (1996)
4. Ratio of rainfall intensity	Japan: fair danger: $\sigma > 5$ Especially care: $\sigma > 10$	Ikeya (1976) Ikeya (1976)
5. Storage heights in tank model S ₁ S ₂ S ₁ +S ₂	Debris flows: 80 mm Debris flows: 150 mm Landslides. Kure City, Japan: 70 mm	Michiue (1985) Michiue (1985) Michiue (1985)
6. Maximum hourly intensity	Coastal Alaska Japan: 50 mm/h Hong Kong: 70 mm/h Japan: Landslides occurrence: $> 5-6$ mm/h Severe disaster: > 8 mm/h 100mm/3h	Sidle (1992) Ishihara <i>et al.</i> (1976) Brand <i>et al.</i> (1984) Onodera <i>et al.</i> (1978) Onodera <i>et al.</i> (1978) Ishihara <i>et al.</i> (1976)
7. Maximum daily intensity	Hong Kong: > 100 mm/day	Brand <i>et al.</i> (1984)
8. Total event precipitation	Contra Costa County, California: 180 mm	Nilsen and Turner (1975)
9. Daily precipitation	:130-140 mm/day with 50-75 mm/6h Japan: 200 mm/day Serra do Mar, Brazil: > 180 mm/day Manizales, Colombia: > 70 mm	Pierson (1978) Ishihara <i>et al.</i> (1976) Wolle and Hadrich (1989) Terlien (1997)
10. Excess rainfall=rainfall-soil moisture storage	Wellington area, New Zealand: $> 50-55$ mm	Crozier and Eyles (1980)
11. 10 minute rainfall		Ikeya (1976)
12. Continuous rainfall	Ash deposits from volcanic eruption : > 40 mm	Ishikawa (1994)
13. Hourly rainfall	Ash deposit from volcanic eruption: > 10 mm	Ishikawa (1994)

Earthquake-induced landslides have been triggered in areas of very different climatic conditions, with yearly, six months and monthly average rainfalls from 1.0 mm to 5600 mm, 0 to 3500, and 0 to 1140, respectively. It could be concluded that rainfall conditions is not a necessary condition for landslides to be triggered during seismic events; however, as it will be shown in Chapter 7 and 8, the mechanism and severity of hazard are in fact controlled by climatic factors.

Table 3.6 Antecedent and event parameters combined for rainfall-induced landslides analysis

PARAMETERS	EXAMPLE	REFERENCE
1. Seasonal rainfall and event rainfall	Contra Costa County, California Seasonal: >250-300 mm Event: >150-200 mm	Nilsen and Turner (1975)
2. Total precedent and event intensity	California Total precedent: 250 mm Event intensity: 6 mm/h Los Angeles Total precedent: 255mm Event intensity: 6.35 mm/h	Campbell (1975)
3. 15 days precedent and total in an event	Hong Kong 15 days antecedent: >350mm Total event rainfall: >100 mm	Campbell (1975)
4. 5 days antecedent and 24h event	San Dimas Experimental Forest 5 days antecedent: >500 mm 24h event rainfall: >150-200 mm	Rice (1982)
5. Total antecedent from the start of the rainfall event and event intensity	Japan Antecedent (start of rain event): 100-150 mm Event intensity: >20-30 mm/h	Onodera <i>et al</i> (1978)
6. Daily antecedent and event intensity	Japan Daily antecedent: >200 mm Event intensity: 5 mm/10 min	Ishihara <i>et al.</i> (1976)
7. Daily antecedent and total event	Norway Daily antecedent Event rainfall: 1-4 hours	Sendersen <i>et al.</i> (1996)
8. 25 days antecedent and daily rainfall	Manizales, Colombia (Deep slides) 25 days antecedent: >200 mm Daily rainfall: 0-50 mm	Terlien (1997)
9. Daily rainfall and hourly rainfall	Japan Daily rainfall: >200mm Hourly rainfall: >50mm Daily rainfall: >100 mm Hourly rainfall: >20 mm	Kay and Chen (1995) Ikeya (1976)
10. Antecedent from the start of the rainfall event and hourly rainfall		Okada <i>et al.</i> , (1994, 1995)

Actual rainfall values prior to historical earthquakes were compiled and yearly, six monthly and monthly values range from 0 to 6280 mm, 0 to 3300 mm and 0 to 1440 mm, respectively. Excess or deficit rainfall related to these cases range from -720 mm to 2500 mm, -460 mm to 835 mm and 185 mm to 300 mm for yearly, six months and monthly periods, respectively. This confirms that earthquake-induced landslides may be triggered in very dry conditions, depending on other factors such as susceptibility due to geological controls or seismic shaking intensity. Relationships between these factors were combined in developing models for hazard assessment; results are presented in Chapters 7 and 8.

CHAPTER 4

LANDSLIDE HAZARD ASSESSMENT

4.1 Concepts

Landslide hazard is the potential for harm caused by existing and future landslides. It is dynamic because landslides vary in extent, magnitude and frequency over time due to environmental changes, while the interaction with human activities continues to increase due to population growth and economical development. This potential for harm can be reduced through landslide hazard management, in which engineering and planning measures are employed to limit the scale and extent of slope instability and constrain or control areas of moving ground. However, such approaches can only be implemented in a cost-effective manner if the location, nature, scale and extent of the hazard has been determined.

Landslide hazard assessment refers to the procedures and techniques used to identify and delimit the potential for cost-inducing landslide phenomena in terms of magnitude, frequency and type. The hazard due to a natural event must be related to the risk associated with the hazard. Risk is defined as a combination of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of occurrence. This can be expressed as:

$$R = H \cdot E \cdot V \quad (4.1)$$

where R is the specific risk, H is the hazard, E are the elements at risk, and V is the vulnerability of these elements.

In order to avoid conflicts of nomenclature and to establish a set of terms for use in disaster studies, the IUGS Working Group on Landslides (1997) proposed the following terms and definitions, and they will be used in the present analysis.

Risk: A measure of the probability and severity of an adverse effect on health, property or the environment.

Danger: The natural phenomenon (the landslide), geometrically and mechanically characterised.

Hazard: A condition with the potential for causing an undesirable consequence. Descriptions of landslide hazard, particularly for zoning purposes, should include the characteristics of the landslides, which may include the volumes or areas of the landslides and the probability of their occurrence. There may also be value in describing the velocities and differential velocities of the landslide. Alternatively, the hazard is the probability that a particular landslide occurs within a given time.

Elements at risk: Meaning the population, buildings and engineering works, economic activities, public services, utilities and infrastructure in the area potentially affected by landslides.

Probability (P): The likelihood of a specific outcome, measured by the ratio of specific number of outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Vulnerability (V): The degree of loss to a given element or set of elements within the area affected by landslides. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given that the person is affected by the landslide.

Risk analysis: The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards.

Risk assessment: The process of risk analysis

Acceptable risk: A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management.

Tolerable risk: A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

Individual risk: The risk of fatality and/or injury to any identifiable (named) individual who lives within the zone exposed to the landslide, or who follows a particular pattern of life that might subject him or her to consequences of the landslide.

Societal risk: The risk of multiple injuries or deaths to society as a whole: one where society would have to carry the burden of a landslide accident causing a number of deaths, injuries, financial, environmental, and other losses.

All forms of hazard assessment are concerned with predicting the potential for threatening events in natural or physical environmental systems, and have as their objective the reduction of loss from a human perspective. Landsliding has for long been one of the potentially most predictable of geologic hazards; properly conducted landslide hazard assessment are extremely cost effective and could result in dramatic reductions in landslide losses. This statement is based on three basic assumptions (Varnes, 1984):

1. that uniformitarian principles are applicable, in that the conditions that led to slope instability in the past and the present will apply equally well in the future. Thus, the estimation of future instability can be based on the assessment of conditions that led to slope failure in the past;
2. that the main conditions that cause landsliding are known, understood and can be identified, and

3. where the causes of landsliding can be identified, it is usually possible to estimate the relative significance of individual factors. This facilitates assessment of degree of hazard by examining the number of failure-inducing mechanisms present in any area.

Landslide prediction is a spatially extensive exercise concerned with establishing different levels of threat (hazard zonation), by using the spatial distribution and magnitude-frequency characteristics of identifiable influencing factors, together with past slope failures to establish probabilities of slope instability.

The level of hazard for slopes can be defined as the product of the magnitude of the triggering factor and the susceptibility of the slope materials to sliding. This approach can be written as:

$$H_s = T \cdot S \quad (4.2)$$

where H_s is the landslide hazard, T is a function representing the triggering factor, and S is a function representing the slope susceptibility.

The precise nature of the interaction between the hazard and the people or structures at risk controls the hazard analysis study. The larger and more geologically varied the area, and the more diverse its condition of human occupation and activity, the more complex the investigation must be, in purpose, content and scale. For this reason most, if not all, hazard zonation should proceed in phases. The main elements during a landslide hazard study are as follows:

1. Problem identification

- Compilation and analysis of the available information (literature and maps)

- Preliminary identification of general risk

- Identification of short-term preventive and control measurements

2. Mass susceptibility assessment

- Topographic maps

Geology and hydrology

Geomorphology

Vegetation, erosion and land use

Geotechnical characterisation of materials

Stability assessment

Susceptibility assessment

Zonation by susceptibility

3. Triggering elements assessment

Climate conditions and hydrology

Dynamic loads especially seismic events

Natural erosion

Anthropogenic intervention

4. Landslide risk management

Hazard assessment

Vulnerability assessment

Risk assessment

5. Control and prevention measures

Alarm systems

Measures to reduce the exposure

Measures to reduce the hazard

Measures to increase resistance

Subsequent studies definition

6. Hazard assessment cost against socio-economic benefit analysis

The products of a hazard assessment usually consist of information in some sort of spatial format, be it maps, point localities, data banks of attributes of points or cells of the ground. Many landslide

hazard zonation schemes employ the concept of superposing and integrating spatial information or maps, showing individually the factors thought to be important in assessing slope stability. Commonly these include maps showing lithology and geomorphologic characteristics, groups of geologic units having common lithology, structures, geotechnical properties or behaviour, hydrologic conditions, rainfall and climate, and sometimes seismic activity and expected seismic response.

It is helpful to assess the role of factors that contribute to slope instability by applying a numerical rating system. The most widely used numerical method for assessing slope stability is a conventional engineering analysis of a specific site using determined values of soil properties, accurate profiles, and pertinent hydrologic information to calculate the factor of safety. Slopes are then divided into groups according to the calculated factor of safety. Due to the uncertainties associated with each factor included in the analysis, statistical analysis can be applied to show the variability of the factor of safety. For seismically-induced landslides similar analyses have been applied by pseudo-static analysis that includes seismic effects as an additional inertial force for calculating the factor of safety. Permanent displacements, obtained by the sliding block model proposed by Newmark (1965), have also been used to assess slope stability under dynamic loads. In this case stability is assessed by comparing calculated with admissible displacement values.

Statistical methods have also been applied in the analysis of large bodies of quantitative and qualitative information obtained over areas rather than at a specific site, concerning geologic, topographic, geomorphic, hydrologic, vegetative and other attributes that may contribute to slope instability. Computer processing, and particularly GIS, have made feasible the exploration of interrelations among many factors and the evaluation of their relative significance. The most significant factors can be isolated by the method of discriminant functions, which leads to an expression involving a small number of attributes that can be measured or given a numerical designation, and whose combination in a derived formula yields a specific number. The magnitude of that number, as correlated with actual failed and non-failed slopes, then becomes a predictor of whether a newly studied slope is likely to fail, with an estimated probability of being correct. Computer-assisted statistical analysis of factors combined with automated plotting of a grid of cells to build up a map results in a very powerful and flexible method of hazard zonation.

4.2 Seismic hazard analysis as part of earthquake-induced landslides hazard assessment

As it was discussed in previous chapters, soil and slope behaviour, and consequently slope stability under seismic loading, depend on the intensity of the seismic shaking. Furthermore, as stated in the last section, a hazard analysis includes the assessment of triggering factors, in this particular case the earthquake shaking intensity. Thus, for stability analyses of particular slopes and regional landslide hazard zonation, it is necessary to carry out the evaluation of the earthquake ground motions that are likely to be induced by future earthquakes at specific localities. To evaluate these ground motions the following key elements need to be determined (Idriss, 1985): a) the seismic sources on which future earthquakes are likely to occur, b) the size of the possible earthquakes and the frequency with which an earthquake is likely to occur on each source, and c) the relationship of each source with respect to the site in terms of distance and orientation.

Given these elements, the characteristics of the ground motion at the site can be evaluated; such as an evaluation is referred as a *seismic hazard analysis*. Elements of hazard evaluation are carried out in varying degrees of detail depending on the nature of the hazard and the elements at risk.

4.2.1 Seismic sources

As stated in Chapter 2, earthquakes are the result of sudden release of elastic strain energy by brittle failure on faults. The three basic types of failures are normal, thrust and strike-slip, and involve extension, shortening and lateral movement of geologic deposits around the fault, respectively. During very short periods of the geological time scale of engineering interest, the sense of motion on a particular fault is observed to be the same in successive earthquakes. Repeated movement along the same fault will therefore create a topography in active regions characteristic of the type of faulting. This allows the geomorphology associated with the sense of motion to be used as an important guide in identified active faults (Allen, 1975; Idriss, 1985; Ambraseys, 1990; Keller and Pinter, 1996).

A major earthquake typically leaves a distinct geologic fault offset which can be preserved for thousand and possible hundreds of thousands of years. Surface ruptures are neither continuous nor do they follow precisely the surface outcrop of pre-existing faults. They rather follow planes of weakness with a broad shear zone, from a few meters to a few kilometres wide, shifting laterally from a shear plane in one part of the zone to another elsewhere, in most cases with well developed large scale en echelon patterns. Tension features and grabens are often connected with en echelon shear (Ambraseys, 1990). Figure 4.1 shows schematically the different geologic and geomorphologic features related to different fault types.

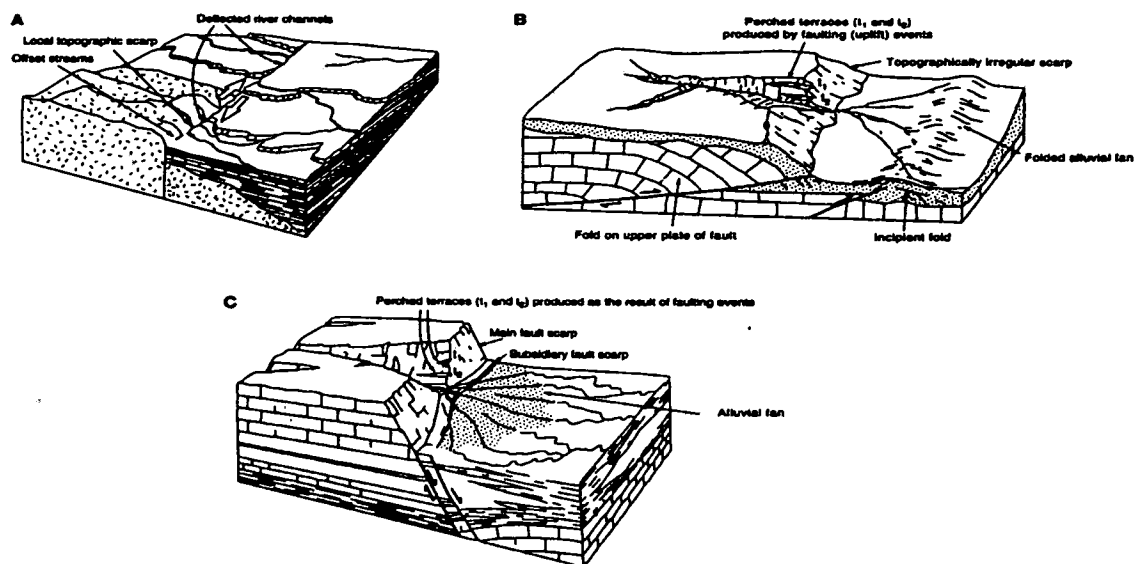


Figure 4.1 Diagram showing types of topographic expression possible with different types of faults. a) Strike slip, b) thrust fault and c) normal fault. (Keller and Pinter, 1996).

The preserved geological features along faults can be investigated by geologic and geophysical studies that may include: review of available literature, especially with regard to structural and tectonic history; interpretation of various types of remote imagery to identify regional structures; reconnaissance of the geology and geomorphology of the region; and the use of trenching, borehole, age-dating and geophysical exploration techniques.

Geologist and seismologist have studied the detailed characteristics of faults and a degree of activity is assigned to each based on recency of fault displacements. Most geologists would

consider a fault to be *active* if it has moved during the past 10,000 years (Holocene Epoch). The Quaternary period (the past 1.65 M y) is the most recent period of geologic time, and most of the present landscape has been produced during this time. Any fault that has moved during the Quaternary may be classified as *potentially active*. Faults that have not moved during the past 1.65 M y are generally classified as *inactive* (Keller and Pinter, 1996). For critical facilities such as nuclear plants, an active fault is considered to have shown evidence of multiple displacement in the past 500,000 years, or a single evidence of displacement in the past 35,000 years (Idriss, 1985). These figures vary amongst different regulatory bodies for nuclear power stations; the figure 35,000 years was fixed in the 1980's only because it was, at that time, the limit of carbon dating techniques. Assessment of fault activity may be based on geological, historical and seismological data as presented in Table 4.1.

Table 4.1 Data for assessing fault activity (Adapted from Ambraseys, 1990).

	Historical evidence	Geological evidence	Seismological evidence
Active Faults	Historical or recent records of surface faulting with associated strong earthquakes. Fault creep or geodetic indications of movement	Young deposits displaced or cut by faulting. Fresh geomorphic features characteristic of active fault zones along fault trace. Physical ground water barriers in young deposits. Offset streams	Earthquake epicentres associated with individual faults with a high degree of confidence, including historical earthquakes.
Potentially active faults	No reliable report of historical surface faulting. Faults are not known to displace or cut the most recent alluvial deposits	Geomorphological features characteristic of fault zones are subdued, eroded and discontinuous. Water barriers may be present in older materials. Fault may be in geologic setting in which its geometric relationship to nearby active or potentially active faults suggest similar levels of activity.	There may be an alignment of some earthquakes epicentres along the fault trace, but locations are assigned with a low degree of confidence.
Faults of uncertain activity	Available data insufficient to provide criteria	Available data insufficient to provide criteria	Available data insufficient to provide criteria
Inactive faults	No historical activity based on thorough study of local sources of information.	Features characteristic of active fault zones are not present, and geological evidence is available to indicate that the fault has not moved in the recent past.	Lack of known epicentres on the fault trace. However, this is not sufficient evidence.

Recently other parameters have been identified to be associated to relative fault activity such as: rate of strain release or fault slip; amount of fault displacement in each event; length and area of fault rupture; earthquake size; and earthquake recurrence interval. Idriss (1985) stated that these

parameters are a function of the tectonic environment, the fault type and geometry, the rate of strain accumulation, the structural character and physical properties of the Earth's crust, and the complexity and physical properties of the fault zone.

4.2.2 Earthquake size and recurrence

The earliest measure of earthquake size was based on the maximum intensity and areal extent of perceptible ground shaking. Instrumental recordings led to the development of magnitude scales, which represent a measure of the energy released by the earthquake, independent of the place of observation. These parameters were discussed in Chapter 2.

For seismic hazard analysis it is important to determine the maximum earthquake magnitude that can occur from a specific source. Several methods have been proposed to assess this magnitude (Idriss, 1985; Ambraseys, 1990; Keller and Pinter, 1996; Kramer, 1996). These methods are based on empirical correlations between magnitude and fault parameters such as fault rupture length and surface fault displacement measured following earthquakes; and fault length and width estimated from studies of aftershock sequence. Some of those relations were presented by Idriss (1985). Figure 4.2 shows some relationships proposed by Wells and Coppersmith (1994). Magnitude can also be calculated from seismic moment through a relationship between magnitude and slip rate. As pointed by Idriss (1985), use of a number of magnitude estimation methods can result in more reliable estimates of maximum magnitude for each source.

Faults having different degrees of activity differ significantly in the average recurrence intervals of significant earthquakes. Recurrence intervals can be calculated directly from slip rates or by statistical analysis of historical and instrumental seismicity.

The most usual way to represent the fault activity is in the form of a recurrence relation which relates earthquake magnitude to frequency of occurrence. Gutenberg and Richter (1954) found that this relation can be expressed by:

$$\log(N_c) = a - bM \quad (4.3)$$

where M is the earthquake magnitude and N_c is the number of earthquakes of magnitude greater or equal to M per unit time. In general, the numbers are normalised to a year and unit area for area source or unit length for linear source. a therefore is a measure of the activity, when normalised. b is a measure of the brittleness of the region. If the crust is highly faulted so that there exists many small faults and few large faults, then b will be large and there will be a tendency for many small earthquakes compared to large earthquakes. Table 4.2 gives values of a and b for shallow earthquake in different regions in the world.

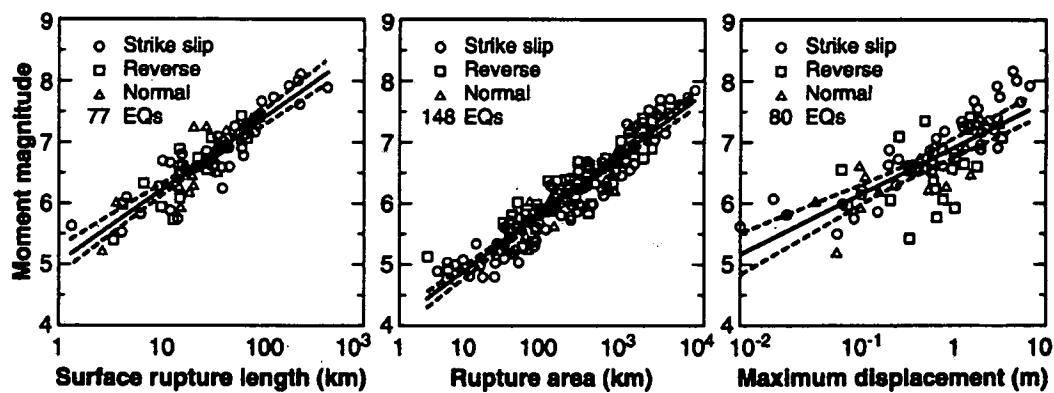


Figure 4.2 Relationships between magnitude and fault characteristics (Wells and Coppersmith, 1994).

For engineering applications, the recurrence is limited to a range of magnitudes between M_0 and M_U . M_0 is the smallest magnitude of concern in the specific application. M_U is the largest magnitude the fault is considered capable of producing, which can be estimated from the study of past earthquakes and tectonic activity of the fault. Different recurrence relations have been proposed by different authors, which have been shown to be dependent on the particular regional tectonic characteristics; Purcaru (1975) summarises these relations, and classified them based on similarities in shape.

Table 4.2 *a* and *b* values for different regions in the world. Normalised for areas of 2° x 2°. (After Dowrick, 1987).

Region	<i>a</i>	<i>b</i>
Japan	6.86	1.22
New Guinea	7.83	1.35
Western USA	5.94	1.14
Eastern USA	5.79	1.38
Central America	7.36	1.45
Colombia-Perú	5.60	1.11
Mediterranean	5.45	1.10
Iran-Turkmenia	6.02	1.18
East Africa	3.80	0.87

To calculate the probabilities of various hazards occurring in a given time period, the distribution of earthquake occurrence with respect to time must be considered. Earthquakes have long been assumed to occur randomly with time. This assumption of random occurrence allows the use of simple probabilistic models, but is inconsistent with the implications of elastic rebound theory. The temporal occurrence of earthquakes is most commonly described by a Poisson model. This model provides a simple framework for evaluating probabilities of events that follow a Poisson process, one that yields values of a random variable describing the number of occurrence of a particular event during a given time interval.

For a Poisson process, the probability of a random variable *N*, representing the number of occurrences of a particular event during a given time interval, *t*, is given by:

$$p = \frac{e^{-v} (v)^n}{n!} \quad (4.4)$$

where *v* is the average number of earthquakes > *M* per unit time.

The probability of occurrence of at least one event in a period of time L , is given by:

$$p = 1 - e^{-\lambda L} \quad (4.5)$$

The Poisson model is useful for practical seismic risk analysis except when the seismic hazard is dominated by a single source for which the time interval since the previous significant event is greater than the average interval. Other models have been proposed that incorporate the pattern of earthquake occurrence and relate it to elastic rebound process. This implies the necessity of additional information on the fault characteristics that are difficult to obtain which makes these models of limited applicability in most cases (Kramer, 1996).

The probability of occurrence may be used in defining design magnitudes based on known useful life. These magnitudes can be used in conjunction with the appropriate attenuation relationship to derive design ground-motion parameters.

4.2.3 Deterministic Seismic Hazard Analysis

A deterministic seismic hazard analysis involves the development of a particular seismic scenario upon which a ground motion hazard evaluation is based. The scenario consists of the postulated occurrence of an earthquake of a specific size occurring at a specified location. A typical deterministic hazard analysis can be described by the following steps (Reiter, 1990):

1. Identification and characterisation of all earthquake sources capable of producing significant ground motion at the site.
2. Selection of a source-to-site distance parameter for each source. In most cases, the shortest distance between the source and the site of interest is selected. This parameter depends on the relationship to be used to determine the ground-motion parameter selected for design.
3. Selection of the controlling earthquake; this is the earthquake that is expected to produce the strongest level of the design parameter. The selection is made by comparing the levels

of shaking produced by earthquakes assumed to occur at the specified distance parameter.

The controlling earthquake is described in terms of its size and distance to the site.

4. The hazard is defined in terms of the design parameter produced by the controlling earthquake at the site.

4.2.4 Probabilistic Seismic Hazard Analysis

This method allows for the uncertainties in the size, location, and rate of occurrence of earthquakes and in the variation of ground-motion characteristics to be included. A probabilistic seismic hazard analysis provides a framework in which these uncertainties can be identified, quantified, and combined in a rational manner to provide a more complete picture of the seismic hazard.

The probabilistic seismic hazard analysis can be described by the following steps (Reiter, 1990):

1. Identification and characterisation of earthquake sources. In this case distribution of the potential rupture locations within the source must also be characterised. In most cases, uniform probability distributions are assigned to each source zone, implying that earthquakes are equally likely to occur at any point within the source zone. These distributions are then combined with the source geometry to obtain the corresponding probability distribution of source-to-site distance.
2. Seismicity or temporal distribution of earthquake recurrence is characterised. A recurrence relationship, which specifies the average rate at which an earthquake of some size will be exceeded, is used to characterise the seismicity of the source zone.
3. The ground motion produced at the site by earthquakes of any possible size occurring at any possible location within each source zone is determined from attenuation relationships. Uncertainty in prediction of the design parameter can also be included in this step.
4. Uncertainties in earthquake location, earthquake size, and ground-motion parameter prediction are combined to obtain the probability that the ground motion parameter will be exceeded during a particular period of time.

4.3 Landslide hazard zonation

Landslide hazard assessment refers to techniques used to identify and quantify the different hazard characteristics: type, location, magnitude and recurrence. The basis for these techniques to be capable of identifying those characteristics, is an inventory of landslides and their spatial and temporal distribution. The spatial distribution of landslide hazard is commonly referred as *landslide hazard zonation*, which involves the mapping of inventoried landslides and factors that influence the landsliding process.

Relationships between landslide occurrences and various influencing factors can be obtained by direct observations during a survey of as many existing landslide sites as possible. The opposite to this method is the mapping of large number of parameters considered to be important in landslide occurrences and subsequently analysing all these possible contributing factors by statistical analysis. Results of this kind of analysis give hazard assessment in terms of values or absolute values. Relative values are usually referred to analyses based only on landslide occurrence cases, whereas absolute values can be obtained from analysis that include positive and negative evidence of occurrences.

The working scale for landslide hazard zonation is determined by the requirements of the user for whom the survey is executed. However, Rockaway (1976) stated that selection of map scale must depend on some extent on the nature and variability of the geologic environment. He also suggests that the selection must consider the balance between increase in details and reduction in coverage, and the reduction in benefit/cost as the scale increases. The scales presented in Table 4.3 were differentiated by the International Association of Engineering Geology (1976).

Landslide hazard zonation requires the knowledge of triggering and susceptibility factors and their interrelation with landslide occurrence. The input data needed to assess the influence of these factors depends on the working scale. However, the availability of data with respect to different factors in turn can control the scale that can be applied. The input data necessary for landslide hazard zonation may be summarised as shown in Table 4.4.

Table 4.3 Different scales for landslide hazard zonation (Based on IAEG, 1976)

Scale	Range	Applicability	Coverage
National Scale	<1:1,000,000	General inventory of problem areas for an entire country that can be used to inform national policy makers and the general public	Thousands of km ²
Regional Scale	1:100,000 to 1:500,000	Used by planners in the early phases of regional development or for engineers evaluating possible constraints due to instability in the development of large engineering projects and regional development plans.	≅ 1000 km ²
Medium Scale	1:25,000 to 1:50,000	Determination of hazard zones in areas affected by large engineering structures, roads and urbanisations.	Few hundreds of km ²
Large Scale	1:5,000 to 1:15,000	Site investigation before the design phase of engineering projects. This scale allows evaluation of the variability of factor of safety as a function of variable slope conditions or under the influence of different triggering factors.	Up to several tens of km ²

Various methodologies have been used for landslide hazard zonation, which were reviewed in detail by Hansen (1984) and Varnes (1984). The most straightforward approach to landslide hazard zonation is the *landslide inventory*, based on any or all of the following: aerial photo interpretation, ground survey, and a database of historical occurrences of landslides in an area. The final product gives the spatial distribution of mass movements, which may be represented on a map either as affected areas to scale or as point symbols. Such mass movement inventory maps are the basis for most other landslide hazard zonation. This method provides information only for the period shortly preceding the date of data acquisition, and does not provide insight into temporal changes in movement distribution. Landslide distribution can also be shown in the form of density maps, which shows density values by means of lines that enclosed areas of similar landslide density called *landslide isopleths* (Wright *et al*, 1974).

The *heuristic methods* are based on the expert opinion of the geomorphologist to classify the hazard. These methods combine the mapping of mass movements and their geomorphologic setting as the main input factors for hazard determination. Two types of this kind of method can be applied: geomorphic analysis and qualitative map combination. *Geomorphic analysis* is based on the determination of the hazard from direct observation in the field, hence it is based on personal experience and the use of reasoning by analogy. In the *qualitative map combination*, the Earth scientist uses the expert knowledge of an individual to assign weighting values to a series of parameter maps. The terrain conditions are summed according to these weights, leading to hazard values that can be grouped into hazard classes. The problem with this method is in determining the

exact weighting of various parameter maps. Often, insufficient field knowledge of the important factors prevents the proper establishment of the factor weights, leading to unacceptable generalisations (Soeters and van Westen, 1996).

Table 4.4 Input data required for a landslide hazard zonation

Input Data	Attributes
Geomorphology	
1. Terrain homogeneous units	Terrain geomorphological units
2. Geomorphological sub-units	Geomorphological description
3. Recent Landslides	Type, activity, dimensions, etc.
4. Old landslides	Type, activity, dimensions, etc.
Topography	
5. Altitude	Altitude classes
6. Slope angle	Slope angle classes
7. Slope direction	Slope direction classes
8. Slope length	Slope length classes
9. Plan Shape	Plan shape classes
10. Profile Shape	Profile shape classes
Geology	
12. Lithologies	Lithology, strength, weathering, etc.
13. Materials sequence	Deposit profile with materials classification and description
14. Geological structures	Discontinuities type and characteristics
Seismicity	
15. Tectonics	Plate boundaries and slip rates
16. Faults	Type, activity, characteristics
17. Seismicity	Earthquakes recurrence and characteristics
18. Strong ground-motions	Strong-motion records, characteristics, seismic parameters
Hydrology	
19. Drainage	Type, order, length
20. Drainage basin	Order, size, area, etc
21. Rainfall	Type, Rainfall records, magnitude-duration-frequency curves
22. Temperature	Temperature records
23. Evapotranspiration	Evapotranspiration records
24. Water table	Water table records
Land-use	
25. Recent infrastructure	Road types, railway lines, urban extension, etc
26. Older infrastructure	Road types, railway lines, urban extension, etc
27. Recent land-use	Land-use classes and distribution
28. Older land-use	Land-use classes and distribution

Statistically-based landslide hazard analysis combines, statistically, factors that have led to landslides. Two different statistical approaches have been used in this kind of method: bivariate and

multivariate analysis. In *bivariate statistical analysis*, each contributing factor is combined with the distribution of landslides, and weighting values based on landslides density are calculated for each parameter class. In *multivariate statistical analysis*, all relevant factors are sampled on a large-grid basis or in morphometric units. For each of the sampling units, the presence or absence of landslides is also determined. The resulting matrix is analysed by using multiple regression or discriminant analysis.

Deterministic approaches are based on physical models such as slope stability analysis. Applicability of these methods is restricted to areas with fairly homogeneous geomorphic and geologic conditions, and when landslide types are simple. A summary and classification of some available landslide hazard zonation methods are presented in Annex C.

4.4 Earthquake-induced landslides hazard assessment

Slope stability basically depends on two factors: an external driving force and the resistance of the material to movement. The external driving force includes gravitational and seismic forces, while the material resistance is governed by geological and geotechnical conditions. Standard approaches to slope stability require detailed information, but over a wide area existing geological and topographical data are generally insufficient in quality, and *in situ* investigations are too expensive to conduct to enable these approaches to be used. The Technical Committee for Earthquake Geotechnical Engineering (TC4) of the International Society of Soil Mechanics and Foundation Engineering stated that for each kind of geotechnical hazard, three grades of approach to zonation can be obtained, depending on the quality and quantity of the available data (ISSMFE, 1993).

The first level of zonation (Grade 1: General zonation) is based on compilation of existing information available from historic documents, published reports and other available databases. This is the crudest and cheapest approach, used for covering a wide region such a country, state, province or prefecture, or local areas. The quality of the Grade-1 zonation map may be improved considerably, at moderate cost, by making use of additional data sources. For example, aerial photographs can help to better define fault structures and geologic conditions. In some cases, older

photographs may be more helpful in understanding the structure of local geologic units if they pre-date urban development. When this kind of information is available, a Grade-2 zonation (Detailed zonation) may be developed. When a very high and very detailed level of zonation is required, for example using site investigation data for a specific site, it is possible apply a Grade-3 zonation (Rigorous zonation). Data used for these three levels of zonation for ground motions, slope stability and liquefaction are compiled in Table 4.5. Some of the different zonation methods for earthquake-induced landslides, which have been proposed, were classified into these three different levels. A brief discussion of these methods is made in following section.

Table 4.5 Use of data for three levels of zonation (TC4, ISSMFE, 1993)

	Grade-1	Grade-2	Grade-3
Ground motions	*Historical earthquakes and existing information *Geological maps *Interviews with local residents	*Microtremor *Simplified geotechnical study	*Geotechnical investigation *Ground response analysis
Slope stability	*Historical earthquakes and existing information *Geological and geomorphological maps	*Air photos and remote sensing *Field studies *Vegetation and precipitation data	*Geotechnical investigation *Analyses
Liquefaction	*Historical earthquakes and existing information *Geological and geomorphological maps	*Air photos and remote sensing *Field studies *Interview with local residents	*Geotechnical investigation *Analyses
Scale of mapping	1:1000000/1:50000	1:10000/1:10000	1:25000/1:5000

4.4.1 Grade-1 methods

The lowest cost but most cursory level of zonation of earthquake-induced landslides is based on earthquake magnitude or seismic intensity. This level of zonation relates potential areas of slope instability to seismic parameters based on past earthquake information. Since the rate of slope failures reduces with distance from the seismic source, the exact outer boundary of slope instabilities is generally not well defined. In zoning studies, the aerial density defines the slope failure rate and it is used as parameter to define the outer boundary.

Magnitude does not directly nor indirectly incorporate the effects of local geology and soils. In addition, groundwater conditions have an important influence on slope stability, and therefore rainfall patterns in the region are also taken as a key parameter. Topographic, geologic, and hydrologic maps may also be applied at little cost to delimitate areas that are relatively safe against slope instability.

4.4.1.1 Takamura (1978)

Based on results from 37 earthquakes in Japan, Takamura (ISSMFE, 1993) obtained relationships between magnitude (scale not specified, probably M_{JMA}) and maximum epicentral distance and maximum distance from fault to outer boundary of the zone where many and few slope failures occurred. These relationships are presented in Figure 4.3. D_p and d_p refer to epicentral distance to farthest landslides within areas containing many and few landslides, respectively. D_f and d_f refer to distance from fault projection to the farthest landslide within areas containing many and few landslides, respectively.

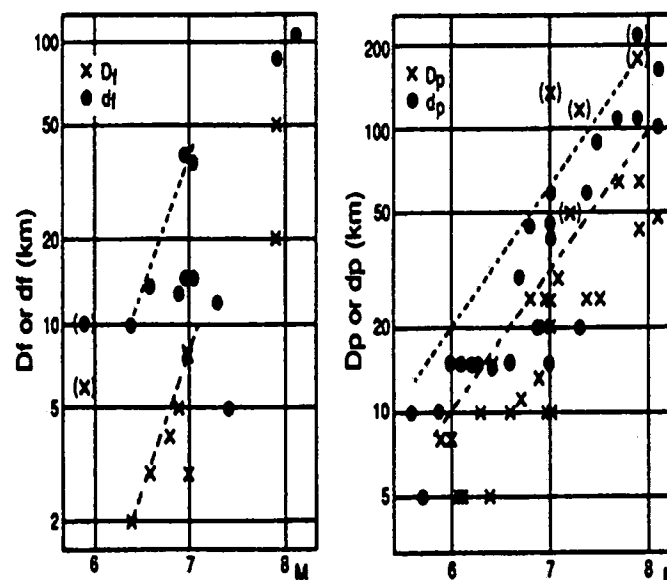


Figure 4.3 Relationships between magnitude and distance to slope failure in Japan. (From ISSMFE, 1993)

These relationships show, in general, an increase in distance to farthest landslide as magnitude increases, although, a large scatter in the data can be observed. This scatter may reflect differences

in environmental conditions such as precedent precipitation or lithology susceptibility for the different zones considered. As expected, fault distances are shorter than epicentral distances, although, for smaller magnitude these values tend to be similar, which suggests that limits such as those presented in this graph should converge for low magnitude values rather than remain parallel as presented.

An upper limit for the “few landslides” areas, as defined by Takamura, is confirmed by the present world-wide dataset as shown in Figure 4.4. This figure also shows that local or regional limits, such as these for Japan, cannot be used for other regions, and that an attempt to define a unique limit is useless due to the great scatter in data.

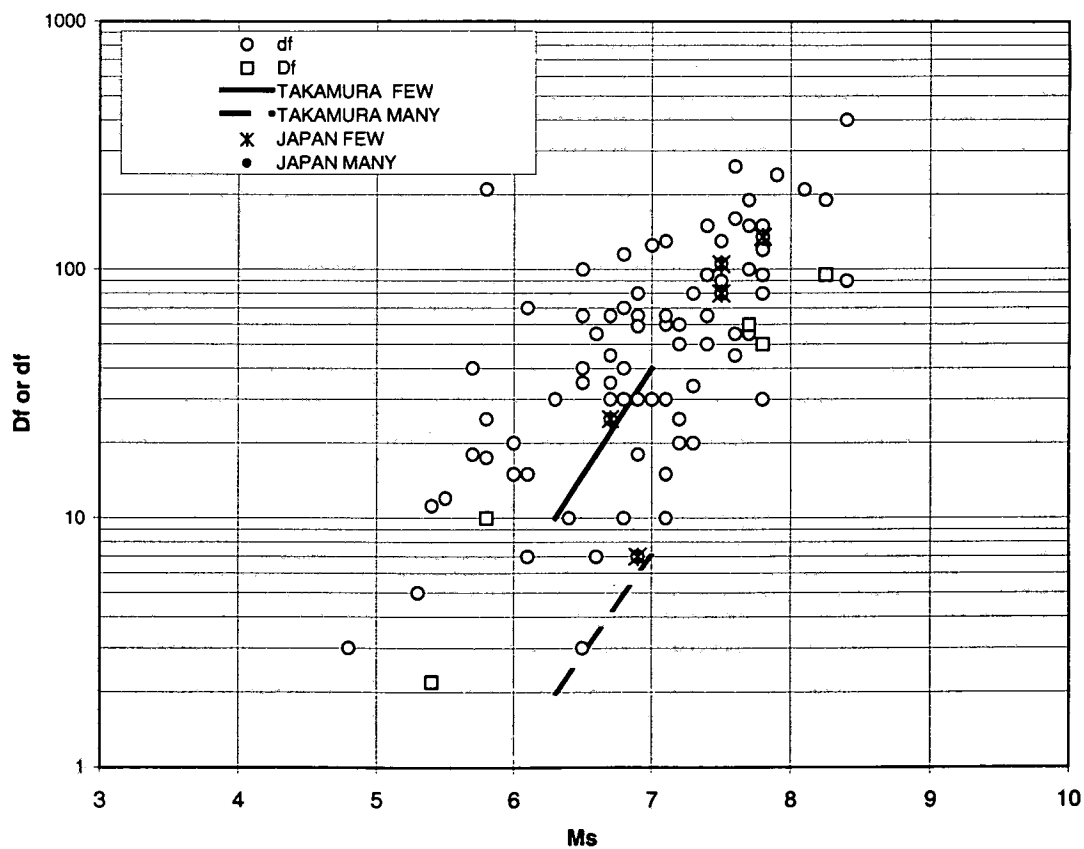


Figure 4.4 a Relationships between magnitude and maximum distance to fault projection from farthest landslide for world-wide cases included in the present dataset. Lines shown are the Takamura upper limits. Stars are Japanese cases in present dataset; no data for “many landslides” areas obtained.

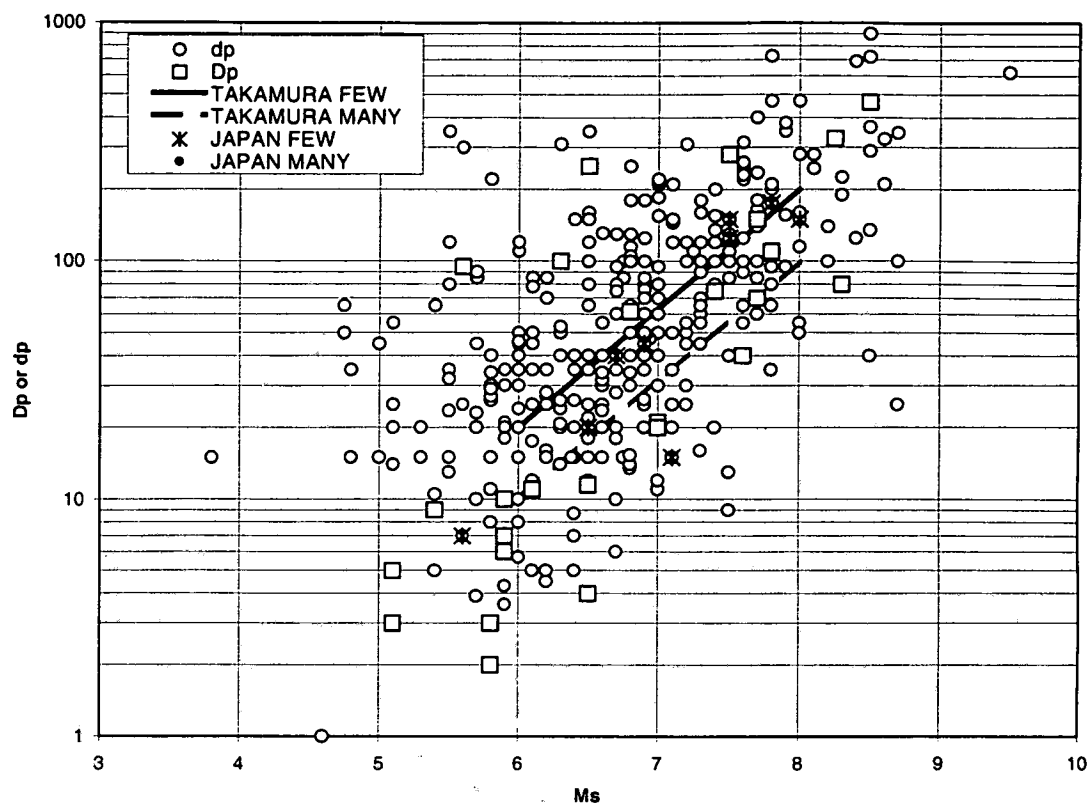


Figure 4.4 b Relationships between magnitude and maximum epicentral from farthest landslide for world-wide cases included in present dataset. Lines shown are the Takamura upper limits. Stars are Japanese cases in present dataset; no data for "many landslides" areas obtained.

4.4.1.2 Yasuda and Sugitani (1988)

These authors studied slope failures in Japan over the past 100 years, and classified 105 cases into surface and deep slides. Their relationship is presented in Figure 4.5, which suggests that seismic shaking levels needed to induced deep slides are higher than those needed for shallow slides, or that maximum distance for shallow slides is greater than for deep slides. However, although the differences seem to be very small from comparing suggested upper limits, the scatter of the data is very large and the shaking intensity necessary to trigger different slide mechanisms will depend on many other factors, among them slope susceptibility and climatic conditions.

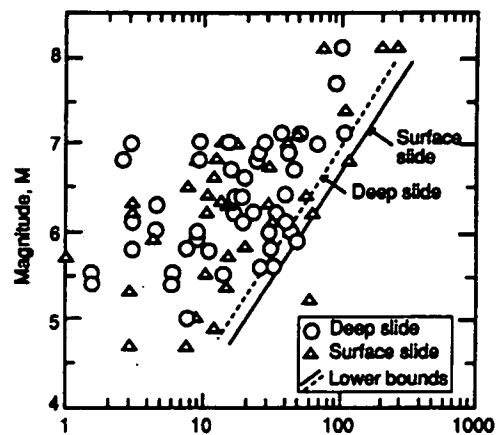


Figure 4.5 Relationship between maximum epicentral distance of slope failures and magnitude (ISSMFE, 1993)

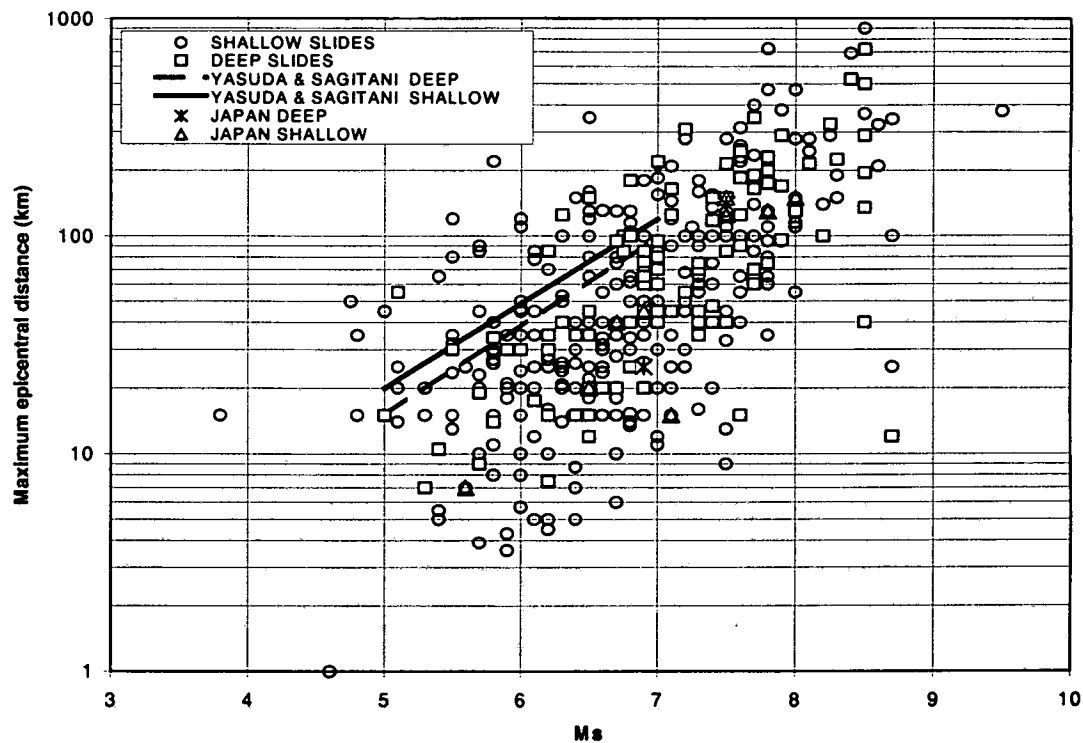


Figure 4.6 Relationship between maximum epicentral distance of slope failures and magnitude for world-wider deep and shallow slides.

Figure 4.6 confirms the Japanese upper limits and that regional or local relationships are not necessarily applicable in a generalised manner.

4.4.1.3 Keefer (1984)

Keefer (1984) presented magnitude (using different scales)-distance (epicentral and fault) relationship for three different landslide categories: coherent, disrupted slides and falls, and lateral spreads and flows. Figures 4.7 to 4.9 show present the data classified into these three categories and the upper limits defined by Keefer.

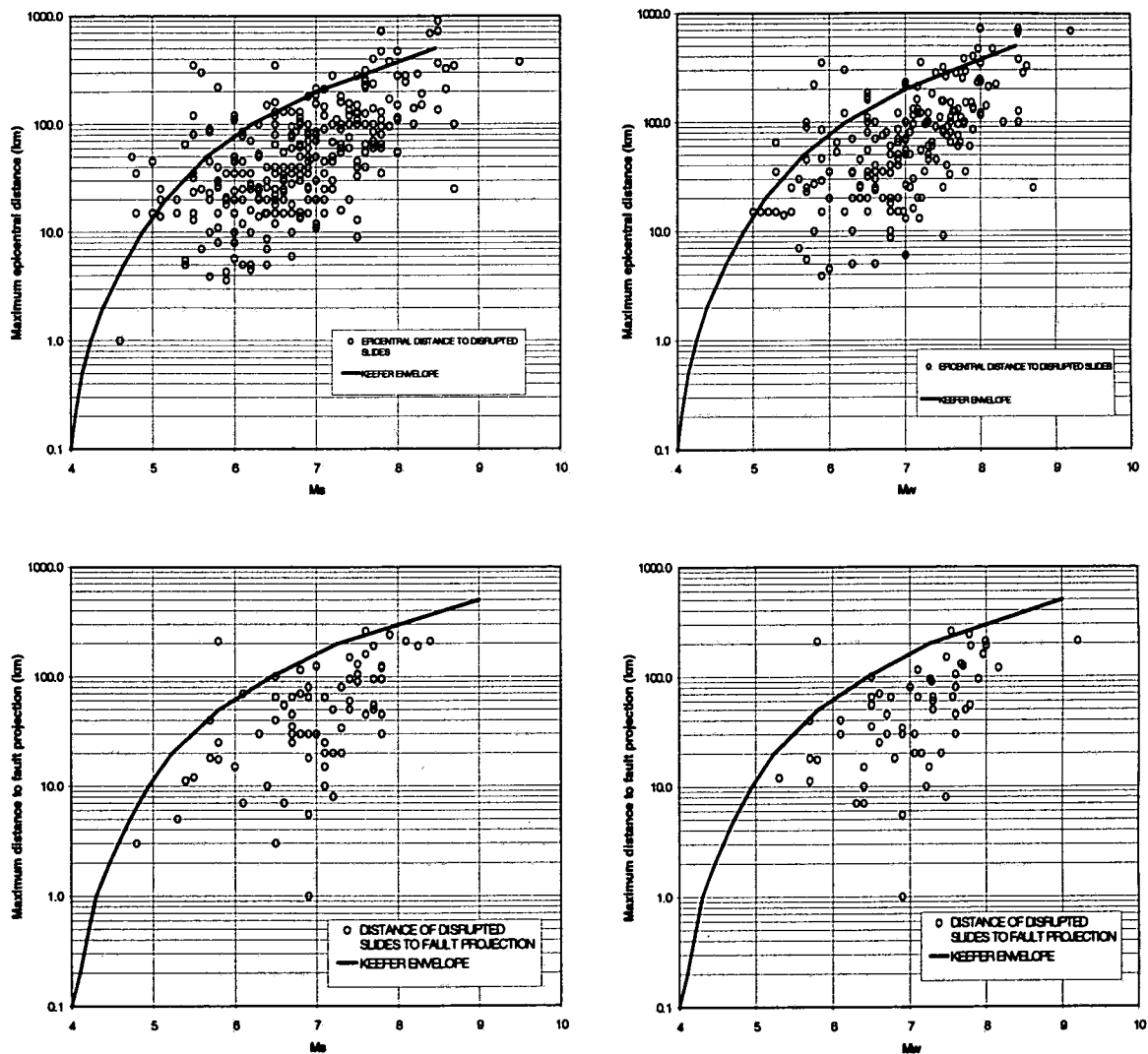


Figure 4.7 Magnitude-distance relationships for disrupted slides. Upper graphs for epicentral distance, lower for distance to fault projection. Left graphs for M_s , right for M_w . Lines are Keefer's envelopes.

As shown in Figures 4.7 to 4.9, many points of the present dataset lie well above the Keefer's upper limits for epicentral distances; this suggests that a new enveloped may be proposed, although, due to the great scatter it seems that a general boundary is not of practical use. A detailed analysis of

some of these data points was presented by Rodríguez *et al.* (1999), and it was found that these relationships are strongly influenced by factors such as antecedent climatic conditions, earthquake mechanism, aftershocks, special seismic waves travel effects such as simultaneous arrival of different phases of seismic waves, and slope susceptibility.

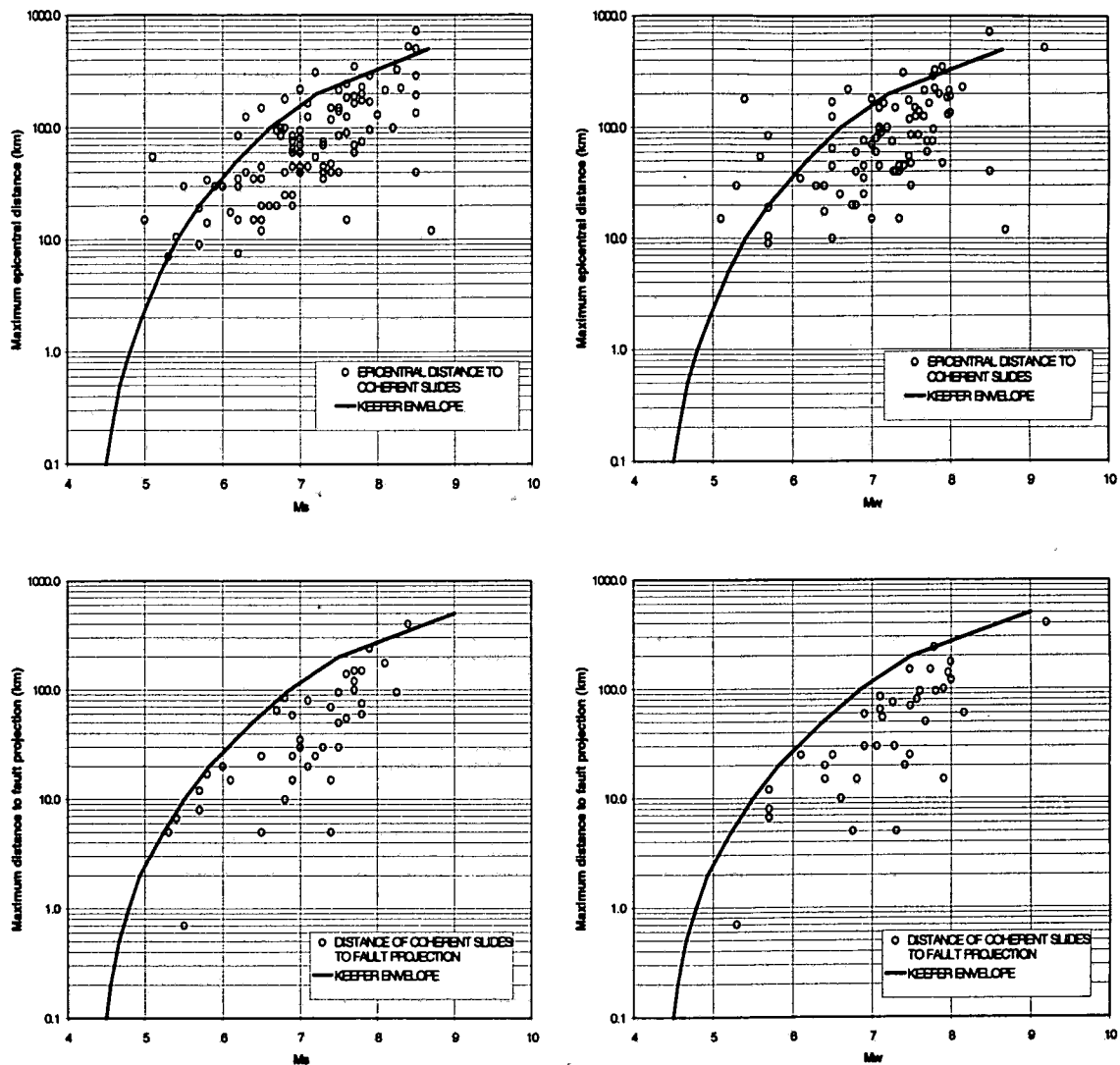


Figure 4.8 Magnitude-distance relationships for coherent slides. Upper graphs for epicentral distance, lower for distance to fault projection. Left graphs for M_s , right for M_w . Lines are Keefe's envelopes.

Based on Takamura and Yasuda & Sugitani results, it is apparent that these relationships are more consistent in a regional scale, however, Figure 4.10 shows results for Central America and it can be seen that relationships are more consistent for particular lithologies rather than for geographical regions. In this case, it is shown that El Salvador and Guatemala cases clearly define a different

relation from that for Costa Rica and Panamá. These differences may be explained not only by differences in climatic conditions, as suggested by Figure 4.11, but also by differences in geologic environments and slope failure mechanisms.

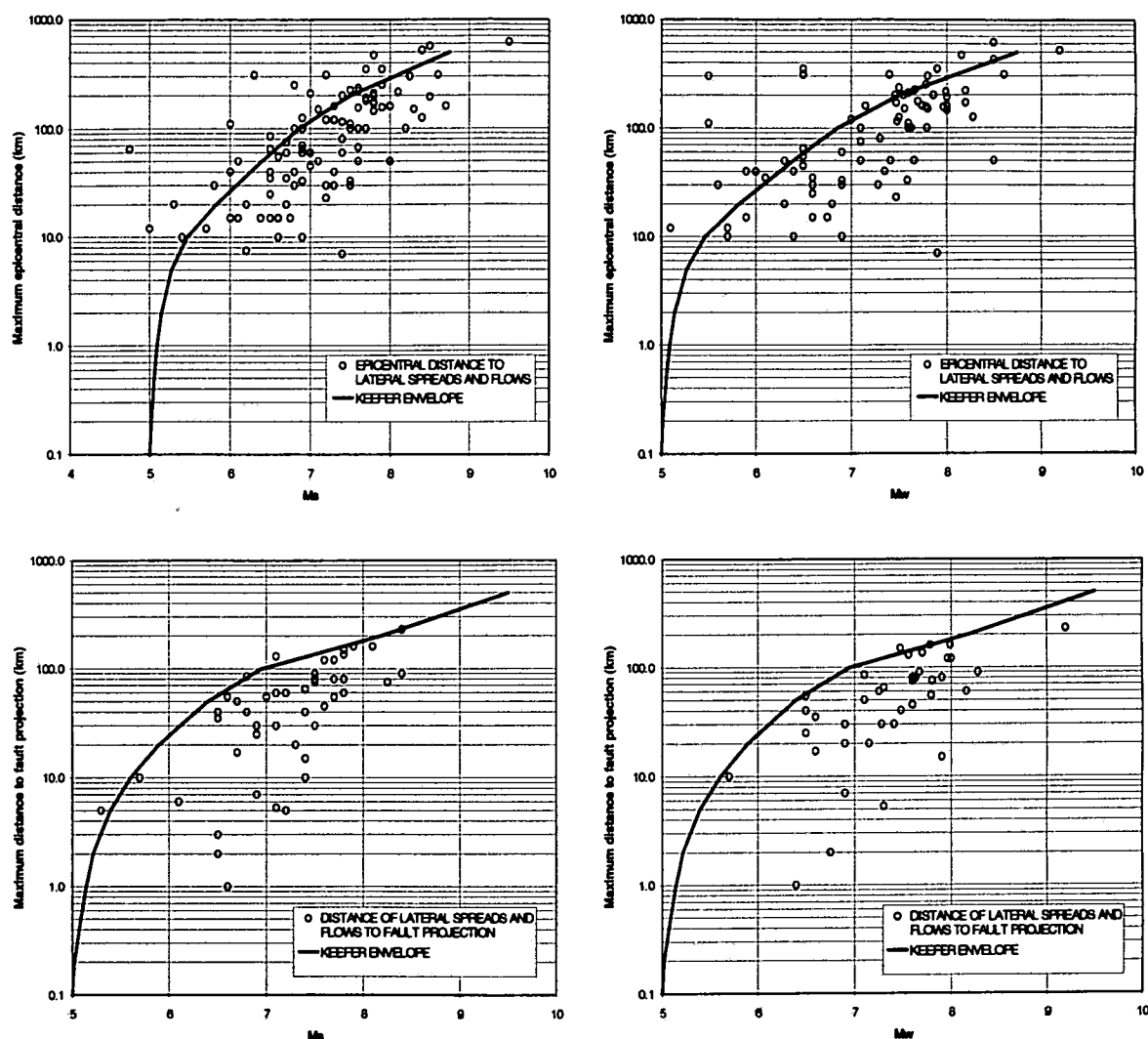


Figure 4.9 Magnitude-distance relationships for lateral spreads and flows. Upper graphs for epicentral distance, lower for distance to fault projection. Left graphs for M_s , right for M_w . Lines are Keefe's envelopes.

In El Salvador and Guatemala, the most common earthquake-induced landslides are rock falls and soil falls from volcanic deposits, whereas in Costa Rica and Panamá translational shallow slides of residual soils have been shown to be the most frequent. The magnitude of triggering factors as seismic energy for these two different mechanism in different lithologies are also different as suggested by Figure 4.10. Even more, for a given terrain unit, the seismic intensity needed to induce

landslides is a function of slope susceptibility including antecedent climatic conditions, which alter this susceptibility.

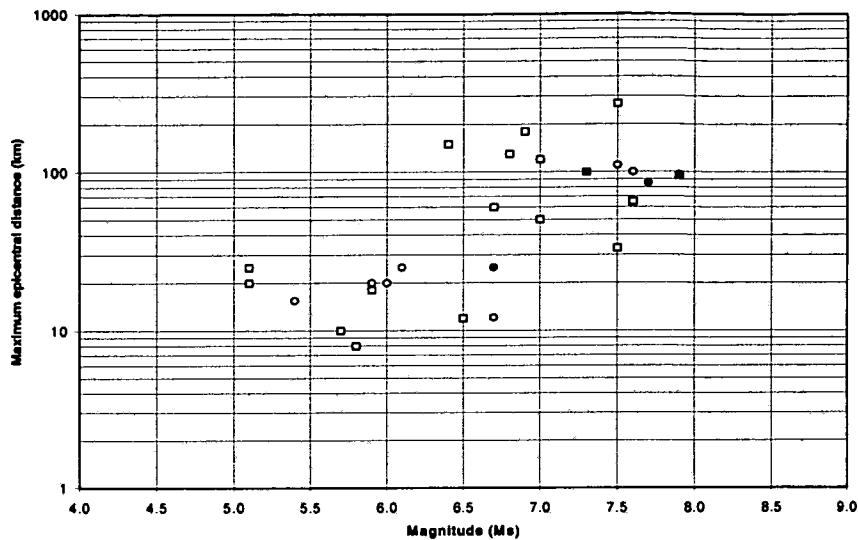


Figure 4.10 Magnitude-distance relationship for disrupted slides in Central America. Squares represent Costa Rican and Panamanian cases. Circles are Salvadorian and Guatemalan cases. Black points are subduction earthquakes, whereas white points are upper-crustal earthquakes.

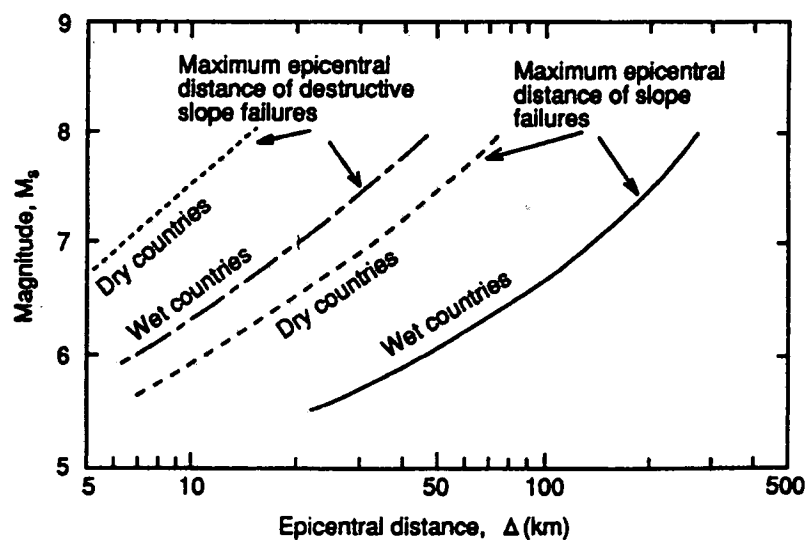


Figure 4.11 General relationship between magnitude and epicentral distance of slope failures (ISSMFE, 1993).

Perkins (1997) points out that an inherent weakness in using the maximum distance of expected landsliding as a function of earthquake magnitude is that it assumes that the same hazard exists at all locations within the area defined by this distance. Clearly, the possibility of landsliding increases with the intensity of the ground motion and therefore correlations with macroseismic intensity rather

than magnitude are likely to provide greater insight and ultimately a more flexible tool for hazard assessment. Modified Mercalli Intensity was also proposed by Keefer as parameter to relate landslides and seismic triggering as shown in Figure 4.12, which shows the minimum MM intensity that triggered disrupted slides, coherent slides and lateral spreads in each of the earthquakes for some cases in the new database and in Keefer's database. A slope that is imminently unstable could be caused to fail by very weak shaking, therefore, the landslides associated with intensities of IV and V probably correspond to highly susceptible slopes. Of greater significance is the modal value on each plot, giving an indication of the level of shaking that typically is required to induce different types of landslides. The shapes of the histograms for the combined databases are very similar to those presented by Keefer with a modal value of VI for coherent slides and VII for lateral spreads and flows, though the new database suggests that the modal value for disrupted slides may be slightly higher at VI+.

The main conclusion that can be drawn from these observations is that the triggering of landslides during earthquakes is strongly dependent on the susceptibility of the slope and hence the implied lower bounds are of limited use. It was for these reasons that the modified version of the MSK intensity scale produced by the European Seismological Commission (1993) omits many of the former references to effects on nature, including landslides, noting that "*evidence is insufficient to establish good correlation between these effects and particular intensity grades*". The resulting European Macroseismic Scale (EMS) recommends that effects of earthquake shaking on nature should be employed with caution and in conjunction with other observations, and that intensities should never be assigned purely on the basis of such observations. In the 1998 version, the geotechnical criteria for assessment have been removed completely.

Figure 4.12 shows also the total number of individual slides in each category for some cases in the new database, plotted against the MM intensity that triggered the slide. In Figure 4.12 there is no data value below VI because all the particular slides included were in areas with MMI greater than VI. The interpretation of these plots should take into account the fact that in general the lower values of intensity will be felt over a relatively wider area. On the other hand, only few earthquakes from the new database have been shown to cause landslides in areas below an intensity of VI, suggesting that

landslides in lower intensity areas are isolated cases that may depend mainly on the antecedent slope susceptibility.

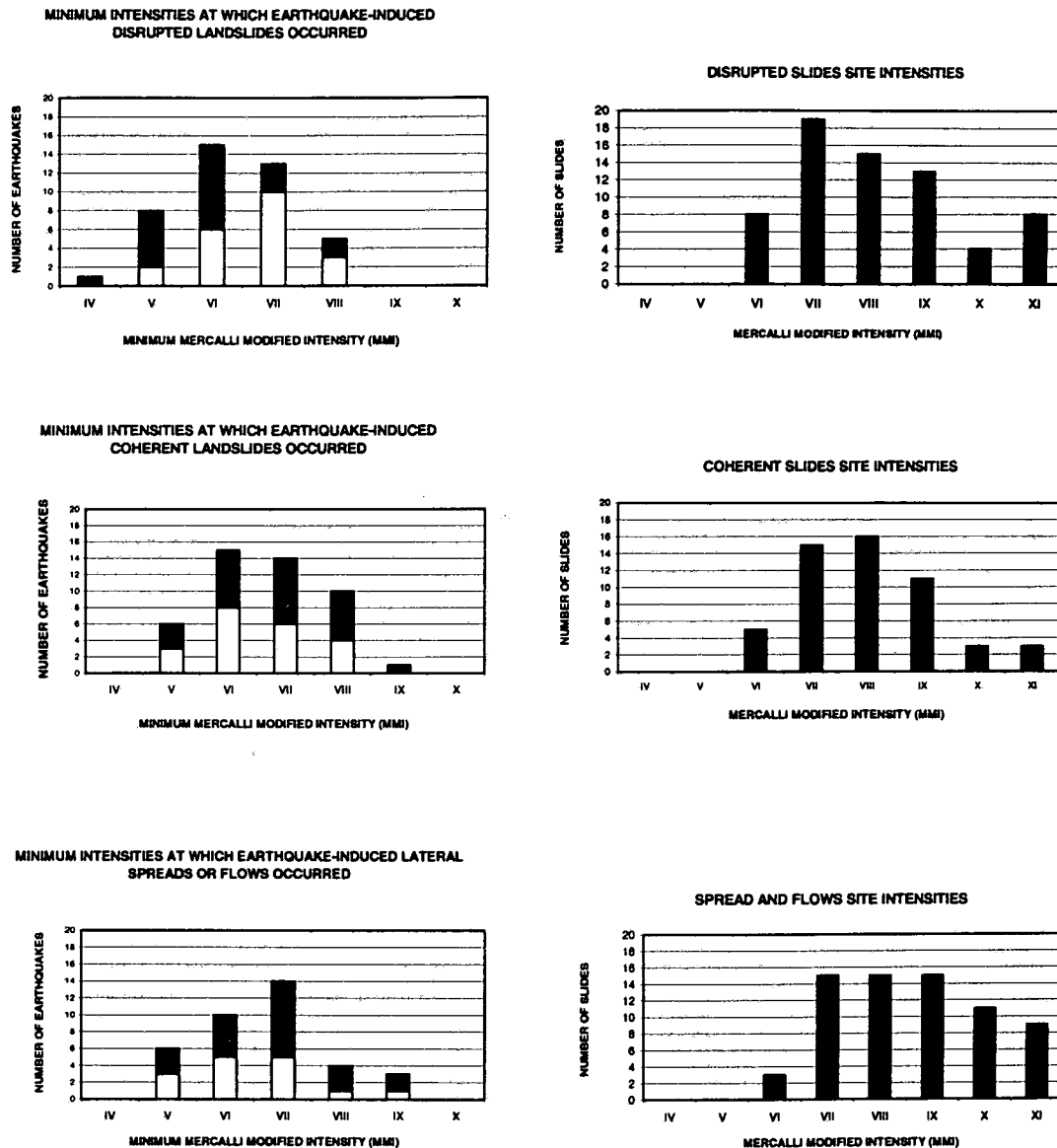


Figure 4.12 Mercalli Modified Intensity associated with different landslide mechanisms. Left graph shows the minimum intensities for each mechanism during different historic earthquakes; shadow areas represent Keefer's data. Right graph shows the intensity for individual landslides (Rodríguez *et al.*, 1999).

Correlations between earthquake magnitude and the total area affected by landslides, in the new database, using both M_S and M_W as the measure of earthquake size, are presented in Figure 4.13. The corresponding upper bounds found by Keefer (1984) and Rodríguez *et al.* (1999) are also drawn, and it can be seen that a number of the data points lie above these lines. This suggests that the upper bound of affected areas is above that suggested by the previous studies. Rodríguez *et al.* (1999)

suggested that data points above Keefer's line may be cases associated with multiple seismic events, but also slope susceptibility and climatic conditions may play an important role in determining extension of damage by landslides.

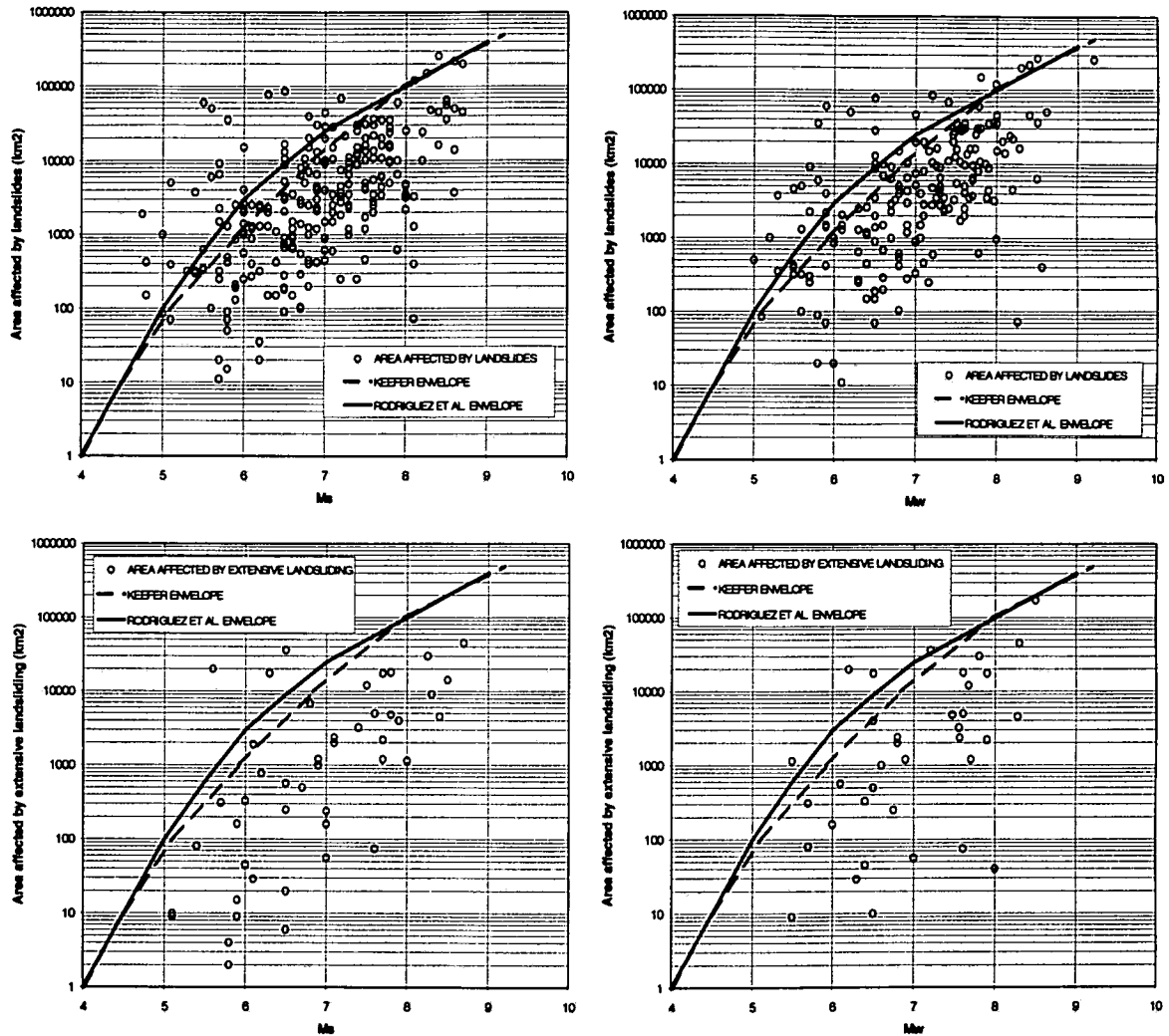


Figure 4.13 Area affected by landslides (km^2) as a function of earthquake magnitude M_s (left) and M_w (right), and density of landsliding low density (top) and high density (bottom). Dashed and solid lines are the upper limits proposed by Keefer (1984) and Rodríguez *et al.* (1999), respectively.

Another observation that can be made from Figure 4.13 is the considerable scatter of the data, which is much larger than that shown by Keefer and Rodríguez *et al.* Scatter has been attributed to different seismological and geological factors. The most significant element contributing to the scatter is likely to be the variation in the geographical environments where the earthquakes occur, with some located in areas of pronounced relief, whereas others occur in generally flat areas with isolated slopes.

Alternatively, landslides may be concentrated along man-made slopes, especially along highway cuts. Those cases which are associated with offshore events, or those in which the epicentre is very close to the coast, tend to fall well below upper bound lines.

A new upper limit is not suggested here because, as in the distance correlations, a global boundary is meaningless. Development of regional correlations and relations for similar geological environments seems to be more practical, as shown in Figure 4.14, which presents area-magnitude relations for cases in Central America. Similar to distance, the area affected by landslides shows to be a function of geology and failure mechanisms. Different upper limits can be drawn for El Salvador and Guatemala, and for Costa Rica and Panamá. The scatter of data in a given geological environment may be explained by differences in susceptibility including antecedent climatic conditions.

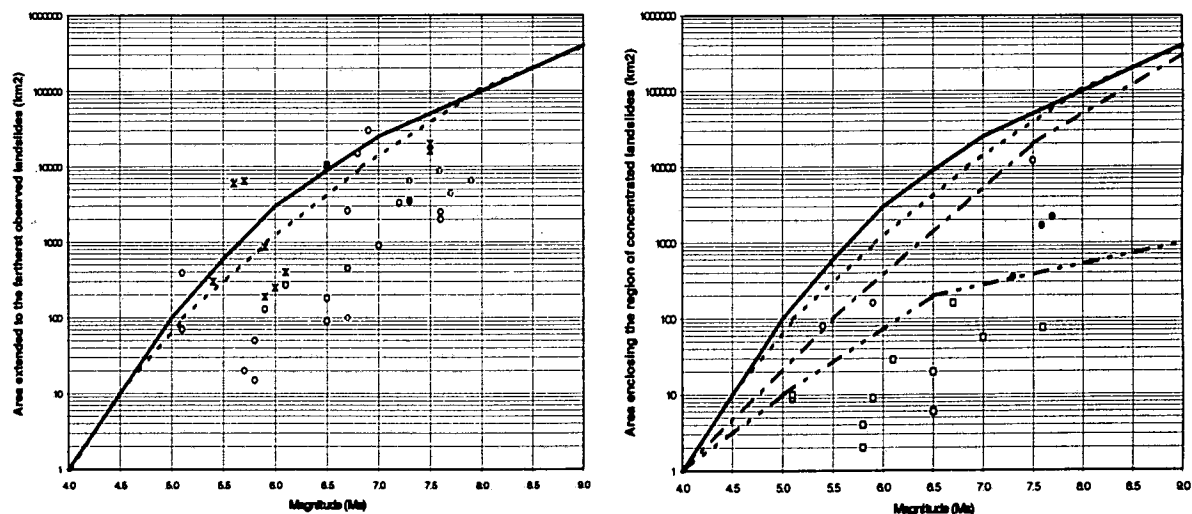


Figure 4.14 Area affected by landslides-Magnitude relationship for cases in Central America. Area extended to the farthest observed slide (left) and area enclosing extensive landsliding (right). On the left graph crossed points correspond to El Salvador and Guatemala case, for Panamá and Costa Rica otherwise. On the right graph circles correspond to El Salvador and Guatemala, and squares to Panamá and Costa Rica.

It is clear that although these cases belong to a unique geographic region, which is commonly considered homogeneous, the magnitude of triggering events, failure mechanisms, extension and intensity and potential damage by landslides are very different. This conclusion limits the applicability of graphs such as those presented in Figures 4.13 and 4.14.

Comparison of Figures 4.7 to 4.9 and Figure 4.13 reveals apparent incongruence since there are a number of cases that lie above upper bounds in terms of area affected by landslides, but not in terms of maximum distance of landslides from the source, particularly evident for distance from the fault projection. This is mainly due to the fact that these outliers in terms of affected areas represent cases where the zone of landsliding is more circular than elongated, and centred around the earthquake source. The cases of very long distances to the furthest landslide correspond to cases where the zone of landsliding is elongated and eccentric with respect to the fault rupture. The shape, extension and eccentricity of the landsliding area with respect to the earthquake source depends on regional slope susceptibility and directivity of the seismic radiation, as was demonstrated for the case of the 1991 Valle de la Estrella (Costa Rica) earthquake by Mora (1997b).

4.4.1.4 Ishihara and Nakamura (1987); Mora and Mora (1992)

These authors also proposed maximum distance to landslides-magnitude relationships (ISSMFE, 1993). In this case they discriminate data into different classes based on the density of landsliding. Ishihara and Nakamura show result for the 1987 El Napo earthquake in Ecuador, whereas Mora and Mora show data for Costa Rican earthquakes. Results are presented in Figure 4.15, which shows that these kind of relationships are more consistent for local than for regional data. Although magnitudes of Costa Rican earthquakes, included by Mora and Mora, have been re-evaluated (Ambraseys, 1995; Ambraseys and Adams, 1996), and their relationship is not as uniform as shown, the regional applicability of that relationship continues to be valid.

4.4.1.5 Japanese Meteorological Agency (JMA), 1993

The Japanese Meteorological Agency presented a frequency plot of minimum intensity (JMA intensity) where slope failures occurred during 11 recent Japanese earthquakes (ISSMFE, 1993). This plot is reproduced in Figure 4.16. As shown in this graph, there are some cases of landslides related to JMA intensities as low as III, even though level IV is the minimum where landslides are used as evidence, as stated in Chapter 2. This plot confirms the weakness of intensity data as predictive parameter.

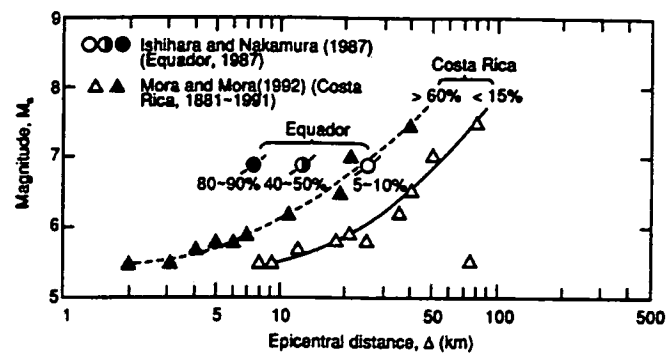


Figure 4.15 Maximum epicentral distance-Magnitude relationships for Ecuador and Costa Rica (ISSMFE, 1993).

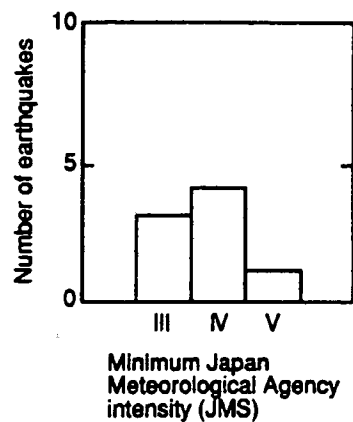


Figure 4.16 Minimum JMA intensities at which slope failures have occurred (From ISSMFE, 1993)

4.4.1.6 Maugeri *et al.* (1994)

Based on historical cases of earthquake-induced landslides in Southwestern China and Italy, Maugeri and co-workers presented some relationships between seismic parameters, mainly magnitude and macro seismic intensity, and landslide occurrence.

Relationships between area affected by landslides and magnitude for two different regions in China are presented in Figure 4.17. In this case relationships are obtained by linear correlation of data and not as an upper bound. It is strengthened by the fact that local data give more consistent results for this kind of relation.

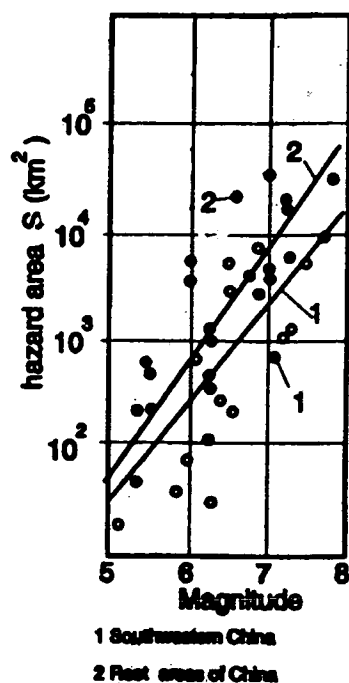


Figure 4.17 Magnitude-area affected by landslides relationship from China (Maugeri *et al.*, 1994)

These authors also stated that due to the linear character of faults, isoseismals have an elliptical contour, which is also assumed for areas affected by landsliding. Furthermore, they point out that for China and Italy, the ratio between the axes of this ellipse is 2:1, and prediction of maximum and minimum axes may be made from earthquake magnitude; some expression are given for China.

Based on case histories, Maugeri *et al.* proposed that slope susceptibility may be divided into three groups: potentially stable slopes; potentially unstable slopes and fairly unstable slopes. Characteristics of these types are presented in Table 4.6.

Although in general slope angles considered as criteria for different susceptibility grades are confirmed, by the present data, slope heights taken as critical limits seem to be overestimated. As was presented in the previous Chapter, slope heights for soil and rock slides may be found as low as 5 to 10 m. Geomorphology description in this method includes an important factor that is not usually considered, which is the erosion processes on the slope surface.

Table 4.6 Slope susceptibility criteria according to Maugeri et al. (1994)

SLOPE SUSCEPTIBILITY	SLOPE ANGLE	SLOPE HEIGHT	LITHOLOGY AND GEOMORPHOLOGY
Potentially stable slopes	Soils < 15° Rocks < 20°	Soils < 50 m Rocks < 100 m	Cemented, compacted sands or clays, or intact rock without weak intercalations. No evidence of slope deformation
Potentially unstable slopes	Soils < 20° Rocks < 30°	Soils < 100 m Rocks < 200 m	Flat slopes with few gullies and slight water erosion Interbedded sands and clays or weak rocks or jointed rocks Slope in which there exist saturated layers and weak intercalations, but gently dipping
Fairly unstable slopes	Soils > 20° Rocks > 30°	Soils > 100 m Rocks > 200 m	No flat slopes with numerous gullies. Soil slopes containing saturates silty sands, soft clays or other weak layer; rock slopes with numerous joints or major faults. Weak discontinuities may serve as sliding surface No favourable environmental and groundwater conditions to slope stability Old landslides or rock slides already existing or some evidence of slope deformation

4.4.1.7 Harp and Wilson (1995)

Arias Intensities calculated from accelerograms of the 24 October 1987 Superstition Hills and the 1 October 1987 Whittier Narrows earthquakes in southern California were used by Harp and Wilson for comparisons with the limits of landslides, specifically, falls and slides from rock and soil during these two events. These comparisons establish two shaking thresholds for the limits of falls and slides. A range of 0.08 to 0.6 m/s is observed for Tertiary and younger deposits, while the range of 0.01 to 0.07 m/s is observed for Mesozoic and older rocks. These two ranges of shaking levels were found by these authors to be controlled by the relative degree of fracturing and the aperture of fracture systems within the two general age groups of rocks and soils in the Los Angeles area and in Imperial Valley.

Minimum values of Arias Intensity for different cases in the present dataset are presented in Table 4.7. Values were calculate from epicentral and fault distances to different types of landslides by using regression expressions obtained from observed values for each earthquake. As shown in this table, minimum Arias Intensity ranges related to landslides are: 0.10 to 5.28 m/s for coherent slides; 0.01 to 2.70 m/s for disrupted slides; and 0.1 to 7.60 m/s for lateral spreads and flows. In general, it is apparent that disrupted slides need lower shaking intensity to be triggered, whereas coherent

slides and lateral spreads need higher shaking intensity, although, these values were calculated using the maximum distance at which each type of landslide was reported for each case, and this distance may be controlled by slope susceptibility.

Table 4.7 Minimum Arias Intensity related to different types of landslides for cases included in present dataset. Bold numbers calculated from distance to fault projection, otherwise from epicentral distance.

EARTHQUAKE	ARIAS INTENSITY THRESHOLD VALUES (m/s)		
	COHERENT SLIDES	DISRUPTED SLIDES	LATERAL SPREAD AND FLOWS
1933-03-11	0.93	0.09	1.34
1952-07-21	0.59	0.34	
1957-03-22	0.10	0.10	0.10
1966-06-28	0.14	0.11	
1971-02-09	1.71	0.06	3.42
1979-04-15		0.28	0.38
1979-10-15	0.26	0.13	0.26
1980-11-23	0.14	2.70	5.60
1981-04-26		0.73	0.73
1983-05-02		0.52	
1984-04-24		0.14	
1985-03-03	5.28	2.40	7.60
1986-10-10	1.17	1.55	
1987-10-01		0.21	
1988-11-25		0.01	
1989-10-18	0.96	0.13	0.96
1990-06-20	3.54	1.99	1.12
1992-03-13	0.98	0.74	0.74
1992-04-25	3.59	1.94	2.58
1992-06-28		0.38	
1994-01-17		0.26	
1995-01-16	4.13	1.57	2.81

4.4.2 Grade-2 methods

Due to the lack of information on material parameters, zoning maps based on a Grade-1 approach do not provide definitive information for site specific evaluations. Such assessments require zoning that incorporates additional topographical and geological data within the area concerned. This may require additional field investigations. To minimise the effort and expense of further investigations, however, existing topographical and geological reports should be used wherever possible.

4.4.2.1 Kanagawa Prefectural Government (1986)

Slope susceptibility within the Kanagawa Prefecture in Japan was defined based on slope failures during the 1974 Izuanto-oki, the 1978 Izuohshima-kinkai and 1984 Naganoken-seibu earthquakes

(ISSMFE, 1993). In this case four areas affected were divided into elements of 500 m x 500 m in 1:25.000 and 1:50.000 maps. From discriminant data analyses, seven factors were identified as contributing in landslide processes: a) maximum surface acceleration, b) length of contour line at the average elevation, c) maximum difference in elevation, d) hardness of rock in a typical slope, e) total length of faults, f) total length of artificial cut and filled slopes, and g) topography of typical slopes. Weighting factors and susceptibility ranks for this method are summarised in Table 4.8. Despite the simplicity of this method, its applicability is restricted to areas very similar to that where it was developed, if not only to that area.

The minimum acceleration considered as important by the Kanagawa Prefecture method is 0.2 g; this limit must be used carefully, since in the present earthquake-induced landslides dataset about 35% of the cases compiled lie below that limit. On the other hand, 37% of the data lie above 0.4 g, which was considered as a critical level. Peak ground acceleration for the present data ranges between 0.001 g and 1.34 g, which means that very low accelerations can trigger landslides depending on other factors such as climatic or groundwater conditions, which are not considered in this method. Magnitude and precedent rainfall conditions for the three earthquakes, from which the model was developed, are completely different and combining them without taking this into account can bias results. The 1984 earthquake had a magnitude of $M_S=5.6$, and occurred in a very dry season, whereas the 1978 earthquake had a magnitude of $M_S=6.7$, and yearly and six months precedent rainfalls are well above average rainfalls within the area. Finally, the 1974 earthquake had a similar magnitude to the 1978, $M_S=6.5$, but occurred after an exceptionally wet month.

As the weighting factor for soil sites is taken as zero, the model is restricted to rock failures, which also may explain the high weight given for very irregular and steep topographies. As stated in Chapter 3 rock slopes tend to be steeper and more irregular than soil slopes, and rock falls and rock slides are generally associated with slopes steeper than 35° . Rock deposits in this method are classified into two main groups: Soft and Hard Rocks. Within the region where model was developed volcanic deposits are predominant, hence this model is also restricted to these materials; moreover, those deposits classified as soft rocks may correspond to volcanic ashes and tephra, which have been shown to fail as rock and soil falls, apparently induced by tension stresses, as was

described previously for cases in Central America. This kind of deposits are more susceptible when artificial or natural cuts are present, which may explain the presence of this factor as contributing in the model.

Table 4.8 Weighting factors related to slope failure for the Kanagawa Prefecture, Japan (Based on ISSMFE, 1993)

FACTOR		CATEGORY	WEIGHT	
a) Maximum surface acceleration (g), W_1		0-0.204	0.0	
		0.204-0.306	1.004	
		0.306-0.408	2.306	
		>0.408	2.754	
b) Length of contour line (m), W_2		0-1000	0.0	
		1000-1500	0.071	
		1500-2000	0.320	
		>2000	0.696	
c) Difference between the highest and lowest site (m), W_3		0-50	0.0	
		50-100	0.550	
		100-200	0.591	
		200-300	0.814	
		>300	1.431	
d) Hardness of the rock, W_4		Soil	0.0	
		Soft rock	0.169	
		Hard rock	0.191	
e) Length of faults (m), W_5		No fault	0.0	
		0-200	0.238	
		200-	0.710	
f) Length of artificial slopes (m), W_6		0-100	0.0	
		100-200	0.539	
		>200	0.854	
g) Topography of slope, W_7		Convex	0.0	
		Concave	0.151	
		Linear	0.184	
		Concave-Convex transition	0.207	
Slope Susceptibility = $W = W_1+W_2+W_3+W_4+W_5+W_6+W_7$				
W	2.93		3.53	3.68
Rank	A	B	C	D
Number of landslides within one mesh	0	1-3	4-8	>9

Finally, weighting factors considered for profile shapes consider that convex slopes are not susceptible to landsliding, whereas concave slopes are taken to be more susceptible than linear slopes. These conditions may also represent specific characteristics within the area, which cannot be generalised. Take a weighting factor of zero for convex slopes should not be interpreted as these kinds of slopes not being susceptible, given that these slopes present a higher kinematic feasibility than other profiles, whereas concave profiles may be the result of the slope evolution.

4.4.2.2 Mora and Vahrson (1993, 1994)

Mora and Vahrson combined information from earthquake-induced and rainfall-induced landslides in Central America and particularly for Costa Rica. Factors included in this model are classified into triggering and susceptibility factors. Macroseismic intensity and rainfall are taken as triggering factors whereas relative relief, lithology and soil humidity are the susceptibility factors. These factors and their weights are presented in Table 4.9. Although this method is the only one found to combine rainfalls and seismicity, its applicability is also restricted to very particular conditions, and any attempt to use it in other regions, even within Central America, requires major revisions.

A recent and meticulous study has applied the Mora and Vahrson method to produce a map of landslide hazard in El Salvador (Alvarado *et al.*, 1998). The highest hazard rating assigned to any geo-reference on the map is grade IV (Medium), which does not concur with observations and data presented in this study. One possible reason for the apparent discrepancy is the use of 1 km x 1 km grid squares for the geo-references; Yasuda (1999) found that it was necessary to reduce the grid size considerably in his applications to three areas in Japan in order not to severely underestimate the landslide hazard.

In the method of Mora and Vahrson (1994) the relative relief is measured by the maximum difference in elevation in each 1 km² geo-reference, which reflects the fact that the most important and abundant landslides in Costa Rica occur on the slopes of hills and volcanoes, for which this measure of relief is appropriate. Mora and Vahrson (1994) clearly state that the optimum grid size needs to be identified for each individual application of the method. Squares of 1 km are unlikely to capture the small, near-vertical slopes that contribute significantly to the hazard. With a population density of almost 5,000 people per square kilometre in the Metropolitan Region of San Salvador, identifying a single hazard indicator at this geographical resolution may not be particularly useful for land-use and urban planning.

Table 4.9 Weighting factors and hazard assessment for the Mora and Vahrson method (Mora and Vharson , 1994)

SUSCEPTIBILITY FACTORS			
Relative relief (m/km²)		Susceptibility	Parameter S_r
0-75		Very low	0
76-175		Low	1
176-300		Moderate	2
301-500		Medium	3
501-800		High	4
>800		Very high	5
Lithology		Susceptibility	Parameter S_l
Permeable compact alluvium; permeable limestone, slightly fissured intrusions, basalt, ignimbrite, gneiss, hornfels; low degree of weathering, low water table, clean-ruggose fractures, high shear resistance		Low	1
Higher degree of weathering of above mentioned lithologies and hard massive sedimentary rocks; lower shear resistance and shearable fractures		Moderate	2
Considerably weathered sedimentary, intrusive, metamorphic, volcanic rocks, compacted sandy regolithic soil, considerable fracturing, fluctuating water tables		Medium	3
Considerably weathered, hydrothermally altered rocks of any kind, strongly fractured and fissured, clay filled; poorly compacted pyroclastic and fluvio-lacustrine soils, shallow water tables		High	4
Extremely altered rocks, low shear resistance alluvial, coluvial and residual soils, shallow water tables		Very high	5
Average monthly precipitation (mm/month)		Assigned value	
<125		0	
125-250		1	
>250		2	
Summation assigned values of precipitation averages for 12 months		Susceptibility	Parameter S_n
0-4		Very low	1
5-9		Low	2
10-14		Medium	3
15-19		High	4
20-24		Very high	5
TRIGGERING FACTORS			
Intensity (MMI), T_r=100 years		Susceptibility	Parameter T_s
III		Slight	1
IV		Very low	2
V		Low	3
VI		Moderate	4
VII		Medium	5
VIII		Considerable	6
IX		Important	7
X		Strong	8
XI		Very Strong	9
XII		Extremely strong	10
Maximum rainfall (mm), n>10 years; T_r=100 years	Rainfall (mm) n<10 years; average	Susceptibility	Parameter T_p
<100	<50	Very low	1
101-200	51-90	Low	2
201-300	91-130	Medium	3
301-400	131-175	High	4
>400	>175	Very high	5
HAZARD ASSESSMENT			
$H_i = (S_r \cdot S_l \cdot S_n) \cdot (T_s \cdot T_p)$			
H_i Value	Class	Susceptibility of hazard	
0-6	I	Negligible	
7-32	II	Low	
33-162	III	Moderate	
163-512	IV	Medium	
513-1250	V	High	
>1250	VI	Very high	

In terms of the S_i parameter, poorly consolidated pyroclastic soils are assigned a value of 4 on a range of 1 (low susceptibility) to 5 (high susceptibility), which seems reasonable. However, it appears that the factors presented by Mora and Vahrson (1994) for all of the parameters seem to be based on data primarily, if not exclusively, from Costa Rica, and therefore their applicability to other areas of Central America needs to be investigated.

Equation for hazard assessment given in Table 4.9 suggests that the landslide hazard increases linearly with the intensity of ground shaking, whereas in fact the evidence suggests that for steep slopes in volcanic soils there is a single threshold value for the ground-shaking inducing instability (depending on the ground-water conditions), below which the hazard may be close to zero. Keefer (1984) and Rodríguez *et al.* (1999) found that the threshold intensity for landsliding is commonly VI-VII on the Modified Mercalli scale. It is also questionable if the same return period should be applied to maximum rainfall and to seismic shaking, since the return periods of extreme climatic events is generally much shorter than for earthquakes. Furthermore, T_p has a maximum value of 5 and T_s a maximum of 10, which would suggest that seismic triggering has much higher weighting. However, intensities of XI and XII are almost never encountered in reality and for a return period of 100 years it is likely that T_s will not exceed 7 or 8.

Mora and Vahrson (1994) propose only that their method provides a framework that can be adapted to local and regional trends, although it has been adopted without modification in other parts of Central America. Clearly, for landslides in certain types of deposits, and particularly for steep slopes in volcanic soils, the method requires extensive revision. Mora and Vahrson (1994) also propose that further development of the method should include statistical analysis and it certainly seems to be the case that multivariate analysis of the various parameters involved could produce more robust and indeed more meaningful landslide hazard models.

Particular care must be taken with rainfall data when using this model since it was developed for Costa Rica, which is one of the most rainy areas in the world, and rainfall-induced landslides are very common, hence inclusion of rainfall-induced landslides in the data may bias results. Application of this method to other areas may be possible by including local rainfall data, although since the

authors did not explain how the weighting values were assigned to each parameter, it would be arbitrary to try to re-evaluate this factor alone.

Assessing the susceptibility factor as the product of three parameters leads to incongruent results and limits the model applicability to specific failure mechanisms, which are not mentioned in the analysis but can be presumed to be shallow translational slides on residual soils. Although alluvial and coluvial deposits are considered very susceptible, usually these deposits have a low relative relief, as defined by authors, these are even commonly found on level ground, which gives a zero relative relief value and consequently a negligible hazard. This means that lateral spreads and coherent slides in these deposits are excluded.

Steep slopes in fractured rocks, very common in seismically active zones, may be stable under dry conditions, although relative relief and lithology factors give a very susceptible slope, ground water conditions may substantially reduce the hazard value. However, as can be seen from present datasets rock falls are the most common failure mechanism during seismic events even for arid and semi-arid environments, such as Iran and Perú.

4.4.2.3 Japan Road Association (1988)

The Japan Road Association proposed a simple method to assess slope susceptibility based on data collected from the 1974 Izuohshima-kinkai earthquake (ISSMFE, 1993). Slope susceptibility is defined by using the twelve weighted factors shown in Table 4.10. This method appears very simple for assessing rock falls in road cuts, but it has a serious shortcoming, which is that it does not consider shaking intensity among the factors. Analysing the weighting values it is apparent that applicability of this method is restricted to rock falls due to proposed ranges of characteristics such as height, slope angle and overhanging, which are common in slopes affected by this mechanism during earthquakes, and these factor are weighted more than others factors common for different kind of failures such as lithology or water flow. On the other hand, the degree of detail needed for assessing most of the parameters limits its use to large scales.

Table 4.10 Slope stability assessment. Japanese Road Association (Based on ISSMFE, 1993)

PARAMETER	CATEGORY	WEIGHT OR RANK	
1. Height of slope, H (m)	H≥50 50≥H≥30 30≥H≥10 H<10	10 8 7 3	
2. Gradient of slope, α	α≥1:0.6 1:0.6>α≥1:1.0 α<1:1.0	7 4 0	
3. Overhang	Formed in no-walled slope Formed in walled slope Not formed	7 4 0	
4. Geology	Many unstable stones Many stones on the surface of slope Very weathered rock Gravelly soil Weathered rock Cracked rock Sand Clay Intact rock	10 7 6 5 4 4 4 1 0	
5. Thickness of weathered soil layer	More than 0.5 m Less than 0.5 m	3 0	
6. Water flow	Flow No flow	2 0	
7. Frequency of rock falls	More than once per year Less than once per year none	5 3 0	
8. Deformation of slope	Deformed Not deformed	I III	
9. Deformation of retaining wall	Deformed Not deformed	I III	
10. Rock falls which induce traffic problems	Occurred Not occurred	I III	
11. Cut or fill on slope	Many cut or fill Few cut or fill nothing	I II III	
12. Stability of retaining walls	Unstable Walls stable Very stable	I II III	
STABILITY ASSESSMENT			
STAGE 1. Without retaining walls			
	Summation of weights from 1 to 7		
Weighting based on 8 to 11	Less than 13	14 to 23	More than 24
Class I≥1	A	A	A
One class I, other class II	B	A	A
All class III	C	B	A
STAGE 2. With retaining walls			
	Stage 1		
Rank in 12	A	B	C
I	A	A	A
II	A	B	C
III	B	C	C
A: Slope failure likely B: Slope failure probable C: Slope failure not likely			

4.4.2.4 Keefer, 1993

Keefer, based on data from 24 world-wide earthquakes, defined a method for the assessment method of rock slope stability. Sixty units were analysed and stability ratings were associated to landslide concentration for each one. Landslide concentration was defined as the number of landslides per square kilometre.

Figure 4.18 shows schematically the decision tree used by this method. Factors included in the analysis are: slope height, slope inclination, degree of weathering, strength of induration, openness of fissures, vegetation and moisture conditions. Although fracturing and degree of weathering are considered, no value is given to the dip direction of discontinuities, which has been shown in many cases, including some within Keefer's database, to control the instability.

This method considers a slope inclination of 25° as a critical value since Keefer found that rock avalanches occurred above this limit as shown in Figure 4.19. Although Figure 4.20 shows that the slope angle limit remains applicable as a critical value, slope heights lower than 150 m, the critical value according to Keefer, may also be affected by rock slides. It is worth noting that within present dataset some rock slides are related to slopes with slope inclinations as low as 15°. However, no data on slope height for those rock slides was found.

For the geologic units, each slope was rated according to Figure 4.18. Then the results from the total number of slopes are averaged for each geologic unit, using the following relationship:

$$LS_{/km^2} = (32)(\% \text{ extremely high}) + (8)(\% \text{ very high}) + (2)(\% \text{ high}) + (0.125)(\% \text{ low}) \quad (4.6)$$

where the multipliers (32, 8, 2 and 0.125) are taken from landslide concentrations used to assign ratings. Percentage of area within each hazard level is determined from total area of landslides classified into each category according with Figure 4.18.

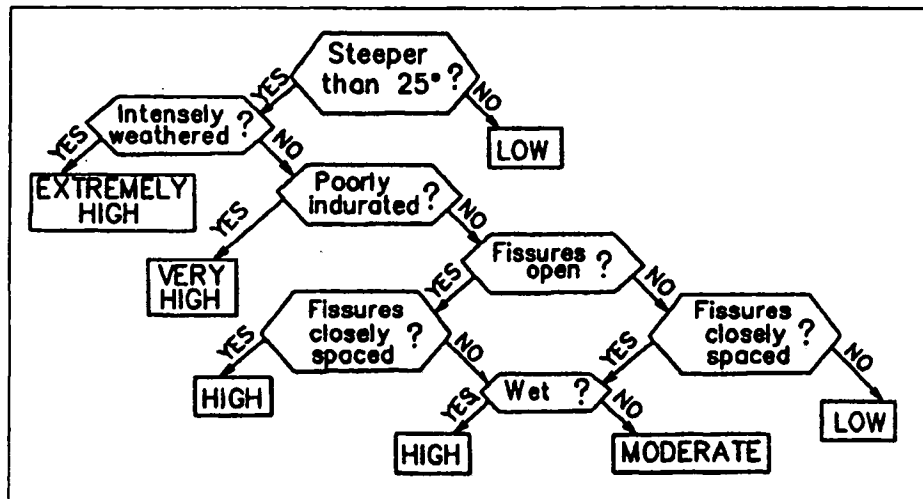


Figure 4.18 Decision tree for susceptibility of rock slopes for Keefer (1993) method (From Keefer and Wang, 1998)

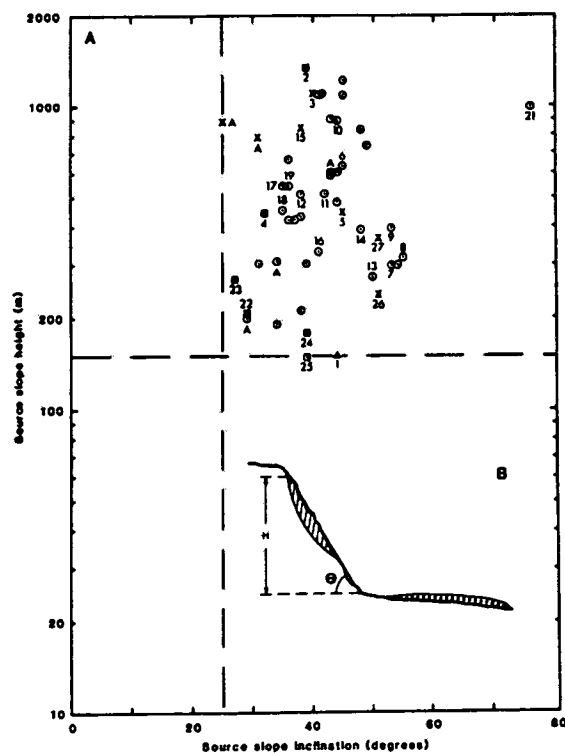


Figure 4.19 Heights and inclinations of rock avalanches induced by earthquakes. (From Keefer, 1984b)

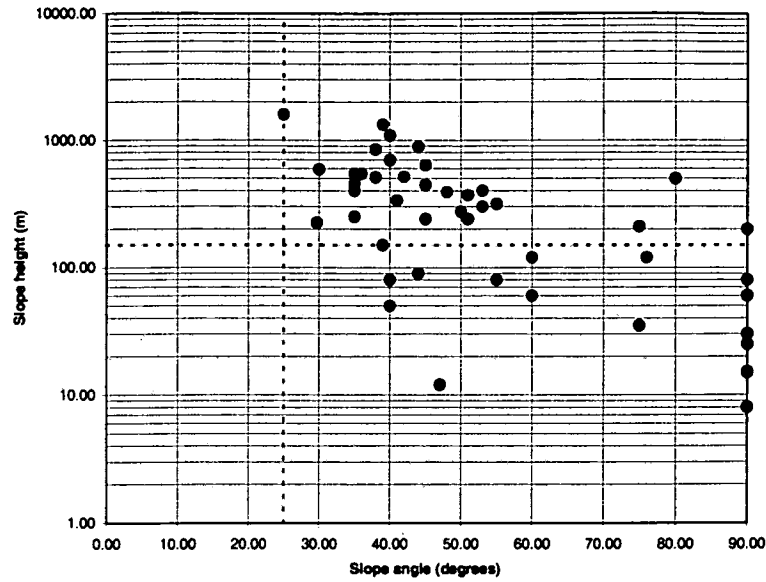


Figure 4.20 Slope angle and slope height for rock slope failures within present dataset.

4.4.2.5 Harp and Noble (1993)

The susceptibility of rock slopes to seismically-induced rock falls was defined by Harp and Noble (1993) based on Q values. This Q value is the Barton *et al.* (1974) parameter and expressed by Eq. (4.7):

$$Q = \left(\frac{RQD}{J_n} \right) \left(\frac{J_r}{J_a} \right) \left(\frac{J_w}{SRF} \right) \quad (4.7)$$

where RQD is the Rock Quality Designation defined by Deere (1963), J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor, and SRF is the stress reduction factor.

Analysis of rock fall concentrations and Q -values for the 1980 Mammoth Lakes, California, earthquake defines a well constrained upper limit that shows the number of rock falls per site decreases with increasing Q as shown in Figure 4.21.

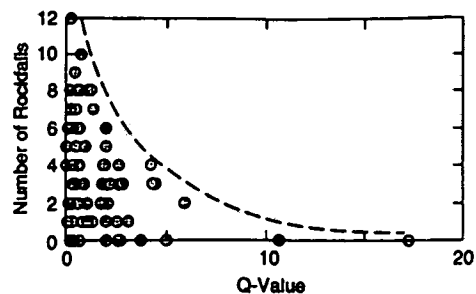


Figure 4. 21 Number of rock falls per site versus Q-value. The dashed line is the upper bound of data. (From Harp and Noble, 1993)

The first factor represents the rock quality in terms of fracturing and blocks size, the higher the rock fracturing, the lower the Q value. The second factor represents the discontinuities strength defined by discontinuity roughness and filling. Finally, the third factor represents the stress state by considering the tendency of zones of weakness to loosen the surrounding rock mass once stress relief has occurred, and the water pressures within rock mass. Harp and Noble stated that for seismically-induced rock falls, the water condition is usually dry and consequently take J_w equal to unity.

Harp and noble (1993) modify the SRF, which is concerned with the tendency of rock mass to become loose, in order to apply the Q value for slope stability, which was originally obtained for use in tunnelling. This factor was redefined to consider the openness or aperture of near-surface fractures in slopes and the orientation of pervasive fractures with respect to slope face; this factor was also renamed as *aperture factor* or AF since the authors considered that aperture is the single most important factor in determining rock mass susceptibility to rock slides since seismic vibration creates relative movement along joint surfaces. Individual blocks have many degrees of freedom in which to move when joint sets of several orientations are open.

The second most important factor in seismic rock stability is the orientation of the most pervasive fractures or joint sets. If major joints dip out of a slope or cliff face, the situation is favourable for failure along these joints, especially if other bounding joints are open. These factors were included

in the AF factor defined for this model. Under these considerations, the third factor in Eq. (4.2) is related to loosening of rock mass during seismic events.

The expression for Q to be applied in seismic rock stability is given as:

$$Q = \left[\frac{RQD}{J_n} \right] \left[\frac{J_r}{J_a} \right] \left[\frac{1}{AF} \right] \quad (4.8)$$

Q value was defined for 92 rock falls after the 1980 Mammoth Lakes earthquake; results are presented in Figure 4.21. The number of rock falls within a small area (0.09 km²) is plotted against the average Q value of cases within the area. It is seen in Figure 4.21 that number of rock falls decreases as Q value increases, and an upper bound is defined. Graph also shows that approximately 14 rock falls are expected in areas with zero Q value, and no rock falls in sites with Q values higher than 17.

The number of rock falls per site were counted for equal intervals of Q ($\Delta Q=0.25$), and used to calculate rock fall probability in terms of Q. Based on the relationship shown in Figure 4.21, representing an exponential decay, the probability density function of rock falls was defined by:

$$P = Nae^{-aQ} \quad (4.4)$$

where N is the sum of all mean values of rock falls per site integrated over all Q and a is the rate of decay of the probability of a rock fall. The mean number of rock falls per site, in any Q interval is equal to the integral of the probability density function over a specific interval of Q. Finally the probability that rock falls will occur within any given range of Q can be calculated. Based on percentage of landslides that lie within different Q ranges, four categories of slope susceptibility to rock falls are defined as shown in Table 4.11.

For application to other sites, the exponential decay of number of rock falls may be established and corresponding probabilities assessed to defined the appropriate local intervals of Q to define

different susceptibility levels. It would be useful to apply this methodology to different earthquakes in order to relate Q value ranges with earthquake magnitudes and local conditions.

Table 4.11 *Susceptibility categories based on Q value (Adapted from Harp and Noble, 1993)*

Q Value	Percentage of cases within Q interval	Category	Susceptibility to rock falls
0-1.41	50	A	Highly susceptible
1.41-2.83	25	B	Susceptible
2.83-3.87	10	C	Moderately stable
> 3.87	15	D	Mostly stable

As pointed out by the authors this method was obtained from the Mammoth Lakes earthquakes, which was a sequence that covered totally the area where rock falls occurred, hence all rock falls were considered equidistant from the earthquake source. However, application to other sites may consider the distance effect on the number of rock falls and Q relationships. Concentration of rock falls tends to increase near the source. Similarly, J_w was considered as 1.0 (dry conditions) but this is not always the case, therefore for other conditions, the most general expression for Q must be evaluated.

4.4.2.6 Feiznia and Bodaghi (2000)

This method was established for the north of Iran, and from field observation of parameters defined as effective in landslide occurrence, which are: a) elevation (E); b) slope gradient (s); c) slope aspect (D); d) susceptibility to earthquake (g), which were used as independent variables in a multivariate analysis. Regression analysis was performed not for actual values of slope angle nor for elevation but they were replaced by class values, as shown in Table 4.12. Similarly, slope aspect (the orientation of slope face) that was defined qualitatively was replaced by class factors. Finally, seismic susceptibility was expressed in terms of the PGA as a percentage of g.

Values of g used for regression were obtained from a local attenuation equation for PGA, and since the model is defined for a very dry zone it does not consider climatic factors, hence its applicability is restricted to regions of similar climatic and seismic conditions, if not to Iran only.

Table 4.12 Slope susceptibility classification. Feiznia and Bodaghi model (Adapted from Feiznia and Bodaghi, 2000)

Elevation classes and class values. Elevation in meters above sea level		
Class	Class value	
1400-2000	4	
2000-2600	2.2	
1000-1400	2	
2600-3200	0.7	
3200-4100	0.3	
Slope gradient classes and class values. Slope in percentage		
Class	Class value	
35-45	6	
20-35	5.4	
45-60	4	
10-20	2	
>60	1.5	
0-10	0	
Slope aspect classes and values.		
Class	Class value	
North	10	
East	8.5	
West	5.1	
South	4.1	
Slope instability index		
$Y=-9.47+0.82E+7.7g+0.52s+0.36D$		
Y value	Class	Slope susceptibility
<-0.5	A	Relatively resistant
-0.5<Y<0.5	B	Low
0.5<Y<1.5	C	Medium
1.5<Y<3.75	D	Susceptible
3.75<Y	E	Very susceptible

4.4.3 Grade-3 methods

By combining geotechnical surveys with methods of Grade-1 and Grade-2, a detailed level of zonation with high accuracy can be achieved for each hazard zone. Grade-3 zonation requires additional information suitable for performing detailed slope stability analysis. These analyses are usually performed on a site-specific basis, and given a sufficiently detailed site investigation, very reliable zonation maps can be compiled. Site investigation data from public and private sources should be used where possible.

4.4.3.1 Wilson *et al.* (1979)

Wilson *et al.* (1979) obtained the critical acceleration of shallow planar slope failure from the following expression:

$$a_c = g \left[\frac{c}{\gamma h} + \cos \theta \tan \phi - \sin \theta \right] \quad (4.10)$$

where a_c is the critical acceleration for the slope, g is acceleration of gravity, c is the cohesion, ϕ is the internal friction angle of the layer along the slip surface, θ is the slope angle, and h is the thickness of soil layer mobilised. Based on areal distribution of these factors a map of slope susceptibility may be plotted, however, no mention of how critical acceleration was used to define slope susceptibility was made. It can be presumed that this critical acceleration is compared with actual peak horizontal acceleration from strong motions in order to defined potential slide areas. Tanaka (1982, referred by ISSMFE, 1993) applied this method to the Nashimoto area in Japan affected by the 1978 Izuohshima-kinkai earthquake. Results are presented in Figure 4.22. This graph shows that actual landslides during the earthquake lay within areas predicted to be susceptible by using this model.

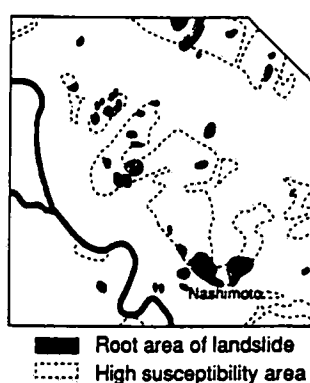


Figure 4.22 Map of Nasimoto area showing actual and predicted slope failures caused by the 1978 Izuohshima-kinkai earthquake (ISSMFE, 1993)

Applicability of this method is restricted to lithologies susceptible to shallow planar translational slides. Residual soil are one of those that have been shown to fail in this way, and probably shallow

dipping layered rock slopes. For other mechanisms the expression for critical acceleration must be reevaluated.

4.4.3.2 Yen and Condoretti (1980)

Yen and Condoretti (1980) developed a zonation method for slope susceptibility in foothill area sbased on data of the Lopez Canyon slope failures during the 1971 San Fernando earthquake. Field observations showed that the slides and subsequent mudflows occurred within the residual soil mantle and were not on bedding planes. All slope failures had a translational mode of failure. The physical model used was an elastic soil layer bonded to an unyielding bedrock. Maximum sliding length, x_{cr} , can be estimated by:

$$x_{cr} = \frac{\sigma_0 \left[1 + \left(\frac{\gamma h}{\tau} \right)^{1/2} \right]}{\left(\frac{\tau}{h} - \gamma a \right)} \quad (4.11)$$

where γ is the unit weight of soil, σ is the lateral stress at toe of a potential slide, x_{cr} is maximum sliding length, c and ϕ the soil strength parameters, h soil thickness, and :

$$\tau = c + \gamma h \cos \alpha \tan \phi - \gamma h \sin \alpha \quad (4.12)$$

$$a = \frac{\tau}{\gamma h} \left[\frac{\sigma_0 + c \cot \phi}{\sigma_0} \right]^2 \quad (4.13)$$

a is the effective acceleration which accounts for the accumulative shaking effects causing sliding failure. α is the slope angle. Slope susceptibility is defined from combining slope angle, previous sliding activity, faulting or weakness of the zone and slope length; suggested susceptibility levels are presented in Table 4. 13.

Table 4.13 Criteria for slope susceptibility used by Yen and Condoretti (1980).

Ranking	Parameter			
	Slope angle (degrees)	Existing slides	Existing faults or weakened soils	Slope length > x_{cr}
High	30-50	Yes	Yes	Yes
Moderate	30-50	No	Yes	Yes
Low	30-50	No	No	Yes
	>50	No	Yes	No
	<30	No	Yes	No
No failure anticipated	>50	No	No	No
	<30	No	No	No

These authors stated that this method is applicable to future events equal to or smaller in size than the previous events at a given site. This is because the strength of the soil weakened by previous shaking will be smaller than or equal to the pre-shaking strength. They also recommended that this method should not be generalized because it was defined with local data, and consequently similar expression may be defined for different earthquakes after field reconnaissance of slope failures.

In order to apply this method to larger earthquakes, Yen and Condoretti defined a pseudo-static yield acceleration, a_y , given by:

$$a_y = \cos \alpha \tan \phi + \frac{c}{\gamma h} - \sin \alpha \quad (4.14)$$

The acceleration calculated from field data to account for the accumulative shaking effect which cause a slope of length x_{cr} to fail is given by:

$$a_f = \frac{\tau}{\gamma h} + \frac{2\tau_0}{\gamma x_{cr}} + \frac{\tau_0^2 h}{\gamma x_{cr}^2} \quad (4.15)$$

For Lopez Canyon cases during the San Fernando earthquake, a_y and a_f were calculated and normalised with respect to PGA at the site. Relationship between a_y and a_f normalised are used to extend results to greater earthquakes.

Although this model includes the cyclic effect and failure length in slope stability analysis, and it is an improved model for shallow planar slides compared with previous methods, it is not clear how

the zonation works, particularly the slope length defined by Eq. (4.11). Apparently, the authors' assumption is that it is not sufficient to have an acceleration higher than that related to previous slides, calculated by using Eq. (4.13), but it is also necessary to have a minimum slope length for the slide to develop, given by x_{cr} .

Application to larger earthquakes is based on the relationship between theoretical critical acceleration and actual critical acceleration. These authors have found that actual value is lower than the theoretical; similar results have been found by Chaplin (1999) and Charalambos (1999). Although this consideration is an important contribution in seismic-slope analysis, Eq. 4.15, used to define the actual acceleration, is dimensionally inconsistent. Hence application of the method in this case must be carried out with care.

4.4.3.3 Wilson and Keefer (1985)

Based on the Newmark's sliding block model, Wilson and Keefer (1985) developed a method for evaluating slope susceptibility in terms of expected slope displacements and critical displacement values for different types of slides. Critical displacements values were assumed to be 10 cm for coherent slides and 2 cm for disrupted slides. Critical accelerations corresponding to each critical displacement were obtained for different earthquake records in California, as shown in Figure 4.23; these accelerations are defined as $(A_c)_2$ and $(A_c)_{10}$, for 2 and 10 cm, respectively. Slopes with critical accelerations higher than these two values should survive this shaking without severe damage.

In order to relate critical displacements to Arias Intensity, $(A_c)_2$ and $(A_c)_{10}$ values as those obtained from Figure 4.23, were plotted against Arias Intensity corresponding to each earthquake. Figure 4.24 shows this relation obtained for 10 different records.

Application of this relationship to slope susceptibility analysis is based on predicting Arias Intensity for a specific site, from which $(A_c)_2$ and $(A_c)_{10}$ are obtained, and compared with critical acceleration of that particular slope.

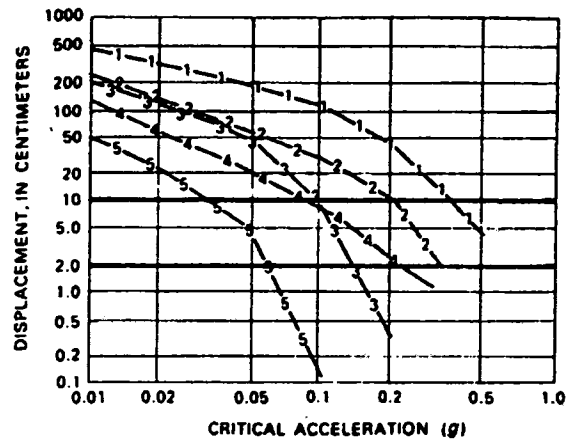


Figure 4.23 Displacements predicted by Newmark analysis as a function of assumed critical acceleration for several strong motion records from California earthquakes. (From Wilson and Keefer, 1985)

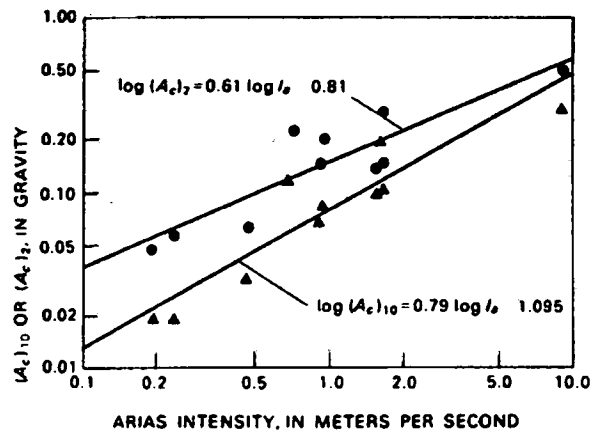


Figure 4.24 $(A_c)_2$ and $(A_c)_{10}$ as a function of Arias Intensity for 10 strong-motion records (Wilson and Keefer, 1985)

Comparing the magnitude-distance relationships for different types of landslides included within the Keefer (1984) data base, and magnitude-distance-Arias Intensity relationship derived by Wilson and Keefer, minimum threshold Arias Intensity values required to trigger coherent slides, disrupted slides, and lateral spreads and flows are: 0.5 m/s, 0.15 m/s and 0.5 m/s, respectively. These values can also be used to define the slope susceptibility. Alternatively, from these Arias Intensity values, the minimum acceleration required to trigger landslides is established to be 0.05 g.

Displacements for different critical accelerations due to different records obtained during the 1989 Loma Prieta and 1994 Northridge earthquakes were obtained by Rodríguez and Alarcón (1997) and plotted in similar way that Figure 4.23. In this case, for each record, maximum acceleration was

obtained, and critical displacement values of 1, 5 and 10 cm were related to critical and maximum accelerations. Given this relationship the return period of maximum acceleration, and consequently of earthquake magnitude, capable of inducing slope failure, may be defined for a specific site knowing its critical acceleration.

Jibson and Keefer (1993) carried out similar analyses for eleven records having Arias Intensities between 0.2 and 10 m/s. For each record they calculated the Newmark displacement for different critical accelerations. The resulting data were analysed by multivariate statistical analysis and a relationship between these parameters was found, and presented as the design chart reproduced in Figure 4.25. For a particular slope, critical acceleration is determined from a pseudo-static analysis, and a critical displacement may also be determined. Using the chart in Figure 4.25, the minimum Arias Intensity required to trigger slope failure is obtained, which can be compared with predicted values to assess slope susceptibility.

Although this method defined different threshold values of displacements, accelerations and Arias Intensity for different failure types, these values come from the Newmark sliding block model, which determines critical acceleration assuming planar failure and rigid block behaviour. As pointed out previously, actual critical accelerations have been found to differ from these values, and threshold values will also differ from those proposed.

Several studies have been carried out to develop a simplified approach for obtaining Newmark displacements without integrating strong-motion records. Yegian *et al.* (1988) derived closed-form solutions for displacements caused by periodic rectangular, triangular and sinusoidal input motions. Solutions are presented as a chart of normalised displacements and critical acceleration ratios, as shown in Figure 4.26. Ambraseys and Menu (1988) developed multivariate regression equations, based on actual ground motions that express Newmark displacements D_N as a function of critical acceleration and PGA, the following equation was proposed:

$$\log D_N = 0.90 + \log \left[\left(1 - \frac{a_c}{a_{\max}} \right)^{2.53} \left(\frac{a_c}{a_{\max}} \right)^{-1.09} \right] \quad \sigma_{\log D_N} = 0.30 \quad (4.16)$$

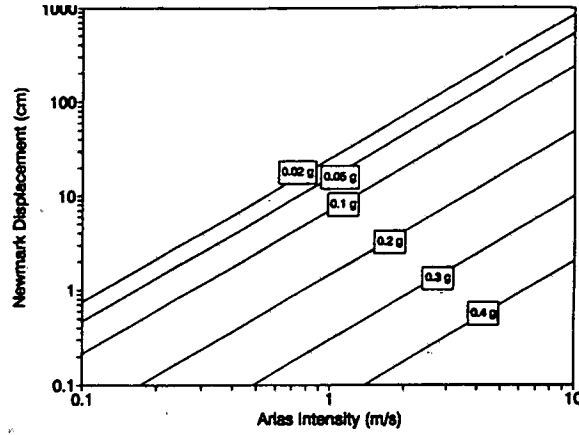


Figure 4.25 Newmark displacement as a function of Arias Intensity for several values of critical acceleration (Jibson and Keefer, 1993).

Yegian *et al.* (1991) considered frequency content and earthquake duration in developing the following equation, which was obtained from 354 records integrated by Franklin and Chang (1977, referred by Ho and Miles, 1997):

$$\log \left[\frac{D_N g}{a_{\max} N_{eq} T^2} \right] = 0.22 - 10.12 \frac{a_c}{a_{\max}} + 16.38 \left(\frac{a_c}{a_{\max}} \right)^2 - 11.48 \left(\frac{a_c}{a_{\max}} \right)^3, \quad \sigma_{\log D_N} = 0.45 \quad (4.17)$$

where N_{eq} is the equivalent number of cycles and T is the predominant period of the input motion. Jibson (1993) used Arias Intensity, AI rather than PGA to better characterise the damaging effects of ground motion; the regression equation developed from eleven records was:

$$\log D_N = 1.460 \log AI - 6.642 a_c + 1.546 \quad \sigma_{\log D_N} = 0.409 \quad (4.18)$$

Ambraseys and Srbulov (1995) proposed the following expression from North American and Asian records:

$$\log D_N = -2.41 + 0.47 M_s - 0.010 r + \log \left[\left(1 - \frac{a_c}{a_{\max}} \right)^{2.64} \left(\frac{a_c}{a_{\max}} \right)^{-1.02} \right] \quad \sigma_{\log D_N} = 0.58 \quad (4.19)$$

where r is the hypocentral distance. Jibson *et al.* (1998) proposed an other relationship between Newmark displacements and Arias Intensity, obtained from data from North America and Iran, given by:

$$\log D_N = 1.521 \log I_a - 1.993 \log a_c - 1.546 \quad \sigma_{\log D_N} = 0.375 \quad (4.20)$$

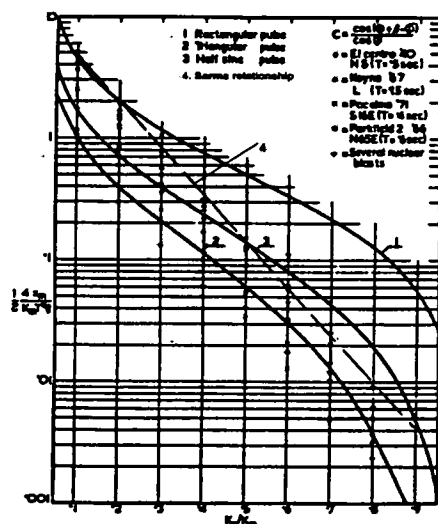


Figure 4.26 Normalised displacement – critical acceleration ratio relationship for rectangular, triangular and sinusoidal motion (Based on Yegian et al., 1988)

As stated by Ho and Miles (1997), these simplified approaches do not adequately characterise the input ground motion and do not account for soil response. The majority of the approaches were developed as a function of the critical acceleration ratio and Newmark displacements. Jibson (1993) states that there is no universal relationship between critical acceleration ratio and Newmark displacement. It is also commonly accepted that PGA does not correlate with damage. Although the model proposed by Jibson (1993) gives a better correlation, it was developed from only eleven records within a specific geographic region, which limits its applicability; Jibson and Keefer (1993) have found that its use for other regions underestimates the Newmark displacements.

Critical displacements considered for each type of slope failure may oversimplify the slope material behaviour, and no general rules should be established. A more rational way to carry out the analysis in terms of Newmark displacements was proposed by Legg *et al.* (1982, referred by Lin, 1990). Threshold values of critical accelerations obtained from critical displacements may also be

oversimplified, and similarly, Legg *et al.* also present a better way to relate slope susceptibility to this parameter, as shown in Table 4.14. Threshold values of Arias intensity found by this method are consistent with the present dataset, as shown in Table 4.6. That is, coherent and lateral spreads and flows require higher shaking intensity than disrupted slides; this may be because disrupted slides are frequently associated with brittle materials such as residual soils and volcanic tuffs, which fail under lower displacements.

Many hazard zonation models have been developed based on Newmark displacements, which use critical acceleration from the simplified planar failure model. No new contributions have been introduced by them except to implement the model in GIS frameworks, and in some cases the stochastic modelling of the strong-motion record. Table 4.15 compiles some application examples of Newmark displacement models for landslide hazard assessment. Due to the volume of data required to assess medium and large scale landslide hazard, data entry and verification are the greatest limitations of GIS applications.

Table 4.14 Slope failure level for different displacements and accelerations after Legg *et al.*, 1982 (Adapted from Lin, 1990)

Slope failure state based on displacements		
Failure state	Damage description	Permanent displacement (cm)
Light	Insignificant ground movement, no apparent potential for landslide failure, ground shaking only effect.	$D < 0.5$
Moderate	Moderate ground failure, small cracks likely to form (having effects similar to lurching phenomena)	$0.5 \leq D \leq 5$
Heavy	Major ground failure, moderate cracks and landslide displacements likely (having effects similar to liquefaction, lateral spreads phenomena)	$5 \leq D \leq 50$
Severe	Extreme ground failure, large cracks and landslide displacements likely (having effects similar in severity to large scale fault rupture)	$50 \leq D \leq 500$
Catastrophic	Total failure, landslides move large distance carrying everything with it	$D > 500$
Relative seismic slope stability based on critical acceleration		
Critical acceleration		Stability level
$0.7g \leq Ac$ $0.5g \leq Ac \leq 0.7g$ $0.3g \leq Ac \leq 0.5g$ $0.1g \leq Ac \leq 0.3g$ $0.01g \leq Ac \leq 0.1g$ $Ac \leq 0.01g$		Very stable Stable High Moderate Low unstable

4.4.3.4 Lin (1990)

Lin (1990) presented a method for constructing seismic slope failure probability matrices. The method is based on a probabilistic sliding block model incorporating the uncertainties associated with ground motion and strength properties of the slope materials. The extent of damage to a slope is defined in terms of the magnitude of the earthquake-induced permanent displacement. The intensity of the ground shaking is expressed by PGA and earthquake magnitude (scale not specified). Possible scatter in ground motion is included by using an equivalent stationary motion model.

Table 4.15 Examples of earthquake-induced landslide hazard zonation based on Newmark sliding block model.

Author	Parameter used for hazard assessment	Application example
Luzi and Pergalani, 1995	Newmark displacement, using a historical event within region	Marche region, Italy
Tika and Pitalikis, 1995	Newmark displacement	Epirus, Western Greece
Ho & Miles, 1997, 1999	Newmark displacement, with stochastic analysis of synthetic ground motions.	Berkeley, California
Jibson <i>et al.</i> , 1998	Newmark displacement obtained from correlation with Arias Intensity	Los Angeles, California
Khazai <i>et al.</i> , 1998	Newmark displacement obtained by using Ambraseys and Menu relationship	San Francisco Bay, California
Mankelaw and Murphy, 1997	Factor of safety from pseudo-static analysis and Newmark displacements from Jibson, 1993 expression. Hazard defined as probability of exceedence a limit displacement (50 mm)	North Santa Cruz Mountains, California
Miles and Keefer, 1999	Newmark displacements obtained by using the Ambraseys and Menu; Yegian <i>et al.</i> ; Jibson <i>et al.</i> expressions, and compared them with Ho and Miles procedure.	Oakland, California
McCalpin, 1998	Newmark displacement obtained from the Jibson expression	Puget Sound, Washington

Probability matrices for different slope damage levels, as defined by the Legg *et al.* (1992) descriptions presented in Table 4.13, were obtained by varying critical acceleration uncertainty, PGA and earthquake magnitude. The probability of exceedence of a given displacement level, which defines different slope damage levels, is obtained by considering uncertainties in strong motion and critical acceleration from records during the 1971 San Fernando earthquake. Figure 4.27 shows the probability matrices obtained.

(a) No uncertainty in A_c
 $A_c = 0.2g$

Damage state	Magnitude 5.5, A_{max} (g)					
	0.05	0.1	0.2	0.3	0.4	0.5
Light	100	100	100	80	15	0
Moderate	0	0	0	20	77	70
Heavy	0	0	0	0	8	30
Severe	0	0	0	0	0	0
Catastrophic	0	0	0	0	0	0
$\sum P$ (%)	100	100	100	100	100	100

Damage state	Magnitude 6.5, A_{max} (g)					
	0.05	0.1	0.2	0.3	0.4	0.5
Light	100	100	100	80	8	0
Moderate	0	0	0	20	79	54
Heavy	0	0	0	0	13	46
Severe	0	0	0	0	0	0
Catastrophic	0	0	0	0	0	0
$\sum P$ (%)	100	100	100	100	100	100

Damage state	Magnitude 7.5, A_{max} (g)					
	0.05	0.1	0.2	0.3	0.4	0.5
Light	100	100	100	80	2	0
Moderate	0	0	0	20	78	33
Heavy	0	0	0	0	20	64
Severe	0	0	0	0	0	3
Catastrophic	0	0	0	0	0	0
$\sum P$ (%)	100	100	100	100	100	100

(b) A_c is lognormally distributed
 $E(A_c) = 0.2g$, $V(A_c) = 0.4$

Damage state	Magnitude 5.5, A_{max} (g)					
	0.05	0.1	0.2	0.3	0.4	0.5
Light	100	100	93	59	27	9
Moderate	0	0	7	36	53	49
Heavy	0	0	0	5	20	40
Severe	0	0	0	0	0	2
Catastrophic	0	0	0	0	0	0
$\sum P$ (%)	100	100	100	100	100	100

Damage state	Magnitude 6.5, A_{max} (g)					
	0.05	0.1	0.2	0.3	0.4	0.5
Light	100	100	92	56	25	8
Moderate	0	0	8	36	47	40
Heavy	0	0	0	8	27	47
Severe	0	0	0	0	1	5
Catastrophic	0	0	0	0	0	0
$\sum P$ (%)	100	100	100	100	100	100

Damage state	Magnitude 7.5, A_{max} (g)					
	0.05	0.1	0.2	0.3	0.4	0.5
Light	100	100	90	53	22	7
Moderate	0	0	9	34	39	29
Heavy	0	0	1	13	35	51
Severe	0	0	0	0	4	13
Catastrophic	0	0	0	0	0	0
$\sum P$ (%)	100	100	100	100	100	100

Figure 4.27 Probability of slope damage as a function of earthquake magnitude, PGA and A_c uncertainty. (Lin, 1990)

For regional slope failure analysis, critical acceleration distribution was assumed to be uniform within different intervals that defined different damage levels, and probabilities for these intervals for each damage state was calculated. Probability matrices for regional slope susceptibility are presented in Figure 4.28.

Slope stability: unstable
 $A_c \leq 0.01g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	0	0	0	0	0	0	0	0
Moderate	37	5	0	0	0	0	0	0
Heavy	57	70	40	25	15	10	7	5
Severe	6	25	57	68	73	74	73	71
Catastrophic	0	0	3	7	12	16	20	24
$\sum P (%)$	100	100	100	100	100	100	100	100

Slope stability: low
 $0.01g \leq A_c \leq 0.1g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	89	59	4	0	0	0	0	0
Moderate	10	28	43	13	2	0	0	0
Heavy	1	13	45	63	55	41	28	19
Severe	0	0	8	23	41	55	64	69
Catastrophic	0	0	0	1	2	4	8	12
$\sum P (%)$	100	100	100	100	100	100	100	100

Slope stability: moderately low
 $0.05g \leq A_c \leq 0.15g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	100	98	41	2	0	0	0	0
Moderate	0	2	46	48	18	5	1	0
Heavy	0	0	13	46	68	66	54	41
Severe	0	0	0	4	14	29	44	56
Catastrophic	0	0	0	0	0	0	1	3
$\sum P (%)$	100	100	100	100	100	100	100	100

Slope stability: moderate
 $0.1g \leq A_c \leq 0.3g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	100	100	94	62	27	4	0	0
Moderate	0	0	6	32	48	47	29	14
Heavy	0	0	0	6	24	44	60	66
Severe	0	0	0	0	1	5	11	20
Catastrophic	0	0	0	0	0	0	0	0
$\sum P (%)$	100	100	100	100	100	100	100	100

slope stability: moderately high
 $0.2g \leq A_c \leq 0.4g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	100	100	100	99	76	41	11	1
Moderate	0	0	0	1	23	50	61	48
Heavy	0	0	0	0	1	9	27	48
Severe	0	0	0	0	0	0	1	3
Catastrophic	0	0	0	0	0	0	0	0
$\sum P (%)$	100	100	100	100	100	100	100	100

slope stability: high
 $0.3g \leq A_c \leq 0.5g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	100	100	100	100	100	87	54	21
Moderate	0	0	0	0	0	13	43	64
Heavy	0	0	0	0	0	0	3	15
Severe	0	0	0	0	0	0	0	0
Catastrophic	0	0	0	0	0	0	0	0
$\sum P (%)$	100	100	100	100	100	100	100	100

slope stability: very high
 $0.4g \leq A_c \leq 0.6g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	100	100	100	100	100	100	92	65
Moderate	0	0	0	0	0	0	8	34
Heavy	0	0	0	0	0	0	0	1
Severe	0	0	0	0	0	2	0	0
Catastrophic	0	0	0	0	0	0	0	0
$\sum P (%)$	100	100	100	100	100	100	100	100

slope stability: stable
 $0.5g \leq A_c \leq 0.7g$

Damage state	$A_{max} (g)$							
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
Light	100	100	100	100	100	100	100	95
Moderate	0	0	0	0	0	0	0	5
Heavy	0	0	0	0	0	0	0	0
Severe	0	0	0	0	0	0	0	0
Catastrophic	0	0	0	0	0	0	0	0
$\sum P (%)$	100	100	100	100	100	100	100	100

Note: (i) Constructed using ground motions recorded on rocks during the magnitude 6.6 San Fernando earthquake.
 (ii) A_c is uniformly distributed within any specified interval.

Figure 4.28 Probability of slope damage as a function of critical acceleration and PGA. (Lin, 1990)

4.4.3.5 Siyahi and Ansai (1993)

Based on pseudo-static slope stability analysis, Siyahi and Ansai (1993) proposed a method for slope stability utilizing a seismic coefficient A to account for the earthquake-induced horizontal forces. The variation in shear strength with depth is assumed linear and the potential failure surface is taken as a circular arc as shown in Figure 4.29.

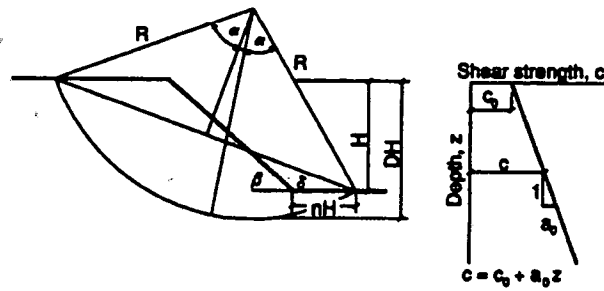


Figure 4.29 Section of slope used by Siyahi and Ansai (1993)

The parameters α , β , δ and n are related to the geometry of the slope and the configuration of sliding surface. Factor of safety is defined by:

$$FS = \frac{a_0}{\gamma} N_1 + \frac{c_0}{\gamma H} N_2 \quad (4.21)$$

where,

$$N_1 = \frac{3(\alpha + \cot \delta - \alpha \cot \alpha \cot \delta)}{\sin^2 \alpha \sin^2 \delta (D_1 + D_2)}$$

$$N_2 = \frac{6\alpha}{\sin^2 \alpha \sin^2 \delta (D_1 + D_2)}$$

$$D_1 = 1 - 2 \cot \beta - 3 \cot \alpha \cot \beta + 3 \cot \beta \cot \delta + 3 \cot \delta \cot \alpha - 6n \cot \beta - 6n^2 - 6n \cot \alpha + 6n \cot \delta$$

$$D_2 = A(\cot \beta + \cot^3 \delta + 3 \cot \alpha \cot^2 \delta - 3 \cot \alpha \cot \beta - 6n \cot \alpha \cot \delta)$$

The most critical surface can be determined by changing α , δ and n to obtain N_1 and N_2 as a function of β and A . It becomes possible at this stage to calculate the minimum factor of safety if the soil strength parameters are known. Hazard levels were defined in terms of factor of safety as follows: High for $FS < 1$; moderate for $1 \leq FS \leq 1.5$; and low for $FS > 1.5$.

4.4.3.6 Keefer and Wang (1998)

Slope stability under dynamic conditions are considered by Keefer and Wang (1998) for a wide spectrum of landslide failure types for slopes that range from steep to gentle. Steep slopes ($>25^\circ$) are most susceptible to rock falls and other fast moving landslides; moderate slopes, between 5° and 25° , are susceptible to deep-seated rotational and translational block slides, and gentle slopes ($< 5^\circ$) may be susceptible to liquefaction-induced lateral spreads. The method proposed by Keefer and Wang produces hazard maps derived from dynamic slope stability and liquefaction analysis, empirical correlations of slope stability with engineering properties of materials, and manipulation of data on local topography, engineering geology, and hydrology using GIS tools. Slopes are divided into four groups: (A) existing landslides; (B) steep slopes; (c) moderate slopes, and (D) gentle slopes. Groups B, C and D are assumed to have different modes of failure.

Existing landslides are assumed to be inherently unstable during seismic events, and a very high susceptibility was assign to them. Susceptibility of steep slopes was assessed by using the Keefer (1993) method described previously. Slopes within the moderate category are considered as susceptible to coherent, relatively deep-seated translational and rotational slides, and are analysed by using a design earthquake to calculate Newmark displacements using Arias Intensity-displacement relationship proposed by Jibson (1993).

Slope susceptibility is divided into three levels depending on displacements as follows: high ($D_N > 100$ cm); medium ($100 \text{ cm} > D_N > 10$ cm); and low ($D_N < 10$ cm). Gentle slopes underlain by units that are pre-Quaternary are assumed to be stable and are automatically given a susceptibility rating of nil. For soils that are susceptible to liquefaction, defined by material properties such as SPT value or

shear wave velocity, lateral spreading is estimated by using the Barlett and Youd (1992) method, which is expressed as:

$$\log D_N = -15.787 + 1.178M - 0.927 \log R - 0.013R + 0.429 \log S + 0.348 \log T_{15} + 4.527 \log(100 - F_{15}) - 0.922 D_{50_{15}} \quad (4.22)$$

where D_N is lateral spread in m, M is moment magnitude, R is horizontal distance to nearest seismic energy source, in km, S is ground slope in percentage, T_{15} is the cumulative thickness, in m, of saturated cohesionless soils with $(N_1)_{60}$ value ≤ 15 ; F_{15} is the average fines content in %, and $D_{50_{15}}$ is the mean grain size. A susceptibility rating is assigned to gentle slopes based on the lateral spread value as: high ($D_N > 100$ cm); medium ($100 \text{ cm} > D_N > 10$ cm), and low ($D_N < 10$ cm), and nil for pre-Quaternary deposits.

The last step involves combining the independent analytical results from each group together to produce a coherent, uniform, relative hazard susceptibility map. Results from each group fall within one of five susceptibility ratings for dynamic slope instability: very high, high, medium, low and nil.

4.4.3.7 Lee *et al.* (2000)

The ratio between critical acceleration and PGA is used to define the susceptibility of submarine slopes by Lee *et al.* (2000). The critical acceleration is defined by using a pseudo-static analysis of a planar failure mechanism. The model also incorporates the cyclic strength of marine deposits by using the Cyclic Stress Ratio (CSR) defined after 10 cycles, and as a function of water content and material bulk density. Slopes associated with lower a_c/PGA ratios are more susceptible to landsliding. These authors found that a value of a_c/PGA of about 0.15 appears to form a boundary separating generally stable from generally unstable slopes. This value theoretically should be 1.0, according to the critical acceleration definition, however, these authors stated that this discrepancy is due to the method ignores several factors that contributes to the fact the development of clearly defined shallow landslides.

CHAPTER 5

PUBLISHED STUDIES OF EARTHQUAKE-INDUCED LANDSLIDES

Stories of landslides caused by earthquakes on natural slopes have been reported world-wide since remote times. One of the earliest landslides recorded occurred in 1767 B.C in China, which was recorded, according to Feng and Guo (1985, 1986), in "Chronicle on Bomboo Slip", and it is said that this slide blocked up the Yi and Luo rivers. In spite of these remote evidences, ancient recorded cases are poorly described.

Although many cases of landslides due to particular earthquakes can be found in the geotechnical and seismological literature, few attempts have been made to compile this information and apply it to hazard analyses.

The most earliest study of world-wide earthquake-induced landslides, which was consulted during the present project, was that written by Montessus de Ballore in "La Géologie Seismologique", published in 1924. Through the whole book historical cases of earthquake-induced landslides were reported and detailed descriptions of geological effects of many earthquakes were compiled. Moreover, data are presented in different chapters which differentiated effects due to different earthquake mechanisms and within different environments. All the data included in this book are now part of the present data set.

Some of the most notorious historical cases of earthquake-induced landslides due to liquefaction of soil deposits were compiled by Seed (1968). In this paper Seed also explains theoretically the relation between landslide mechanisms and strong-motion characteristics. Subsequently, some regional and local studies have been presented (Feng and Guo, 1985; Tanaka, 1985; Mora, 1997b).

However none of these studies have attempt to defined general conclusions on factors influencing earthquake-induced landslides from case histories analysis.

World-wide case histories have been compiled by Keefer (1984a) in order to establish general relationships between landsliding extension and seismic parameters as a preliminary tool for hazard zonation. An extension of these data and relationships was presented by Rodríguez *et al.* (1999) from partial results of the present study.

Other regional attempts to relate landslides and earthquakes were found among others those by Maugeri *et al.* (1994) for China and Italy, Everard and Savigny (1994) for Canada, Guadagno and Mele (1995) for the Isle of Ischia in Italy. In the following sections a brief summary of these studies is presented as starting point for the data set compiled in this study.

5.1 Regional studies

5.1.1 Earthquake-induced landslides in China

A summary of some historical earthquakes that have caused landslides in China was presented by Feng and Guo (1985, 1986). Characteristics of the landslides discriminated by the earthquake magnitude were also presented. These authors reported that as many as 656 $M > 6.0$ earthquakes can be counted in China during the 780 B.C-1976 A.D period, and a third of these events were accompanied by landsliding. These data only include shocks on the mainland of China due to the lack of information for marine areas. In Table 5.1 the principal earthquakes triggering landslides reported in this study are compiled.

Maugeri *et al.* (1994) presented some relationships between landslide extension and seismic parameters, specifically magnitude and macroseismic intensity. These relationships were discussed in the previous chapter. A brief description of landslide mechanisms, areal distribution and morphometric characteristics of landslides induced by the 1974 Zhaotong (Yunnan Province) and the 1976 Sunpan (Sichuan province) earthquakes was also presented.

Table 5.1. Earthquake triggering landslides in China (Based on Feng and Guo, 1985, 1986)

DATE	EARTHQUAKE	M	LANDSLIDES CHARACTERISTICS	LANDSLIDES EFFECTS
1767 B.C.				Landslides blocked the Yi and Luo rivers
780 B.C.	Qishan County Shaanxi Province	6.7	Landsliding in Qishan mountains and valley of Jin, Wei and Luo rivers. Flows along river channels	Jin, Wei and Luo rivers were dammed
17-Sept-1303	Hongdong County Shanxi province	8.0	A landslide in Xumbao mountain. This slide was reported to have a length of 1600 m, 1400 m wide, and composed of loess of quaternary lying over a gravel deposit of 2 m thick. Landslide mechanisms was liquefaction due to high water level	This landslide buried a village killing all its habitants
21-July-1654	Gansu Tianshui	8.0	Earthquake covered more than 1000 km ² and triggered 59 landslides. Most of these landslides were concentrated inside an area of about 158 km ² , of which 25% was affected by landslides. Biggest landslide was the Luojiabao with 45 km long, 2 km wide	Chouniho river was dammed by the East Mount landslide
19-June-1718	Tanguei County Gansu province	7.5	572 km ² were covered by landslides one of them was 665 km ² in area, and has 40 km long, and 18 km wide. Total number of landslides was 337, among them the Yangning landslide with 8 km long, 3 km wide, 17 km ² of area, and travelled 5 km	The Yangning landslide buried the Yangning town killing 2000 people
10-Oct-1786	Kangding-Louding Sichuan province	7.5	Caused many landslips including the Mamianshan, which dammed the Dabu river	Natural dam was breached 10 days after, and subsequent flows washed out the villages at both sides of the river. Waves were reported up to 10 m high. Flow run out about 1400 km. Casualties due to landslides >100000 compared to 400-500 due to the shock
-1879	Targe earthquake Wudu, Gansu	8.0	Area covered by landsliding is approximately 400 km long and 200 km wide, with a major concentration in an area of 48 km long and 24 km wide. Average landslides length was 1 km	Rivers were dammed
16-Dec-1920	Haiyuan	8.5	Promoted 657 landslides, their bodies rushed down swiftly like waterfalls with some sliding and moving a few km, some flying over valleys and pushed against the waist of the opposite hill.	Many rivers were dammed. A landslide in Xijitan buried Ma Yuanhong killing 660 people. Total casualties reached 240000 and many of them were due to landslides
-1920	Ningxia	8.5	Area covered by landsliding was approximately 50000 km ² . Total number of slides was 675. Lengths were a few hundred metres, and average height was 150 m	The Dangjiaca landslide dammed a river forming a lake of about 5 km long and 380 m wide

Table 5.1 (continued)

DATE	EARTHQUAKE	M	LANDSLIDES CHARACTERISTICS	LANDSLIDES EFFECTS
23-Aug-1933	Diexi	7.5	Many landslides were triggered	Taidi and Jiaochangoo landslides dammed the Minjiang river creating four large lakes along the river. 6800 people died among of them 577 were buried in a small town. Total casualties due to landslides were 2500
12-Aug-1952	Nagu Sang Xong, Xizang	7.5	Extensive landsliding within regions with intensity VIII (scale ?)	
30-Sept-1952	Mianning, Sichan	6.8	A landslide in the epicentral area had 2.5 km long. Minimum intensity IV for triggered landslides	
14-April-1955	Kangding Sichuan	7.5	30 landslides	
14-April-1961	Jiashi Xinjiang	6.8	Serious landsliding within the epicentral area	
5-Jan-1970	Tonghai Yunnan	7.7	Over 30 landslides in the epicentral area	Rivers were dammed
6-Feb-1973	Luhuo Sichuan	7.9	137 landslips in the epicentral area of intensity IX	
14-May-1974	Doguan Yunnan	7.1	Widespread landsliding in the epicentral area, the largest landslide has 200 m wide, 300 m long and formed a dam about 30 m high. Landslides found in areas with intensities VI, VII, and VIII	

Landslides in China were identified as precursor events of large earthquakes by Zengjian and Baoyan (1986). This relationship of landslides and earthquakes was obtained from study of historical cases of landslides during the 1920 Haiyuan, the 1927 Gulang, and the 1961 Mingxian earthquakes. These authors suggested that before large earthquake, due to landslides and cracking in the mountains sounds can be heard and earthquakes can be predicted, and disasters due to landslides can be prevented.

5.1.2 Earthquake-Induced landslides in Japan

A compilation of landsliding events due to Japanese earthquakes was presented by Tanaka (1985). These cases occurred during the period 1949-1984 and are summarised in Table 5.2. Many other

cases in Japan have been reported for individual earthquakes, which are part of the database in the present study.

Table 5.2. Earthquake-induced landslides in Japan. 1949-1984 period (Based on Tanaka, 1985).

DATE	EARTHQUAKE	MAGNITUDE	LANDSLIDES DESCRIPTION
26-Dec-1949	Imaichi Tochigi prefecture	6.4, 6.7	Main type of slope failures was rock falls along steep slopes or cliffs and several soil falls occurred in pyroclastic granite areas. Slides occurred in Pleistocene clayey beds originated by rock weathering
21-Feb-1968	Ebino Miyazaki prefecture	5.7	The main types of landslides were soil falls and soil slumps. Soil is a kind of pumice layer produced by volcanic rock weathering. Distribution of landslides relates well to lacustrine deposits
16-May-1968	Tokaichi-oki	7.9	Many landslides occurred around Gonobe in Aomori prefecture. The majority of slope failures were rock slides whose rupture surfaces were situated in a pumice layer (Hachinohe) About 200 mm in antecedent precipitation was recorded for three days
9-May-1974	Izuhanto-oki Izu peninsula	6.9	Main types of slope failures were rock falls and rock slumps. Largest landslide was the Nakagi landslide, which was in white altered tuff
21-April-1975	Oitaken-chubu	6.4	Slope failures occurred in areas of low consolidated lacustrine diatomaceous earth, pyroclastic rocks, and lava. Types included rock falls, rock slides and rock slumps
14-Jan-1978	Izu Oshima-kinkai	7.0	The main area damaged was Kawazu-cho, Izu peninsula, which consists of pyroclastic rocks from the Amagi volcano in the Quaternary period. Rock falls, soil falls, and rock slides were the main types of landslides
14-Sept-1984	Nagahoken-seibu	6.8	Earthquake was near the Ontake Mountain. Slope failures can be divided into groups. The first group was controlled by geological structures (especially pumice layers), and the second one was caused by topographical factors meaning steep slopes and cliffs. The result of the first group were rock slides and the second were rock falls. Geology of Ontake-kogen, where rock slides took place, was an alternation of pumice layers and volcanic ash layers. Rupture surfaces were situated in the lower pumice layer. There had been 165 mm of antecedent precipitation for 5 days before the earthquake

A comparative study of seismically-induced landslides during the 1978 Izuoshima and 1984 Nagano-ken earthquakes was presented by Kobayashi (1997). Kobayashi stated that 30% of deaths produced by these earthquakes were produced by landslides, pointing out the importance of earthquake-induced landslide hazard analysis. A description of the areal distribution of landslides is presented and their relationship with topographic, climatic and seismic conditions is analysed. For

the 1984 Nagano-ken earthquake, statistics of data from the landslide inventory are shown and reproduced in Figure 5.1. Severity of damage due to landslides during both earthquakes was compared based on volume-frequency plots shown in Figure 5.2. Based on these graphs, the author concluded that the 1978 Izuoshima earthquake was more severe than the 1984 Nagano-ken earthquake in terms of landslides.

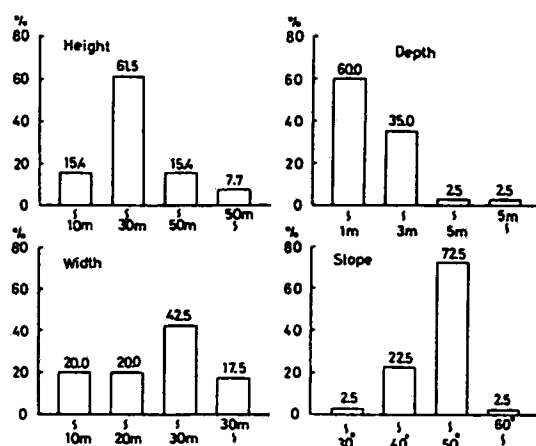


Figure 5.1 Characteristics of landslides during the 1984 Nagano-ken earthquake in Japan (Kobayashi, 1997).

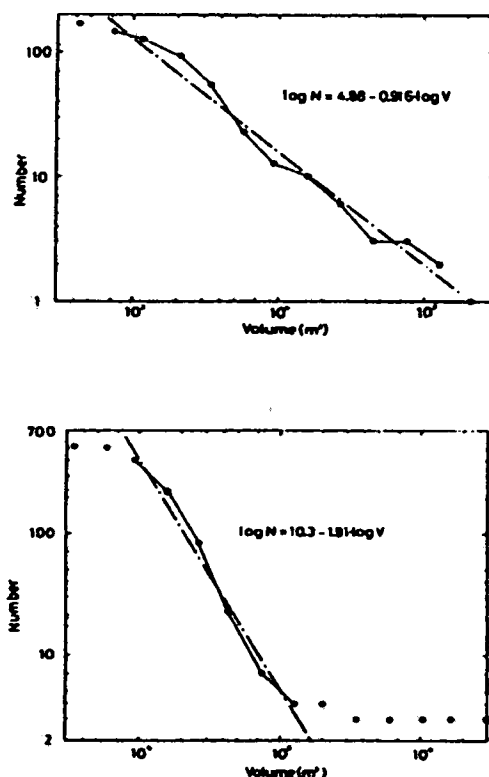


Figure 5.2 Frequency of landslide volumes. Upper graph for the Nashimoto area during the 1978 Izuoshima earthquake. Lower graph for Otaki area during the 1984 Nagano earthquake. (Kobayashi, 1997)

Landslides produced by the 1978 and 1984 earthquakes were also studied by Ishihara and Hsu (1986). In this case pseudo-static analysis of planar failure were carried out for some individual landslides, in order to determine their critical acceleration and static factor of safety based on topography and material properties. The dynamic effect on shear strength is considered by increasing the cohesion component.

The minimum acceleration for each landslide to be triggered was related to the static factor of safety for different slope angle ranges as shown in Figure 5.3. As the dynamic shear strength is increased compared to the static strength, the lines defined in this figure lie well above those defined by other relations such as that proposed by Newmark, and also above the relationship defined for actual data obtained from back analysis (Charalambos, 1999).

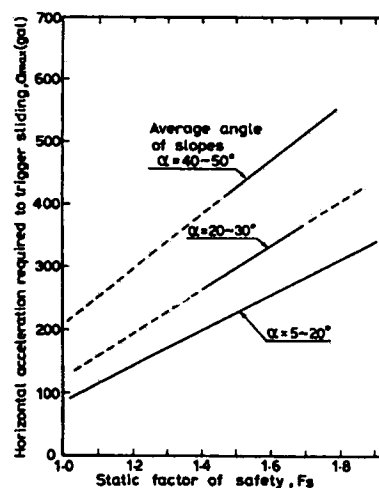


Figure 5.3 Critical acceleration-static factor of safety relationship as a function of slope angle (Ishihara and Hsu, 1986)

5.1.3 Earthquake-induced landslides in Costa Rica

Landslides during earthquakes have been an important source of damage in Costa Rica. Mora (1997b) showed that most severe Costa Rican earthquakes have triggered landslides since a long time ago. A summary of the major events is compiled in Table 5.3, which includes events during the period 1888 to 1993. In this region, landslides have occurred for earthquakes with magnitude >5.2 ,

and extension and distribution of landslides have been shown to be dependent on the climatic conditions. On the other hand, Mora has found a very close relationship between slope susceptibility and topographic roughness, which was defined by the relative relief parameter included in the table.

Table 5.3. Earthquake-induced landslides in Costa Rica. (After Mora, 1997b)

DATE	EARTHQUAKE	M_s	MMI	AREA AFFECTED (km ²)		MAXIMUM EPICENTRAL DISTANCE (km)		MINIMUM RELATIVE RELIEF (m/km ²)	
				>60%	<15%	>60%	<15%	>60%	<15%
30-Dec-1888	Fraijanes	5.9	VIII-IX	15	7	21	450	350	-
10-Oct-1911	Guatua	5.7	VIII+	6	90	4	12	475	375
29-Aug-1911	Bajos del Toro	5.5	VII+	4	15	3	15	485	350
6-June-1912	Sarchí	5.5	VII	4	70	3	9	515	375
4-Mar-1924	Orotina	7.0	IX-X	56	900	21	50	425	275
30-Dec-1952	Patillos	5.8	VIII+	9	130	6	18	450	375
1-Sept-1955	Bajos del Toro	5.5	VII+	2	50	2	8	500	400
14-Apr-1973	Tilarán	6.5	IX	18	750	19	40	475	225
3-July-1983	División-Buvia	6.2	VIII-IX	29	550	11	35	500	225
May-Dec-1990	Puriscal Swarm	Max =5.8	VIII	9	390	5	25	500	300
22-Apr-1991	Valle de la Estrella	7.45	VIII-IX	75	1525	40	80	500	300
July-1993	Pejibave Swarm	Max =5.2	VI+	-	20	-	10	-	400

5.1.4 Earthquake-induced landslides in New Zealand

Due to the tectonic setting and seismic activity in New Zealand, this area is very prone to landslides triggered by earthquakes. Adams (1981 a, b) stated that due to the terrain, most of the earthquake-induced landslides are rock avalanches and rock falls. Landslides are shallow features that fall rapidly from the ridge crests into the valley and pile up against the far valley walls.

Landslides triggered by earthquakes in New Zealand have been studied by using natural lakes as paleoseismic evidence. Some studies have tried to relate natural lakes to historical earthquakes

(Adams, 1981b; Perrin and Hancox, 1991), and a summary of these events is presented in Table 5.4.

Table 5.4 Earthquake-induced landslides in New Zealand (Based on Adams, 1981; Perrin and Hancox, 1991)

DATE	EARTHQUAKE	MAGNITUDE (M_s)	LANDSLIDES DESCRIPTION	
			LAKES	LANDSLIDE VOLUME ($\times 10^6 \text{ m}^3$)
1855	West Wairarapa	8.2	Bruce	10
1929	Arthur's Pass and Murchinson	7.1/7.8	Burgoo	0.15
			Earthquake lakes	1.0
			Ellis	5.0
			Elmer	4.0
			Falls	15.0
			Glasseye	3.5
			Lower Lindsay	2.5
			Marina	22.0
			Moonstone	0.3
			Mud	15.0
			Perrine	0.5
			Sandra	4.0
			Sandstone	1.6
			Six Mile	0.1
			Stanley	40.0
1968	Inangahua	7.4	Thompson	16.0
			Upper Matiri	0.2
			Buller	4.3
			Downey	0.6
			Ram Creek	1.5
			Waitahn	0.8

Epicentres and earthquake magnitude of three different pre-historical events in Taranaki (North Island) were determined from landslide distribution by Crozier (1991). Landslides were dated and three different events were inferred, for which seismic parameters were defined by using Keefer's (1984a) relationships for area affected and maximum distance to epicentre. Results of this study are summarised in Table 5.5, whereas location and area affected by landslides for each case are presented in Figure 5.4.

Rock fall deposits were dated and correlated to earthquakes in the Southern Alps using lichenometry by Bull and Brandon (1998). The epicentral area of historical earthquakes may be defined by growing state and number of lichen on rock blocks. This method was calibrated by knowing rock fall deposits of well dated events, including some due to earthquakes, which are compiled in Table 5.6. Rock falls in 1833, 1836 and 1841 were dated by this method but no

earthquakes occurred on those dates, hence landslides, origin is assumed to be different from seismic shaking.

Table 5.5 Landsliding characteristics due to pre-historic earthquakes in Taranaki, North Island, New Zealand. (Adapted from Crozier, 1990)

EVENT	DATE	AREA AFFECTED BY LANDSLIDES (km ²)	MAXIMUM EPICENTRAL DISTANCE (km)	M _L
A	31000 yr BP	1015	23	6.5-7.4
B	12000-1400 yr BP	825	20	6.4-7.3
C	1400 yr BP	1855	28	6.8-7.5

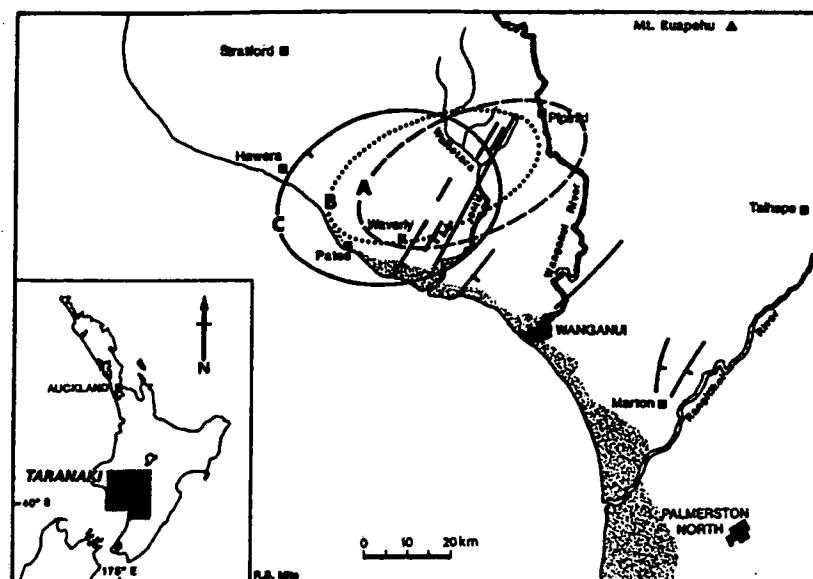


Figure 5.4 Areas affected by landslides during pre-historical earthquakes in Taranaki, North Island, New Zealand (Crozier, 1991).

Table 5.6 Rock falls deposits due to earthquakes in Southern Alps, New Zealand, used for lichenometry calibration by Bull and Brandon (1998).

Earthquake	Date	M _s	Description
West Wairarapa	1855	8.2	Large concentration of rock fall deposits around epicentral area
Hurunui	1881	>6.5	Mass movement deposits near earthquake epicentre. Rock fall abundance around epicentral area
Murchinson (Buller)	1929	7.8	Mass movement deposits near earthquake epicentre. Not witnessed but photographs were taken after the earthquake of the Rough Creek landslides and the 60x10 ⁶ m ³ Falling Mountain rock avalanche
Seaward Kaikoura	1955	5.1	Mass movement deposits near earthquake epicentre. Not witnessed but local residents described coseismic landslides.

5.1.5 Earthquake-induced landslides in Italy

Murphy (1993, 1995) showed a brief description of several earthquakes throughout the history of Italy known to have caused landslides, summarised in Table 5.7. The Calabrian earthquake of 1783 resulted in slope failures over a large area, and two phenomena seem to be related with this considerable extension: the antecedent rainfall and the widespread seismic aftershocks that occurred after the earthquake. Landslides triggered by the 1908 earthquake in the Strait of Messina occurred in three main groups. The first were lateral spreads due to liquefaction, the second group occurred mainly in clays and clay marls, and the final group occurred in weathered crystalline gneisses. In the 1976 Friuli earthquake, landslides were mainly rock falls and rockslides. In the 1980 Irpinia earthquake most landslides were mudflows, and the delay in landslides after the earthquake seems to be related to the regional groundwater regime change.

Table 5.7 Earthquake-induced landslides in Italy (From Murphy, 1993, 1995)

DATE	EARTHQUAKE	MAGNITUDE M _L	MAXIMUM INTENSITY (MCS)	MAXIMUM DISTANCE TO LANDSLIDES (km)
1693	Val di Noto	6.9	XI	25
1783	Calabria	6.6	XI	40
1790	Nascemi	4.0	VII	20
1849	Siracusa	5.6	IX	100
1905	Pizzo Calabro	7.0	X	34
1907	Ferruzzano	5.9	IX	16
1908	Strits of Messina	7.1	XI	130
1976	Friuli	6.1	IX	50
1976	Friuli	6.0	IX	-
1978	Ferruzzano	5.5	VIII	11
1980	Irpinia	6.9	X	200

On the volcanic island of Ischia, geomorphological investigations, as well as historical and archeological evidence, suggest the occurrence of landslides due to earthquakes. Guadagno and Mele (1995) presented a list of earthquakes accompanied by landslides; this list is shown in Table

5.8. The analysis presented by these authors is of particular interest for studies on the seismic behaviour of volcanic deposits; landslides distribution due to these events are presented in Figure 5.5.



Figure 5.5 Landslides of the North-West sector of the Island of Ischia. Legend: 1) Debris slide and year, 2) Debris flow, 3) Fall scarps (1881, 1863, 1828, 1797); 4) Area affected by surficial landsliding in the earthquakes of 1881 and 1828, 5) Supposed rear scarp for the 4th century collapse. (Guadagno and Mele, 1995)

Landslide extension and mechanisms for the 1976 Friuli and 1980 Irpinia earthquakes were described by Maugeri *et al.* (1994). Individual landslides were also studied for the Irpinia earthquake. These data were complemented with Chinese cases to propose the hazard analysis model that was explained in Chapter 4.

Erosion rates due to earthquake-induced landslides may be determined by using the Keefer (1994) relationship. For application of this relation in Italy, Parise (2000) obtained seismic moments from macroseismic magnitudes for historical earthquakes reported to have caused landslides. Table 5.9 shows those cases compiled for this purpose.

Table 5.8 Earthquakes inducing landslides at Camicciola in Ischia Island (Adapted from Guadagno and Mele, 1995)

Year	1228	1828	1863	1881	1883
Maximum MMI	IX-X	VIII-IX	VII	IX	X

Table 5.9 Earthquakes inducing landslides in Italy (Adapted from Parise, 2000)

Date	I ₀ MCS	Epicentral zone	M ₀
1125-10-11	VIII	Sannio-Molise	6.9
1456-12-05	XI	Central-southern Italy	7.1
1561-07-31	VII	Buccino	6.0
1561-08-19	IX-X	Vallo di Diano	6.4
1688-06-05	XI	Sannio	6.6
1694-09-08	X-XI	Irpinia-Basilicata	6.8
1805-07-26	X	Molise	6.5
1851-08-14	X	Basilicata	6.3
1853-04-09	IX	Irpinia	5.9
1857-12-16	XI	Basilicata	6.9
1910-06-07	VIII	Irpinia-Basilicata	5.8
1930-07-23	X	Irpinia	6.7
1980-11-23	X	Irpinia-Basilicata	6.8

5.1.6 Earthquake-induced landslides in Greece

A large number of strong earthquakes have been reported to cause landslides and other ground failures in Greece, as shown in Table 5.10. Some of these cases were presented by Papadopoulos and Plessa (1999), who also showed relationships between maximum epicentral distances and magnitudes. In general, the Greek distance-magnitude relationship was found to be similar to that reported by Keefer (1984a). However, by considering a small aftershock ($M_S = 3.8$) of the 1993 Pyrgos earthquake that triggered a landslide, the relationship becomes quite different. Data presented by Papadopoulos and Plessa, and Keefer upper bounds, are shown in Figure 5.6. A similar analysis was carried out by Papadopoulos *et al.* (1993) for liquefaction.

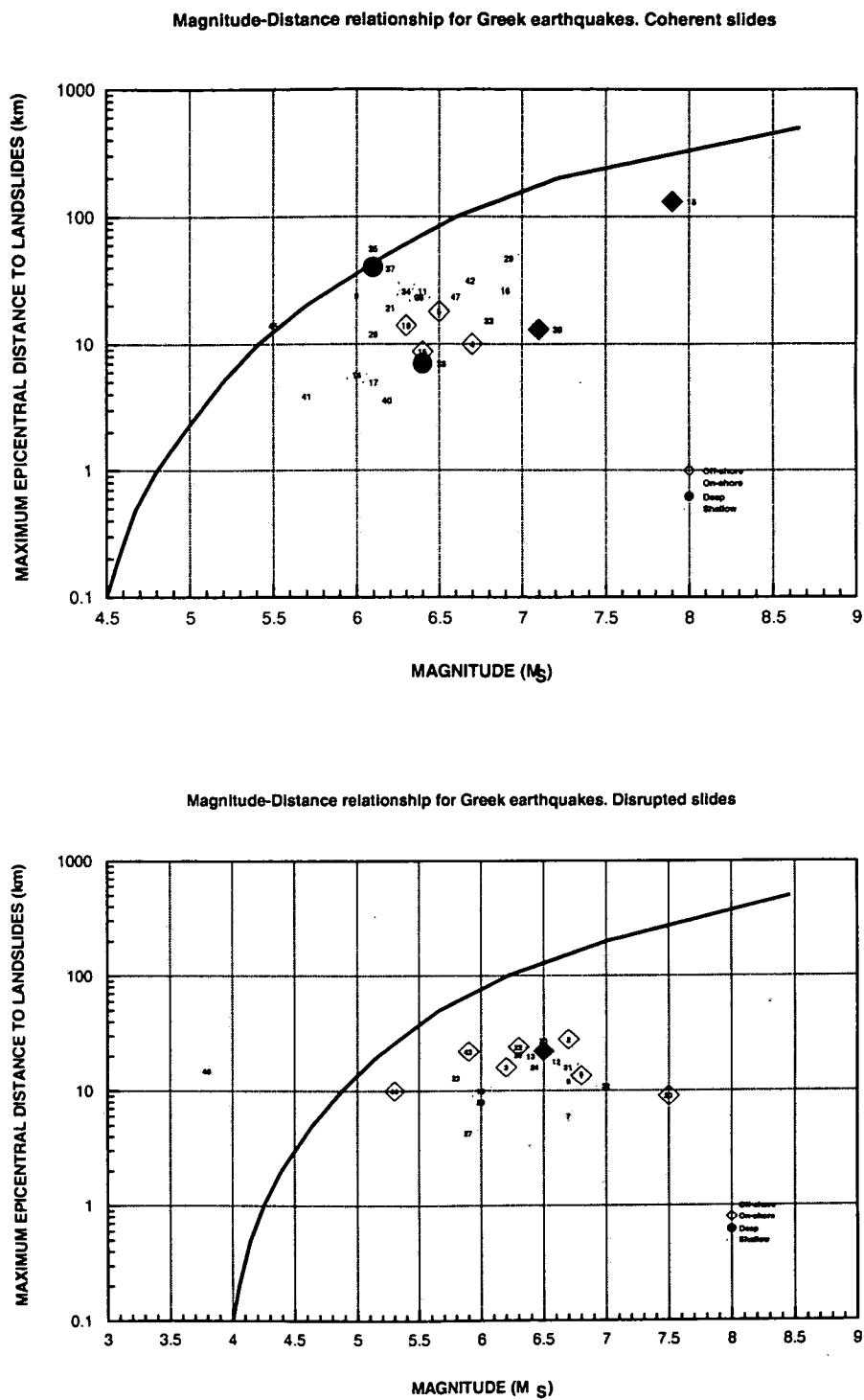


Figure 5.6 Magnitude-epicentral distance relationship for coherent (upper) and disrupted (lower) landslides in Greece. Numbers referred to Table 5.10. Solid lines are the Keefer (1984a) envelopes.

Table 5.10 Earthquake-induced landslides in Greece (After Papadopoulos and Plessa, 1999)

	DATE	EARTHQUAKE	MAGNITUDE (M _s)	MAXIMUM EPICENTRAL DISTANCE (km)	TYPE OF FAILURE
1	09-Oct-1650	Santorini Island	6.8	13.5	Rock fall
2	18-June-1798	Palaeopolis, Kythira Island	6.7	28	Rock fall
3	20-Mar-1837	Hydra Island	6.2	16	Rock fall
4	30-Oct-1840	Zakynthos Island	6.7	10	Rock slide
5	18-Oct-1843	Chalki Island	6.5	18	Rock slide
6	11-Oct-1845	Lisvori, Lesvos Island	6.8	14	Rock fall
7	21-Feb-1858	Corinthos	6.7	6	Rock fall
8	1-Aug-1870	Arachova, Fokis	6.7	15	Rock fall
9	18-Mar-1874	Eretria	6.0	24	Landslide
10	26-June-1876	Nemea	6.0	10	Rock fall
11	3-Apr-1881	Chios Island	6.4	26	Landslide
12	25-Oct-1889	Hidra, Lesvos Island	6.7	18	Rock fall
13	17-Apr-1893	Keri, Zakynthos Island	6.4	20	Rock fall
14	23-May-1893	Thiva	6.0	5.7	Landslide
15	20-Apr-1894	Malesina, Martino	6.4	8.7	Landslide
16	27-Apr-1894	Atalanti	6.9	26.3	Landslide
17	22-Jan-1899	Kiparissia	6.1	5	Landslide
18	11-Aug-1903	Milata, Kythira Island	7.9	131	Landslide
19	20-Jan-1905	Skite, Magnesia	6.3	14	Landslide
20	8-Nov-1905	Athos, Chalkidiki	7.5	9	Rock fall
21	30-May-1909	Fokis	6.2	27	Landslide
22	27-Nov-1914	Lefkada Island	6.3	24	Rock fall
23	4-June-1915	Granitsa, Agrafta	5.8	13	Rock fall
24	1-July-1927	Laconia	6.43	16	Rock fall
25	22-Apr-1928	Corinthos	6.3	20.7	Rock fall
26	31-Mar-1930	Puri, Magnesia	6.1	12	Slide
27	17-Apr-1930	Sophiko, Corinthia	5.9	4.3	Rock fall
28	20-July-1938	Oropos	6.0	8	Rock fall
29	6-Oct-1947	Messinia	6.92	47.3	Landslide
30	22-Apr-1948	Vasiliki, Lefkada Island	6.5	22	Rock fall
31	23-July-1949	Kardamyla, Chios Island	6.7	16	Rock fall
32	30-Apr-1954	Sophades	7.0	11	Rock fall
33	8-Mar-1957	Velestino	6.8	15.3	Landslide
34	14-May-1959	Pitsidia, Crete Island	6.3	26	Rock slide
35	5-Apr-1965	Megalopolis	6.1	40	Landslide
36	6-July-1965	Eratini	6.38	23.3	Landslide
37	5-Feb-1966	Kremasta	6.2	28	Landslide
38	1-May-1967	Drosopighi, Arta	6.4	7	Landslide
39	19-Feb-1968	Ag. Efstratios Island	7.1	13	Landslide
40	8-Apr-1970	Antikyra, Boeotia	6.18	3.6	Rock slide
41	31-Dec-1975	Kato Makrinou, Trichonis	5.7	3.9	Landslide
42	24-Feb-1981	Aikyionides	6.69	31.6	Landslide
43	16-Oct-1988	Vartholomio	5.9	22	Rock fall
44	20-Mar-1992	Trioavasalos, Milos Island	5.3	10	Rock fall
45	26-Mar-1993	Pyrgos	5.5	14	Landslide
46	27-Mar-1993	Pyrgos, aftershock	3.8	15	Soil fall
47	13-May-1995	Kozani-Gravena	6.6	23.6	Landslide

5.1.7 Earthquake-induced landslides in the former Yugoslavia

Earthquakes in the whole territory of former Yugoslavia have caused many landslides, which were classified by Sunarić and Nedeljković (1994) into two main groups: gravitational slides and lateral propagation. Gravitational slides have been formed at steep slopes of solid sedimentary rocks and magmatic rocks and usually include superficial residual deposits from the weathering of these materials. Lateral propagation has been reported in incoherent clay sediments, particularly when the groundwater table is high. Schemes of slope failures common in this area are shown in Figure 5.7, whereas Figure 5.8 shows typical deposits susceptible to slumps.

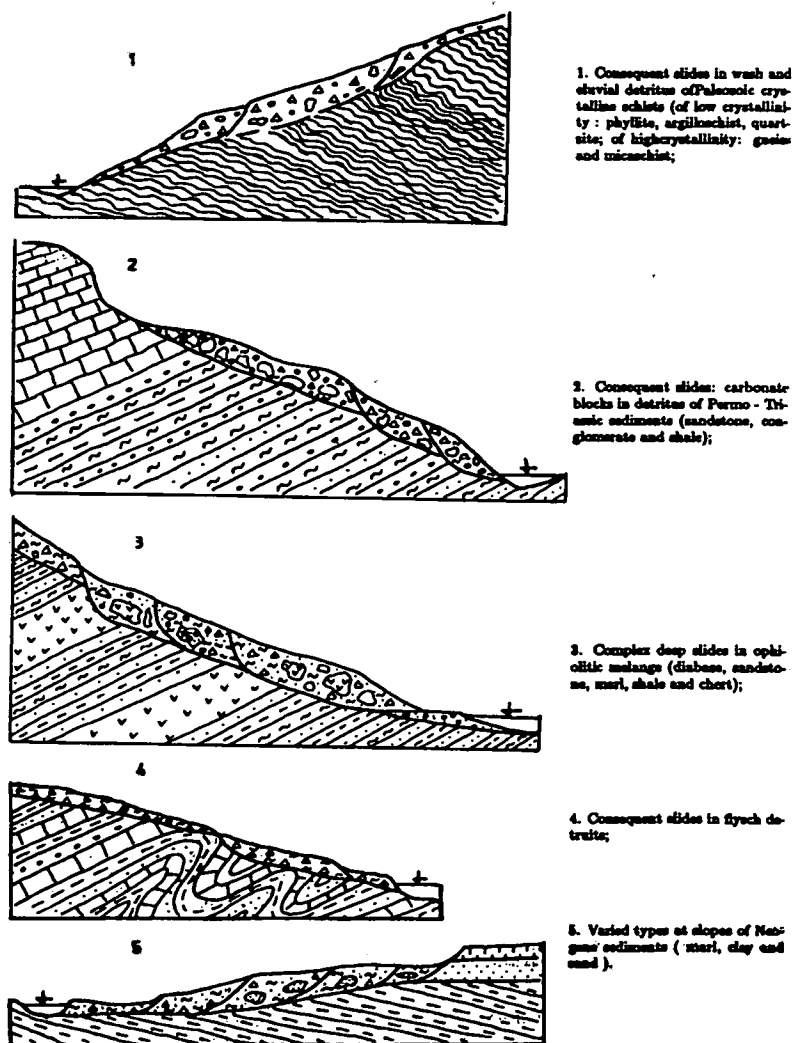


Figure 5.7 Common types of earthquake-induced landslides in former Yugoslavia (Sunarić and Nedeljković, 1994).

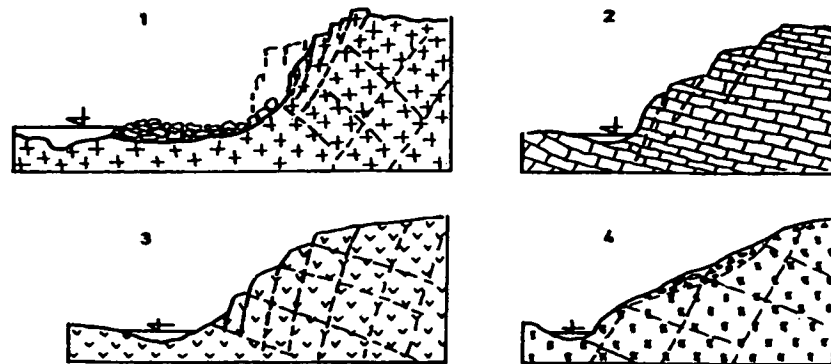


Figure 5.8 Susceptible lithologies to earthquake-induced landslides in former Yugoslavia: 1) Calcareous-dolomitic, karstified rocks; 2) thin- to thick-bedded fractured and karstified limestones; 3) volcanites and granitoids; and 4) ultramafites and basites (Sunarić and Nedeljković, 1994).

5.1.8 Earthquake-induced landslides in the former USSR

Landslides due to earthquakes have been related to three main processes according to Solonenko (1977), who studied historical cases in the former USSR. These processes are the seismic accelerations, change of the gradient angle of critical planes, and the thixotropy effects on rocks.

Based on pseudo-static analysis, Solonenko established that landslides can be triggered by accelerations related to MSK intensities between VI and V. However, from long observations of landslides in the Crimean area, it was found that even earthquakes with intensities of MSK IV cause activation of landslides in a period of strong moistening, whereas in a period of insufficient moistening they may not be displaced even in areas with MSK intensities of VIII.

Seismotectonic deformations may change the gradient angle of slopes in weak zones, especially within the epicentral area. From pseudo-static analyses it can be determined that the change of the gradient angle by tens of seconds may cause the disturbance of equilibrium and the movement of potential unstable and stable slopes. Solonenko reports cases due to this effects during the 1957 Muya earthquake ($M=7.9$) and the 1957 Gobi Altai earthquake ($M=8.6$).

Thixotropy effects on clayey rocks reduce their supporting capacity to zero, at the same time as causing a great dynamic effect in the overlying, often strong, sediments which are broken up and

displaced. This phenomenon was observed over extensive areas during earthquakes in Mongolia (1905 Hangai earthquake, $M=8.4$; 1957 Gobi Altai earthquake, $M=8.6$; 1967 Mogod earthquake, $M=7.7$).

Solonenko, based on observations in the former USSR, identified nine different types of landslides; characteristics of these mechanisms are presented in Table 5.11. It can be seen that in this case, seismotectonic movements are included into the classification.

Table 5.11 Earthquake-induced landslides in the former USSR (Based on Solonenko, 1977)

LANDSLIDE TYPE	CHARACTERISTICS	EXAMPLES
1. Gravitational seismotectonic wedges	At strong earthquakes ($M>8.0$) Subsidence of a wedge of the Earth's crust accompanied by slope collapses	Gobi-Altai earthquake (1957, $M=8.6$) Vertical displacements ≈ 328 m
2. Collapse along faults	During earthquakes separated sections of mountains subside, move towards valley and sometimes become true collapses	
3. Toppling of mountain peaks	Occurs during strong earthquakes with transitional return movement of the Earth's crust. Toppled peaks turn round the axis and small collapse occur	
4. Slipping landslides	In regions of moderately dislocated sedimentary or volcanic-sedimentary formations. If planes of bedding, schistosity or tectonic jointing are clearly defined slipping landslides may occur in crystalline rocks	Dagestan earthquake (1970, $M=6.6$), numerous slipping landslides were formed. The Achijsk landslide dammed the Chvakhun-bak valley
5. Seismo-vibrational landslides and collapses	Landslides are associated to continuous seismic vibrations of moderate force (MSK intensities VIII-X), movement develops towards the sea or valleys	Black Sea shore Mediterranean sea at Ionian Island (1953, M up to 7.0)
6. Seismotectonic landslides and collapses	Confined to the zones of seismo active fractures	Akiba slide (Greater Caucasus)
7. Seismogenic collapses with seismically vibrated collapsed masses	Two conditions are necessary for their formation: an immediate collapse and crushing of rock masses, and the effect of strong seismic vibrations on sliding, fine-to medium-blocked masses. These masses travel at a rapid rate, and cover a very big area	Khait earthquake (1949, $M=7.5$)
8. Seismo-gravitational collapses on the "air-cushion"	Mass is transported on an air-cushion at extremely high velocities	
9. Seismogenic ground avalanches and flows	Developed in regions with thick sediments, especially in loess areas. Individual ground avalanches may cross valleys and rise for hundreds of metres onto the opposite slope. At a reasonably powerful water stream, they may transform into mud flows in the valley. During convergence of the avalanches, by collision with other sliding masses, they get a powerful dynamic impulse and descent at a rapid rate	Khait earthquake

5.1.9 Earthquake-induced landslides in Canada

Neotectonics effects on landslide distribution was studied by Everard and Savagny (1994) for the Yukon area. Relationship between landslides lithology, and earthquake epicentres were defined from a landslide inventory within the area. Rocks having penetrative discontinuities suitably oriented to facilitate slide movement were found to be more susceptible to seismic failure because the critical displacement is minimal. However, it was recognised that this attribute is an important but not an exclusive requirement for slides be triggered. Landslides predominate on slopes that have been over-steepened by glacial scouring of fluvial down cutting. Within the area many large landslides are near major faults and fall within the area of higher seismicity between 1978 and 1990 as can be seen in Figure 5.9.

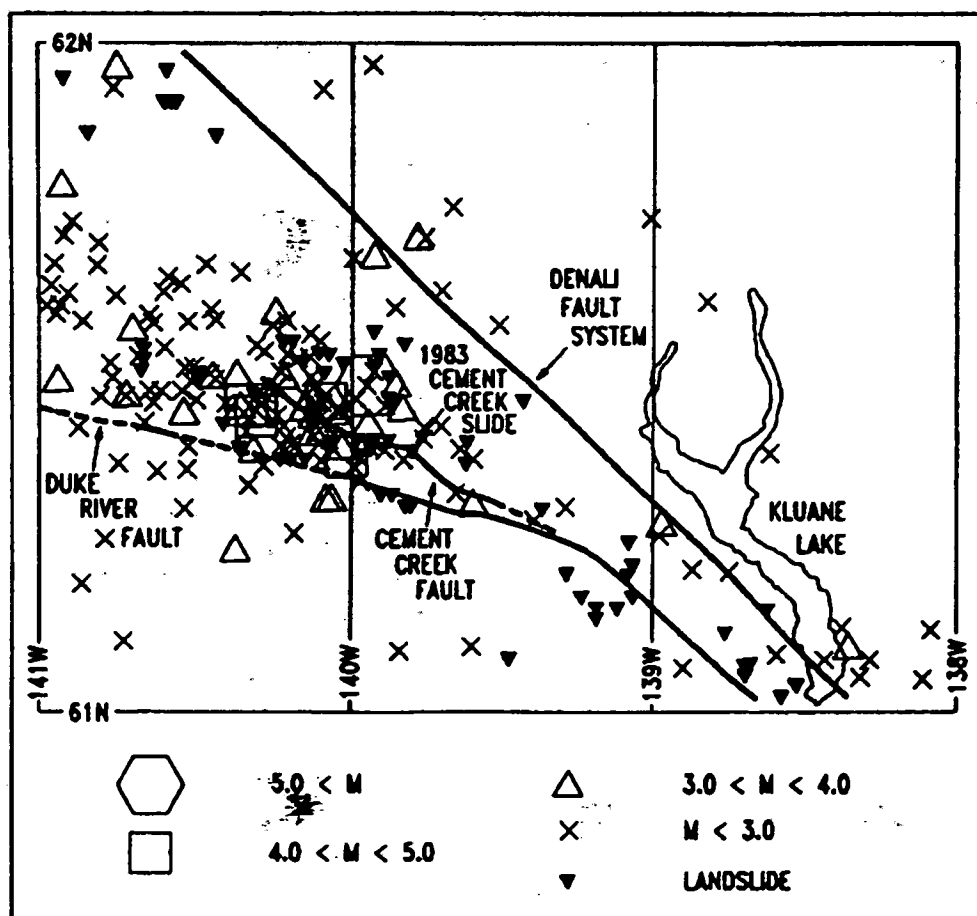


Figure 5.9 Earthquake and landslides location within Yukon area. (Everard and Savagny, 1994)

5.1.10 Earthquake-induced landslides In Papua New Guinea

Landslides within the Torricelli Mountain area have been found to be related to seismic activity (Simonett, 1967). In this area granitic deposits have experienced both single debris avalanches and complexes of deep and shallow avalanches coupled with extensive gullying of deeply weathered bedrock. In some areas where debris avalanches coalesced, giant mud and debris flows surged down the valleys. Major landslides have also been found in sedimentary rocks; in this case they are massive rotational slumps, except on steep scarp slopes where shallow debris avalanche movements also take place.

Comparison by using multivariate regression analysis between landslides within the seismic area of Torricelli Mountain with an aseismic area within Bewani Mountain reveals that both areas present completely different slope behaviours. For each zone individually, slope angle and landslides volume are seen to be related. In the seismic zone, distance to the source also was found to be related to landslide volume. Finally, Simonett suggests that the number of landslides is a function of macroseismic intensity, as the number of landslides decreases with distance from epicentre in a manner similar to intensity, hence a relationship between them was presumed.

5.1.11 Earthquake-induced landslides in El Salvador

Data compiled from literature review of earthquake-induced landslides were presented by Rymer and White (1989). Table 5.12 shows main characteristics of these earthquakes and Figure 5.10 is the distribution of areas affected by landslides due to these historical cases. These authors stated that the contour for Mercalli Modified Intensity VII is a minimum estimate of limit for earthquake-induced landslides; furthermore, they suggest that the contour of MMI VI best approximates the limits of landslides associated with an earthquake. No evidence that this relation should be generalised was found during the present study, and occurrence of landslides in lower levels of intensity depends strongly on slope susceptibility. Moreover, data compiled for the present study (Rodríguez, 2001) found that the Rymer and White's limits for the 1915 and 1982 earthquakes are smaller than the actual limits. Landslides due to earthquakes in El Salvador as shown by Rymer and

White seem to be concentrated along the volcanic chain zone, although some historical evidence of landslides, including the most recent earthquake in El Salvador on 13 January 2001, has shown that seismically-induced landslides may be widely spread along the country including the coastal provinces. Although major concentration of landslides has been found to correlate with volcanic tuff distribution, there are other susceptible lithologies such as residual, lacustrine, deltaic and coastal deposits, which are spread around the country.

Table 5.12 Historical earthquakes causing landslides in El Salvador (Adapted from Rymer and White, 1989)

Year	Magnitude	Landslides description
1857	6.25	Many landslides on hills and in canyons east of Lake Ilopango
1878	6.25-6.5	Landslides on slopes of volcanoes near Santiago de María, a large slide on Cerro El Tigre buried 14 people
1915	7.9	Many great slides throughout western El Salvador
1919	6.0	Many landslides on slopes of Cerro San Jacinto
1936	6.1	Many landslides on slopes of San Vicente Volcano
1947	7.2	Landslides on slopes of Conchagua volcano
1951	6.0 & 6.2	Landslides on volcanic slopes near Santiago de María
1965	6.0	Landslides from San Salvador to Lake Ilopango
1982	7.0	Many landslides southwest of San Salvador
1986	5.4	Hundreds of landslides between San Salvador and Lake Ilopango with about 200 fatalities and at least 100 homes destroyed

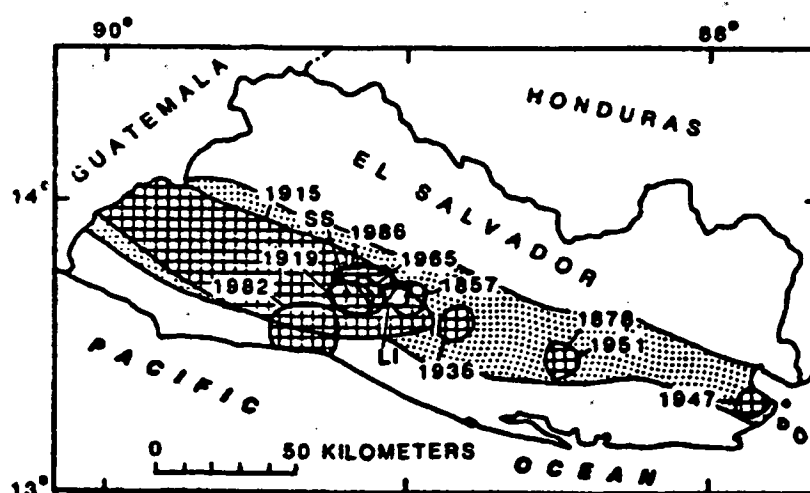


Figure 5.10 Areas affected by landslides in El Salvador due to historical earthquakes (Rymer and White, 1989)

5.1.12 Earthquake-induced landslides in Venezuela

The national inventory of geological hazard in Venezuela was presented by Lugo (1984) by three maps of phenomena distribution within eastern, central and western areas of Venezuela. These maps are reproduced in Figure 5.11. As shown most of the cases are related to seismic events. Geologic effects are shown to be concentrated along the main fault systems in Venezuela.

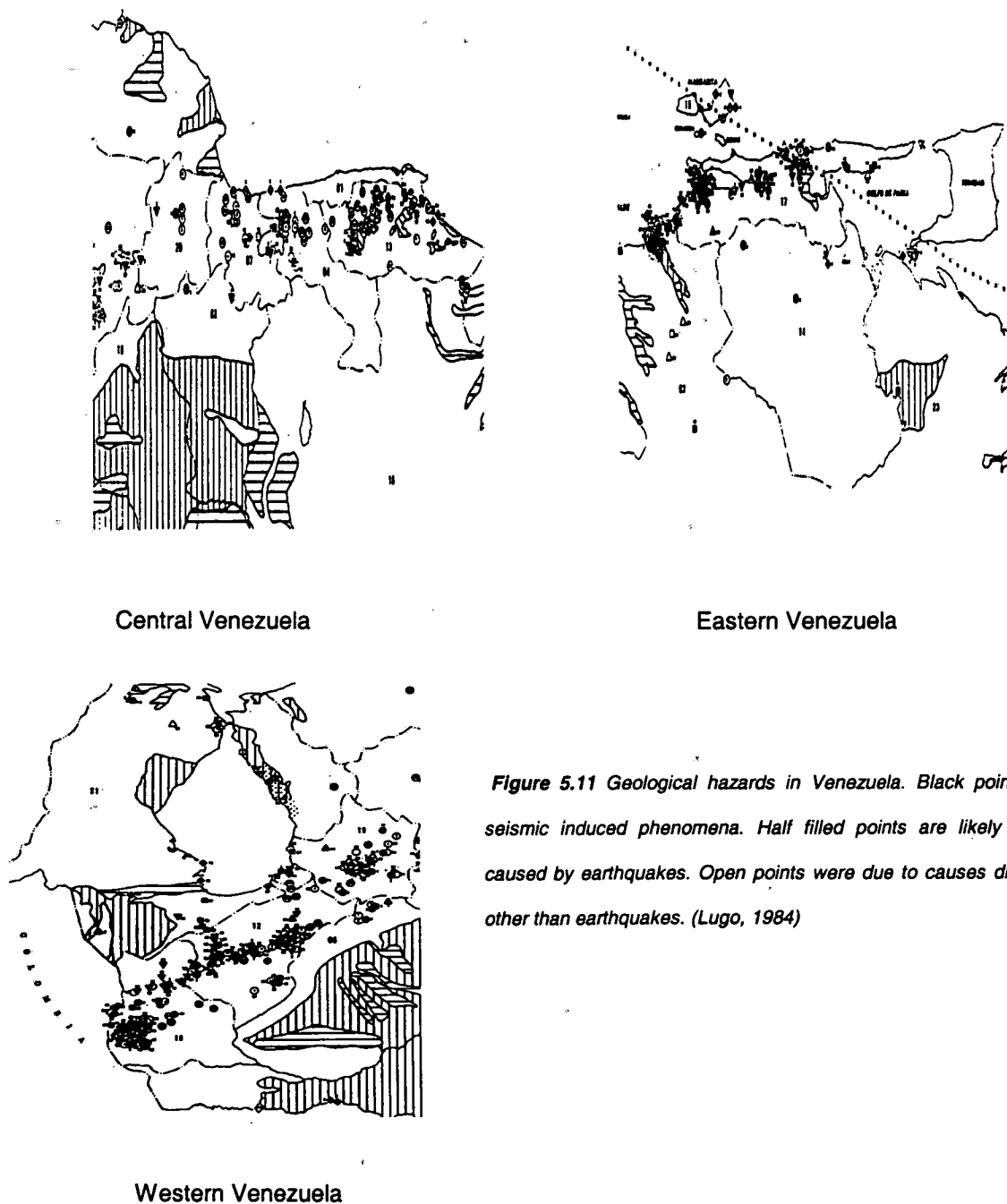


Figure 5.11 Geological hazards in Venezuela. Black points are seismic induced phenomena. Half filled points are likely to be caused by earthquakes. Open points were due to causes different other than earthquakes. (Lugo, 1984)

5.1.13 Earthquake-induced landslides in Colombia

A review of earthquakes inducing landslides in Colombia were compiled by García (1994), based mainly on Ramírez' (1975) descriptions of earthquake effects. Table 5.13 presents these cases.

5.1.14 Earthquake-induced landslides in the United States

Only one compilation of different cases of earthquakes inducing landslides in USA was found (Wilson and Keefer, 1985), which focused on Californian earthquakes. However, reconnaissance reports of individual earthquakes usually give a detail description of damage due to geological effects such as liquefaction and landslides.

Wilson and Keefer (1985) compiled some historical cases of earthquakes that induced landslides in California, which are summarised in Table 5.14. As these authors stated these cases were presented only to illustrate the potential damage that can be triggered by landslides.

5.2 Studies including world-wide cases

5.2.1 Redeposition of sediments due to earthquake-induced landslides

Historical examples of redeposited material yielded by earthquake effects have been documented for a long time and from different regions around the world. Table 5.15 summarises the characteristics of historical events that have produced redeposition of substantial volumes of material following an earthquake. Seguret *et al.* (1984) stated that most observed underwater failures originate in the epicentral region, except for the Chile, Orleansville and Aegean Sea earthquakes; although these authors stated that in the Chilean case, 100 km away from the M=8.2 earthquake epicentre may be still considered as the focal region. Finally, they stated that these cases seem to generally occur under a compressive tectonic regime.

Table 5.13 Earthquakes inducing landslides in Colombia (Adapted from García, 1994)

Date	Zone	M	Description
1530-09-01	Cumaná, N.E Colombia	-	Landslides, ground fissures, subsidence, liquefaction and water expulsion
1610-02-03	Border with Venezuela	-	Large landslide at the Bailadores Valley dammed the river.
1743-10-18	Fómeque	-	Many landslides damming creeks and blocking roads, destroying houses and crops.
1785-07-14	Central region	-	Landslides dammed the Main and Magdalena rivers. Landslides around Santafé.
1790-	Barcelona, San Pedro Alcántara	-	A large landslide produces a natural lake of 600 m in perimeter .
1805-06-16	Central region	-	Landslide in Bogotá dammed the Aguavieja creek, which was part of the water supply system.
1827-11-16	Central and southern regions	-	A landslide dammed an affluent of Cauca River near Popayán. Dam breaching produced severe damage. In Neiva many landslides were triggered. La Honda Creek was dammed many days afterwards dam breaching produced 161 deaths. Large landslides on both sides of Suaza River in Timaná, forming a lake that failed a month later producing additional deaths and floods along the Magdalena River. Many landslides in other zones were also reported.
1834-01-20	Nariño	-	Sibundoy was completely destroyed by a large landslide
1875-05-18	Cúcuta	-	Cucutá and Villa del Rosario destroyed. Many landslides around the region.
1906-01-31	Tumaco	8.9	Liquefaction and landslides reported.
1936-01-09	Nariño	-	A landslide 600x500 m dammed the Sapuyes River and destroyed La Chorrera village producing 300 deaths.
1936-08-14	Nariño	-	Large landslide dammed the Juanambú River forming a dam of about 40 m high, which failed producing 18 deaths.
1947-07-14	Nariño	-	Many landslides, some very large.
1950-07-18	Arboledas N. Santander	-	Cucutillas was blocked by many landslides. Many landslides within epicentral area.
1962-07-30	Caldas	6.0-6.8	Many landslides within epicentral area.
1967-02-09	Huila	-	Ground fissures and landslides spreads around epicentral area. Roads blocked and some rivers dammed between them La Ceiba and El Motilón Rivers. Sand blows and water expulsion also observed.
1970-09-26	Bahía Solano	6.5	Many landslides around Bahía Solano, Puerto Mutis, El Valle and Mecana. In Bahia Solano a landslide 300 m long produced severe damage affecting the cemetery, and blocking the airport road. Sand blows observed.
1979-11-23	Border with Panamá	6.5	Many landslides along Cali-Buenaventura and Honda-Manizales road. Small slides also reported in Bogotá and Manizales. Intense rainfalls considered likely to influence in landsliding processes.
1979-12-12	Tumaco	8.0	Many embankments collapsed.
1983-03-31	Popayán, Cauca	5.5	Many landslide in the Alto Cauca Area affecting the Florida I Power station.

Table 5.14 Earthquake inducing landslides in California (Adapted from Wilson and Keefer, 1985)

Earthquake	Date	M	Description
San Francisco	1906	7.7	Caused thousands of landslides within an area of 32000 km². Landslides killed at least 11. In mountainous terrain of the Northern California Coast Ranges, the most numerous landslides were rock and soil falls, rock slides, disrupted soil slides, and soil slumps. Many pre-existing soil slumps were reactivated, probably because the earthquake occurred within a rainy season when these slides were marginally stable. Two rock avalanches, each involving several hundred thousands or millions of cubic metres of material; killed at least 10 people in Santa Cruz Mountain. Lateral spreads and liquefaction observed in sandfills around coast of San Francisco, and along flood plain of rivers.
Long Beach	1933	6.2	On and near the coast between Long Beach and Newport Beach, man-made fills in areas underlain by wet sand and mud settled and spread. Ground cracks and spreads along canal banks. Soil slumps observed in man-made fills and in unconsolidated sediments. Rock falls and other landslides observed in mountainous terrain from canyons in central Los Angeles County, from as far south as San Clemente, and from as far north and west as Anacapa Island.
Kern County	1952	7.5	Caused hundreds of landslides in the southwestern Sierra Nevada, the Tahachapi Mountains, and the San Gabriel Mountains. Landslides included rock falls, rock slides, rapid soil flows, disrupted soil slides, and reactivation of coherent, deep-seated slides. Landslides blocked numerous roads and dammed several canyons creating small lakes. High concentration of landslides were near fault rupture. On the floor of San Joaquin Valley, ground cracks were reported within a very large area between Arvin and Maricopa, some of them associated with soil slumps, block slides and lateral spreads. Total area affected by landslides was approximately 7000 km².
San Fernando	1971	6.6	Landslides were concentrated in an area of about 250 km² of mountainous terrain north of surface fault rupture, and also occurred throughout an additional 3000 km² stretching southward from the epicentral region to the Palos Verdes Peninsula, and eastward through most of the San Gabriel Mountains. Landslides were rock falls, rock slumps, soil falls, disrupted soil slides, soil avalanches, and soil slumps. Rock falls were common in road cuts and on steep cliffs in intensely jointed conglomerates, gneisses, and granites. Soil falls occurred in uncemented and weakly cemented sands and gravels along stream banks. Disrupted soil slides and soil avalanches involved loose, unsaturated layers of silty and clayey sand less than 1 m thick. Most rock slumps reactivated pre-existing slumps. Soil slumps take place in weak, fine-grained fills. Several lateral spreads occurred in Van Norman Lakes area destroying several facilities costing millions dollars. Lateral spreads involved alluvial-fan materials containing a subsurface zone of saturated, loose sandy silt and fine sand.
Santa Barbara	1978	5.6	Caused a few dozen small rock falls, rock slides, and soil falls throughout an area of approximately 200 km². These slides were confined to steep road cuts in the Santa Ynez Mountains and sea cliffs near Goleta Point. Landslides were primarily in weakly cemented and closely jointed sedimentary rocks and in residual and colluvial soil developed on them.

5.2.2 Earthquake-induced landslides due to liquefaction

Some notes about dynamic behaviour of sandy deposits and description of landslides due to liquefaction were reported for world-wide historical cases by Seed (1968). These cases are compiled in Table 5.16, and according to Seed, this list shows that slides have invariably been

initiated by liquefaction of sandy soils in a loose to medium dense condition, as a result of earthquakes varying in magnitude between 5.5 and 8.5, and at epicentral distances varying from several kilometres to hundreds of kilometres.

Table 5.15 Characteristics of historical earthquake having mobilised sediments (Based on Seguret et al., 1984)

DATE	EARTHQUAKE	MAGNITUDE	LANDSLIDE	
			VOLUME Km ³ (area km x km)	EPICENTRAL DISTANCE (km)
1929	Grand Bank, New Zealand	7.2	1700	0-80
1954	Orleansville, USA	6.7	1x100	50-80
1923	Kanto, Japan	8.2	60	Epicentral area
1922	Chile	8.2	-	100
1953	Suva (Fiji)	6.75	0.15	Epicentral area
1956	Aegean sea, Greece	7.5	-	-
1964	Alaska, USA	8.5	0.075	-
1908	Strait of Messina, Italy	7.5	-	Epicentral area
1968	New Britain trench, New Guinea	6.7	-	55
1980	Eureka, USA	6.5-7.2	1x20	60

5.2.3 Keefer's studies

One of the most comprehensive studies about earthquake-induced landslides was presented by Keefer (1984). In his study, relationships between landslides and earthquakes that triggered them are presented. Most of his results have been discussed throughout previous chapters. Hence here the analysis will concentrate on those points not mentioned above.

A new report was presented by Keefer in 1999 but this is almost the same as that of 1984 complemented by descriptions of some cases after 1980 and the results of Keefer (1994). This last study concentrated on volume of sediments produced by earthquake-induced landslides in different regions.

Table 5.16 Landslides during earthquakes due to liquefaction. (After Seed, 1968)

DATE	EARTHQUAKE	M	LOCATION OF SLIDE	EPICENTRAL DISTANCE TO LANDSLIDE (km)	TPE OF STRUCTURE	SOIL TYPE
373 B.C	Helice, Greece	-	Helice	-	Coastal delta	-
1755	Lisbon, Portugal	≈8.7	Fez	≈ 690	River bans	Fluvial sediments, clays with sand and seams
1783	Calabrian, Italy	-	Soriano	56	Hillsides	Volcanic sediments
			Laureau	-	River banks	Fluvial sediments
			Terranova	8	River banks	Fluvial sediments
1811	New Madrid, USA	-	Mississippi river valley, Mo, Ark, Tenn, Ky, Ill, Ind, Vicksburg Miss.	Major slides:48 Minor slides:225	River valley banks and islands	Fluvial sediments, sand to muds
				467	Island	Fluvial sediments, sands to muds
1869	Cachar	-	Barak River at Silchar	64 to 128	River banks	Fluvial- sand to clay
1886	Charleston, USA	-	24 km SW of Ashley River	8 to 32	Railway fill	Fluvial and deltaic sands and slits
			Ashley River at Greggs	8 to 16	River bank	
1897	Assam, India	≈8.7	Shillong and Tutra regions	0 to 160	Canal banks Road embankments	Founded on alluvial plains
1899	Yukutat, Alaska, USA	-	Valdez	-	Submarine deposits	Deltaic and marine sediments- mainly silty sand and gravel
1902	St. Vincent	-	St. Vincent	-	Coastal delta	-
1906	Sana Francisco, USA	8.2	San Francisco area	16 to 48	Hillsides	
1907	Karatag	-	-	-	Loess slopes	Loess
1907	Chuyanchins	-	-	-	Loess slopes	Loess
1908	Alaska, USA		Valdez	40	Submarine deposits	Deltaic and marine sediments- mainly silty sand and gravel
1911	Alaska, USA	6.9	Valdez	64	Submarine deposits	Deltaic and marine sediments- mainly silty sand and gravel
1912	Alaska, USA	7.25	Valdez	-	Submarine deposits	Deltaic and marine sediments- mainly silty sand and gravel
1920	Kansu, China	-	Kansu province	-	Loess slopes	Loess
1923	Kwanto, Japan	8.2	Yokahoma area	64	Coastal hillsides	-
			Tokyo area	96	Coastal hillsides	
1925	Santa Barbara, USA	6.3	Santa Barbara	11	Earth dam	Silty sand
1928	Chile	8.3	El Teniente	160	Tailings dam	Mining waste

Table 5.16 (Continued)

DATE	EARTHQUAKE	M	LOCATION OF SLIDE	EPICENTRAL DISTANCE TO LANDSLIDE (km)	TYPE OF STRUCTURE	SOIL TYPE
1933	Long Beach, USA	6.3	Long Beach New Port Beach	32 4.8	Highway fills Highway fills	Fills over marshland on shore roads
1934	Bihar, Nepal	8.4	Sitamarhi to Purnea	0 to 129 113	Road and railway fills Lake banks	Fluvial sediments, including sands Alluvium- sand lenses
1935	India (Now West Pakistan)	7.6	Motihari Quetta	32 to 64	River banks	Alluvium- uncertain gradation
1940	El Centro, USA	7.0	All-American Canal	9.7	Canal banks	Levees and foundations of deltaic sands
			Alamo Canal	11 to 40	Canal banks	Levees and foundations of deltaic sands
			Solfatara Canal	40 to 48	Canal banks	Levees and foundations of deltaic sands
			Brawley	32	Road and railway fills	Levees and foundations of deltaic sands Deltaic and fluvial sands
1941	Garm	-	-	-	Loess slopes	Loess
1943	Faizabad	-	-	-	Loess slopes	Loess
1948	Fukui, Japan	7.2	Fukui plain	0 to 24	Levees, river banks, road and railway fills	Aeolian sands, beach sands, fluvial sands and silts
1949	Chait	7.5	Surchob and Yasman River valleys	8 to 40	Loess slopes	Loess
1950	Imperial Valley, USA	5.4	Calipatria area	1.6 to 8.0	Canal banks	Deltaic and aeolian sands
1954	Anchorage, Alaska, USA	6.7	Rabbit Creek	32 to 64	Embankment	Fill on sand
1957	San Francisco, USA	5.3	Lake Merced	13	Lake banks	Aeolian and beach sands
1959	Jaltipan	6.5	Coatzacoalcos	32	River banks, water-front fill	Fine sandy silt, uniform and loose
			Minatitlau-Coatzacoalcos highway	32 to 48	Road and bridge approach fills	Fill over marshland
1960	Chile	8.4	Rinihue	225	River banks Highway and railway fills	Fluvial and glacial sands Foundations of fluvial and glacial gravels, sands and silts
			Puerto Montt	386	Coastal terraces	Glacio-fluvial deposits
			Valdivia	201	Sea walls and quay walls River banks	Fill mainly sands to silty sands, loose Fluvial sediments
1964	Alaska, USA	8.3	Anchorage	113	Coastal bluff	Sand layers and lenses in clay deposit
			Valdez	64	Coastal delta	Silty sand and gravel (N=15)
			Seward Kenai lake	145 129	Coastal delta Lake deltas	Deltaic sandy gravels, some sand lenses

Table 5.16 (Continued)

DATE	EARTHQUAKE	M	LOCATION OF SLIDE	EPICENTRAL DISTANCE TO LANDSLIDE (km)	TYPE OF STRUCTURE	SOIL TYPE
1964	Niigata, Japan	7.3	Niigata area	56	Earth banks	Fluvial sand (N<15)
1965	Chile	7.2	El Cobre	≈40	Tailings dam	Mining waste
			La Patagua	≈15	Tailings dam	Mining waste
			Hierro Viejo	≈26	Tailings dam	Mining waste
			Los Monquis	≈13	Tailings dam	Mining waste
			El Cerrado	≈29	Tailings dam	Mining waste
1965	Seattle, USA	6.7	Capitol lake Boulevard, Olympia	61	Road causeway	Sand/gravel fill over lake and tidal sediments
			UnionPacific at Tumwater	61	Railway on benched slope	Cut/fill slope in outwash sands
			Suquamish	42	Coastal bluff	Till over fine sand and silt strata
			Port Orchard	29	Waterfront fill	Sand over beach sand and bay mud
			E. Mercer way, Mercer Island	16	Roadways on benched slopes	Sand on tills and outwash sands
			Edmonds	47	Dumped fill on slope	Sandy till and refuse on till slope
			Foster golf course, Duwamish	16	River terrace	Fluvial sands and silts
			Victor	39	Highway fill	Sandy fill at toe of coastal bluff
1966	Parkfield, USA	5.5	Cholame creek north of Cholame	27	Stream banks	Fluvial sediments, sand strata or lenses

In Keefer (1984) data from 40 historical world-wide earthquakes were studied to determine the characteristics, geologic environments and hazards of landslides caused by seismic events. These events cover a period from 1811, with the New Madrid earthquake, to 1980, with the Mammoth Lakes earthquake. These data also cover a wide range of earthquake magnitudes and intensities. Table 5.17 presents these cases and some of their characteristics.

The most abundant types of landslides during these events were rock falls, disrupted soil slides, and rock slides, whereas subaqueous, slow earth flows, rock block slides and rock avalanches were the least common.

Table 5.17 Keefer's case histories (Keefer, 1984a). Numbers refer to original Keefer's list.

	DATE	EARTHQUAKE	MAGNITUDE (M _s)	MAXIMUM INTENSITY (MMI)
1	16-Dec-1811	New Madrid, Missouri, USA	7.5*	X-XI
	23-Jan-1812		7.3*	
	7-Feb-1812		7.8*	
2	1-Sep-1886	Charleston, South Carolina, USA	6.8	IX
3	18-Apr-1906	San Francisco, California, USA	7.9 [§] (8.25-8.3)	X-XI
4	16-Dec-1920	Kansu (Haiyun), China	7.8 [§] (8.5)	X
5	15-Jan-1934	Bihar, India-Nepal	8.1 [§] (8.3)	XI
6	19-May-1940	Imperial Valley, California, USA	7.1	XI-XII [§]
7	23-June-1946	Vancouver Island, Canada	7.2-7.3	X
8	28-June-1948	Fukui, Japan	7.25-7.3**	X
9	13-Apr-1949	Puget Sound, Washington, USA	7.0	VIII
10	10-July-1949	Khait, USSR	7.6	IX-X [§]
11	15-Aug-1950	Assam, India	8.6 [§] (8.6-8.7)	VIII
12	22-Mar-1957	Daly city, California	5.3 ^{§§}	
13	10-July-1958	Sotheast Alaska, USA	7.7 [§] (7.9)	X
14	18-Aug-1959	Hobgen Lake, Montana	7.1**	VII
15	22-May-1960	Chile	9.5 [§] (8.3-8.5)	XI-XII
16	28-Mar-1964	Alaska, USA	9.2 [§] (8.3-8.4)	X
17	16-June-1964	Niigata, Japan	7.3	XI-XII
18	29-Apr-1965	Puget Sound, Washington, USA	6.5	X-XI
19	28-June-1966	Parkfield-Cholame, California, USA	6.2	VIII
20	23-May-1968	Inangahua, New Zealand	7.1	VII-VIII
21	31-May-1970	Perú	7.9 [§] (7.8)	VII-IX
22	31-Oct-1970	Madang, Papua New Guinea	7.1	X-XI
23	9-Feb-1971	San Fernando, California, USA	6.5	VIII
24	26-Apr-1973	Honou, Hawaii, USA	6.1	VIII-IX
25	28-Dec-1974	Indus Kohistan, Pakistan	6.2	VIII
26	29-Nov-1975	Kilauea, Hawaii, USA	7.1	VIII
27	4-Feb-1976	Guatemala	7.5	VIII
28	19-Mar-1976	Khulm, Afghanistan	5.5	IX
29	6-May-1976	Fruili, Italy	6.3-6.5	VIII-IX
30	11-July-1976	Darien, Panamá	7.0	VIII-X [§]
31	27-July-1976	Tangshan, China	7.5 [§] (7.7-8.0)	XI [§]
32	21-Mar-1977	Khurgu, Iran	6.9	VIII+
33	23-Nov-1977	San Juan Province, Argentina	7.4	IX
34	14-Jan-1978	Izu-Oshima Kinkai, Japan	6.8	IX-X [§]
35	12-June-1978	Miyagi-ken-oki, Japan	7.4	VII-IX [§]
36	13-Aug-1978	Santa Barbara, California, USA	5.6	VII
37	15-Mar-1979	Homestead Valley, California, USA	5.2 ^{§§}	VI
38	6-Aug-1979	Coyote Lake, California, USA	5.4	VII
39	24-Jan-1980	Mount Diablo, California, USA	5.8	VI-VII
40	25-May-1980	Mammoth Lakes, California	6.1	VII

* M_s determined from relations between magnitude, attenuation of Modified Mercalli Intensity, or particle velocity

§ M_w determined by Kanamori (1977); M_s given in parentheses.

§ Intensity converted to Modified Mercalli Intensity using relations in Medvedev (1962)

** Scale of magnitude not reported

§§ Richter local magnitude (M_L)

Analysis is presented by plots of the area affected by landsliding vs. magnitude, maximum epicentral distance or fault projection distance to landslides vs. magnitude, and minimum intensity (MMI) vs. number of events. These relationships have been discriminated into three slide groups,

depending on the internal material disruption: coherent slides, disrupted slides, and falls and lateral spreading. These relations were discussed in Chapter 4.

Although the exact number of landslides triggered by each earthquake was difficult to define, Keefer shows that an approximate relation between the number of landslides and the earthquake size can be established. An earthquake with $M < 6.0$ can trigger some hundreds of slope failures, whereas one of $M > 7.0$ can trigger thousands of these failures. In the present study no attempt was made to determine number of landslides.

The smallest earthquakes that cause landsliding of these different groups have been defined. However, in very prone areas of highly susceptible slopes a minimum ground shaking may be enough to induce slope movements. Values reported by Keefer are summarised in Table 5.18.

Table 5.18 *Smallest earthquakes to cause landslides of various types in the database of Keefer (1984a)*

LANDSLIDE TYPE	MAGNITUDE
1. Rock falls, rock slides, soil falls, and disrupted soil slides	$M_L \approx 4.0$
2. Soil slumps and soil block slides	$M_L \approx 4.5$
3. Rock slumps, rock block slides, slow earth flows, soil lateral spreads, rapid soil flows, and subaqueous landslides	$M_L \approx 5.0$
4. Rock avalanches	$M_S \approx 6.0$
5. Soil avalanches	$M_S \approx 6.5$

Some relationships between the landslide mechanism and soil deposits can be found. Soil slumps were more frequent in man-made fill, flood alluvia and old slump deposits, whereas lateral spreads are associated more with flood plain alluvia, man-made fills, deltaic deposits and sand dunes. Frequency in this case is referred to the number of earthquakes in which each type of mechanism occurs in a specific soil deposit compared to total number of earthquakes included in the analysis.

Although most or all types of earthquake-induced landslides represent some hazard to human life and property, historical evidence shows that the predominant threats to life are from rock avalanches, rapid soil flows and rock falls. Zones at risk from rock falls extended only a few hundred metres from the base of steep slopes, but zones at risk from rock avalanches or rapid soil flows extend for several kilometres from the landslide source area.

Slope area denuded by historical earthquakes was compiled by Keefer (1999), and volume of sediments produced by different earthquakes were compiled and presented by Keefer (1994, 1999). Combined data from these studies are presented in Table 5.19. The relationship between volume of sediments and earthquake magnitude is shown in Figure 5.12.

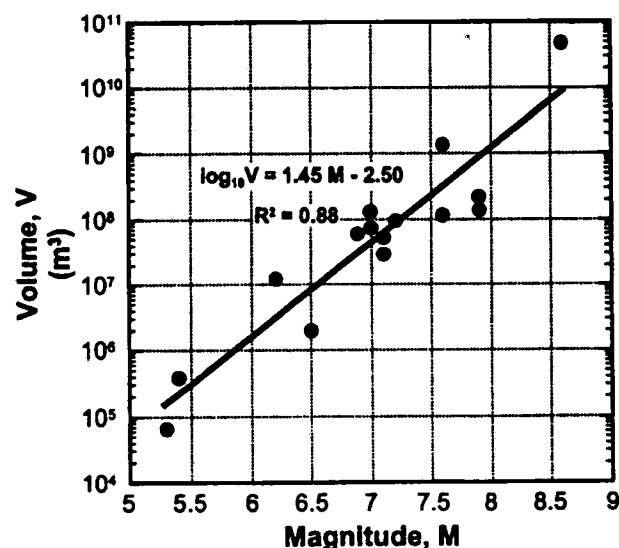


Figure 5.12 Magnitude-Volume of sediments relationship (Keefer, 1994)

5.2.4 Extension of Keefer's database

A database, which extended the Keefer database to 1997, was compiled and presented by Rodríguez *et al.* (1999). The approach adopted was similar to that of Keefer. A total of 36 earthquakes were identified, some of which are multiple events. Table 5.20 presents these cases and their principal characteristics.

Table 5.19 Area denuded and volume of sediments due to historical earthquakes (Adapted from Keefer, 1994 and 1999)

Earthquake	Date	M	Area over which denudation measured (km ²)	Slope area denuded (km ²)	Percent of slope area denuded	Volume of landslide material (m ³)
Chile	1960-05-22	9.5	9000	>250	>2.8	-
Assam, India	1950-08-15	8.6	31000	10230	33	4.7x10 ¹⁰
Torricelli Mts. New Guinea	1935-09-20	7.9	1662	133	8	2.15x10 ⁸
Guatemala	1976-02-04	7.6	-	-	5 to > 50	1.16x10 ⁸
San Juan, Argentina	1977-11-23	7.4	700	91-140	13-20	-
Northeast Ecuador	1987-03-05	7.2	90	>68	75-100	9.5x10 ⁷
Northeast Ecuador			430	108-322	25-75	
Madang, New Guinea	1970-10-31	7.1	240	60	25	2.8x10 ⁷
Darien, Panamá	1976-07-11	7.0	160	32	20	2.8x10 ⁷
Darien, Panamá			193	21	11	
Arthur's Pass, New Zealand	1929-03-09	6.9	820	246-410	30-50	5.9x10 ⁷
Northridge, California	1994-01-17	6.7	-	-	>75	-
Mammoth Lakes, California	1980-05-25	6.2	38	2	5	1.2x10 ⁷
Mammoth Lakes, California			158	6	4	
Perú	1970-05-31	7.9	-	-	-	1.4x10 ⁸
Buller, New Zealand	1929-06-17	7.6	-	-	-	1.3x10 ⁹
Inangahua, New Zealand	1968-05-23	7.1	-	-	-	5.2x10 ⁷
Loma Prieta, California	1989-10-17	7.0	-	-	-	7.5x10 ⁷
Coalinga, California	1983-05-02	6.5	-	-	-	1.9x10 ⁶
San Salvador, El Salvador	1986-10-10	5.4	-	-	-	3.75x10 ⁵
Daly City, California	1957-03-27	5.3	-	-	-	6.7x10 ⁴

Contrary to Keefer's findings, the number of landslides in each seismic event, for this database, does not appear to be dependent on the earthquake size. On the other hand, in the new database there are relatively fewer soil and rock avalanches, slow earth flows, and lateral spreads than in the Keefer's data, but there is a much larger proportion of disrupted soil slides. The smallest earthquakes associated with various types of landslides were defined for the extension of Keefer's database; these values are presented in Table 5.21.

As Keefer pointed out, many landslides occur without seismic triggering, and therefore on a slope that is imminently unstable a landslide could be caused by very weak shaking. The smallest earthquake to have induced landslides found during compilation of the new database was for the Ledu (Qinghai, China) earthquake of 7 March 1984 with $M=2.9$.

Table 5.20 Case histories in the Rodríguez et al.(1999) database

DATE	EARTHQUAKE	MAGNITUDE		MAXIMUM INTENSITY (MMI)	AREA AFFECTED BY LANDSLIDES (km ²)
		M_s	M_w		
23-Nov-1980	Irpinia, Italy	6.9	6.9	X	13000
2-MAY-1983	Coalinga, USA	6.7	6.2	VIII	650
9-May-1983		4.7	5.1	-	-
28-Oct-1983	Borah Peak, USA	7.3	6.9	IX	4200
14-Sept-1984	Nagoken-Seibu, Japan	6.4	6.2	VIII	3500
3-Mar-1985	Valparaiso, Chile	7.8	8.1	VIII	17000
3-Mar-1985		6.4	5.9	-	-
9-Apr-1985		7.2	7.1	-	-
13-Sept-1986	Kalamata, Greece	5.8	6.0	VIII	70
10-Nov-1986	San Salvador, El Salvador	5.4	5.7	VIII	380
7-Jan-1987	Diebu, China	5.4	5.3	VII	280
2-Mar-1987	Edgumbe, New Zealand	6.6	6.5	IX	380
6-Mar-1987	El Napo, Ecuador	6.0	7.1	IX	2500
6-Mar-1987		6.9	6.8	-	-
1-Oct-1987	Whittier Narrows, USA	5.8	6.1	VIII	4200
4-Oct-1987		4.8	3.9	-	-
24-Nov-1987	Supertition Hills, USA	6.2	6.2	-	3300
24-Nov-1987		6.6	6.6	-	-
21-Aug-1988	Nepal	6.6	6.8	VIII	90
16-Oct-1988	Killini, Greece	5.6	5.9	VIII	40
25-Nov-1988	Saguenay, Canada	5.8	5.8	VIII	45000
7-Dec-1988	Spitak, Armenia	6.8	6.7	IX	2200
7-Dec-1988		5.8	-	-	-
23-Jan-1989	Soviet Tajik, Tajikistan	5.5	-	VII	12
17-Oct-1989	Loma Prieta, USA	7.1	6.9	VIII	14000
20-June-1990	Manjil, Iran	7.3	7.4	X	1000
16-July-1990	Luzon, Philippines	7.8	7.7	VIII	3000
22-Apr-1991	Valle de la Estrella, Costa Rica	7.6	7.5	IX	2000
13-Mar-1992	Erzincan, Turkey	6.8	6.7	IX	150
15-Mar-1992		5.8	-	-	-
25-Apr-1992	Cape Mendocino, USA	7.1	7.0	VIII	625
26-Apr-1992		6.6	7.0	-	-
26-Apr-1992		6.6	6.6	-	-
19-Aug-1992	Suusamy, Kyrgyzstan	7.3	7.0	X	2500
		6.7	6.7	-	-
		7.3	7.4	-	-
12-July-1993	Hokkaido-Nansei, Japan	7.6	7.8	X	100
10-Aug-1993	Ormond, New Zealand	6.0	6.2	VII	5
10-Aug-1993	Fiorland, New Zealand	7.0	6.8	VIII	500
21-Sept-1993	Klamath Falls, USA	5.8	5.9	VII	420
21-Sept-1993		5.8	5.9	-	-
17-Jan-1994	Northridge, USA	6.8	6.7	IX	10000
6-June-1994	Paéz, Colombia	6.6	6.8	X	250
18-June-1994	Arthur's Pass, New Zealand	7.1	6.8	VII	85
17-Jan-1995	Hyogu-Ken Nanbu, Japan	6.8	6.9	X	910
19-Jan-1995	Tauramena, Colombia	6.6	6.5	VIII	4550
29-May-1995	Arthur's Pass, New Zealand	6.5	-	-	85
26-Sept-1997	Umbria-Marche, Italy	5.5	5.7	VIII	-
26-Sept-1997		5.9	6.0	-	-
14-Oct-1997		5.5	-	-	-

Comparison of magnitude-distance and magnitude-area affected by landslides relationships proposed by Keefer and this database extension was presented in graphs in Chapter 4. Based on these new data Rodríguez *et al.* (1999) suggest that for an intermediate magnitude range between about 5.3 and 7.0, the upper bound of the affected area is slightly above that suggested by the pre-1980 database. However, except for the Saguenay, San Salvador, and Diedu earthquakes, data points above Keefer's line are all associated with multiple seismic events. Two data points, the most obvious outliers on the surface wave magnitude plot, lie significantly above the Keefer's upper bound. These relate to the Whittier Narrows (California) earthquake of 1987, which was a multiple event, and the Saguenay (Canada) earthquake of 1988, which is discussed below.

Table 5.21 Smallest earthquakes to cause landslides of various types in the extension of Keefer's database (Adapted from Rodríguez et al., 1999)

LANDSLIDE TYPE	MAGNITUDE
1. Rock falls, rock slides, soil falls, and disrupted soil slides	$M_L=5.5$, $M_S=5.4$
2. Soil slumps and soil block slides	$M_L=5.5$, $M_S=5.4$
3. Rock slumps, rock block slides, slow earth flows, soil lateral spreads, rapid soil flows, and subaqueous landslides	$M_L=6.5$, $M_S=5.9$
4. Rock avalanches	$M_S=6.5$
5. Soil avalanches	$M_S=6.0$

The landslides associated with the 1988 Saguenay earthquake clearly represent a very special case. Landslides during this event were concentrated in the epicentral area, though the distance and affected area by landsliding are strongly dependent on slope susceptibility, and on the effect of simultaneous arrival of seismic waves. The most distant landslides on natural slopes were in very sensitive clays underlain by a glacial till deposit that dipped in direction of the slope face. The upper part of the slip surface of these cases coincides with the clay-till interface, while the clay sensitivity at this depth is typically between 20 and 40, and in some areas reaches values as high as 100 to 500. Fill embankments also failed at similarly long distances from the epicentre to those of natural slopes, and in these cases slopes consisted of fine granular deposits sloping at an angle close to

the angle of shearing resistance of the material, and were founded on saturated loose deposits. On the other hand, Somerville *et al.* (1990) found for this earthquake that for distances beyond 50 km, the peak horizontal ground motions are controlled by the reflections of shear waves from interfaces in the lower crust arriving simultaneously with direct radiation from the source.

CHAPTER 6

NEW DATABASES OF EARTHQUAKE-INDUCED LANDSLIDES AND LANDSLIDE-INDUCING EARTHQUAKES

A comprehensive literature survey was carried out, using various sources, all of which are listed in Appendix D. All the accumulated information for the earthquakes and the landslides was entered in two spread sheets on a personal computer, and processed using the EXCEL database facility.

The first step of this survey was the review of reconnaissance reports of damage during historical earthquakes recorded by regional and international agencies, in order to find in which cases landslides were triggered. From these reports, descriptive seismic catalogues were identified for each region, from which additional earthquakes that triggered landslides were identified and individual reports for each one were sought.

No attempt to identify negative evidence was made. It was found by Rodríguez (1996) that many reconnaissance reports concentrate on structural damage with no mention on landslides, although some of them were found to have caused landslides. Only two of around 140 earthquakes studied by Rodríguez (1996) have specific mention of landslides not being triggered.

Many cases have only a visual description of seismic effects on slopes, mainly in those cases where lifelines were interrupted by sliding, hindering and delaying the rescue operations. Few reports have quite complete information of the geological and topographical conditions and morphometric characteristics of slides.

The seismic parameters of the earthquakes relevant to seismic hazard analysis were calculated as explained in Chapter 2. Geotechnical information on the landslides triggered was complemented

with data from stability analysis obtained for some localities from reports in geotechnical Journals and Conference Proceedings. Based on this information, the main parameters that appear to be associated with the instability process were identified, as shown in Chapter 3. Each of these parameters showed a wide variation within the cases studied, so the use of classification systems was required for them.

Two complementary databases were created: a landslide-inducing earthquakes database that contains the regional and earthquake characteristics, while an earthquake-induced landslides database contains the climatic and landslides features. Data was compiled, summarised and classified before transferring them to the databases. In order to optimise this process, data sheets for collecting the information, were designed based on a preliminary review and selection of the most important factors that control slope stability, which were explained in Chapter 3.

Three different data formats were designed for compiling the relevant information: an earthquake report sheet, an earthquake-induced landslides report sheet, and a landslide report sheet. Parameters have been described by using appropriate classification systems in order to reduce the variability of data. Examples of these sheets are presented in Annex D.

The earthquake report sheet contains the earthquake characteristics and regional setting of the affected area. It also contains the strong-motion characteristics calculated or derived from accelerograms.

The earthquake-induced landslides sheet compiles data related to general characteristics of the landslides induced by a specific earthquake, such as areal distribution, and for different mechanisms, relationships between different landslide mechanisms and seismic parameters. It also contains the inventory of individual landslides identified for the respective earthquake, as well as a general description of slope behaviour during the earthquake.

The landslide report sheet contains the landslide description, strong-motion parameters for the slope site and climatic conditions for specific slope. Causes and effects of the specific slide are also included.

These data sheets were completed for the most important cases, and for those cases where sufficient information was found. For many cases descriptions were limited and data sheets were not made but a general summary recorded.

Due to space limitations it was impossible to include all these data in this thesis, but a summary of them was compiled by Rodríguez (2001) as an internal report of the EESE Section at the Imperial College. Maps of areas affected by landslides, from which distance and area relationships with earthquake magnitude were obtained, are also included in this report.

As mentioned previously, two database sheets were designed to compile the relevant information for the proposed models and for future applications. The following is a general description of the fields included in each sheet. Databases of the landslide-inducing earthquakes and the earthquake-induced landslides are also compiled in Rodríguez (2001).

6.1 Landslide-inducing earthquakes database

The study of geotechnical earthquake engineering requires the definition of some seismic parameters that help to quantify the earthquake magnitude and its effects. The earthquake characteristics can be divided into six main groups; a) earthquake identification, b) geographical position and rupture characteristics, c) the earthquake size parameters, d) geological and topographical setting of areas where landslides were induced, e) effects quantification in terms of area affected by landslides and maximum distance to different slide mechanisms, and f) antecedent climatic conditions.

6.1.1 Earthquake identification

An earthquake is identified by date and origin time. For pre-instrumental cases, earthquake parameters are very difficult to define, and in many cases no data were found to completely identify the earthquake. A detailed analysis of different global, regional and local seismic catalogues was carried out in order to identify as complete as possible each earthquake. Sources of more reliable data on earthquake parameters is included in database. The name given to different earthquakes was not reported since many cases were found with the same name, which may lead to confusion in earthquake identification.

The origin time is the time at which the faulting began, which can be estimated by the arrival time of the first P-waves at a recording station and its propagation velocity through the media. In some cases the origin time is reported as the time at which the first P-wave was recorded. Origin time is recorded as UTC (GMT) time.

The date assigned to each earthquake corresponds to that given by the GMT origin time. In some occasions the local time can give a different date as in the case of the 1994 Fierland earthquake in New Zealand that occurred the 9th of August, but it appears reported the 10th August.

6.1.2 Geographical position and failure characteristics

Earthquakes occur around the world, but those zones where tectonic plates connect have shown the highest historical seismic activity, thus it would be expected that the locations of earthquakes would be concentrated near plate boundaries, and source depth would be associated with boundaries contacts. The geographical position of earthquakes is firstly defined by the country where the earthquake occurred and secondly by the geographical co-ordinates of the epicentre and focal depth.

The hypocentral locations for the earthquakes were obtained, wherever possible, from special studies of the particular earthquake. In the absence of such studies, the source parameters were obtained

from the Regional Catalogue of Earthquakes of the International Seismological Centre (ISC). For more recent cases earthquake parameters were obtained from the NEIC catalogues and EERI reconnaissance reports. Source of the most reliable data is also recorded in the database.

Rupture mechanism, aftershocks sequence and the tectonic environment where earthquake occurred were recorded for some cases, and used to complement earthquake identification.

6.1.3 Earthquake size

Maximum intensity reported and different magnitude values for each earthquake were compiled for defining earthquake size. All data reported were obtained from special studies, when available, or from catalogues. Moment magnitude, when not reported, was calculated from seismic moment values reported mainly by Harvard University.

6.1.4 Geological and topographical setting

Local site effects play an important role in earthquake-resistant design, and usually is considered by the development of *design ground-motions* (i.e. motions that reflect the levels of strong-motion amplitude, frequency content, and duration for an specific design site). Site conditions can influence the strong-motion characteristics. The extent of their influence depends on the geometry and material properties of the subsurface materials, on the site topography, and on the characteristics of the input strong motion.

The quantification of the influence of the site conditions on the strong-motion requires the identification of different regional environmental conditions in which the earthquakes occur and of the site conditions where earthquake effects have been observed. The geological environment of a specific region can be defined by: a) materials and their genesis, b) primary and secondary structures that the material shows, c) the influence of climatic conditions and changes produced after formation, especially the degree of weathering, and d) the regional groundwater conditions. These factors for areas with landsliding processes were identified for each earthquake.

Rocks and materials derived from them have traditionally served as the object of study for geologists and geomorphologists and consequently they have been responsible for their original classification and nomenclature. As a result, natural materials have been classified principally by mode of origin, mode of transport, mineralogy and constituent particle size, rather than by any direct consideration of geomechanical properties. To the engineer, natural materials can be divided into two major groups on the basis of strength and behaviour: *rock* and *soil*. Many criteria for establishing the difference in mechanical behaviour between soil and rock have been proposed, especially based on the use of the material in engineering works. For earthquake engineering proposes the regional geology can be based on geological classification of materials whereas for site effects such as slope stability a specific geotechnical classification is more convenient. In the present work a geological classification of natural materials has been used to describe the geological setting.

Depending on their origin, soils can be classified in two main groups: *transported soils* and *residual soils*. Agents of erosion, including rivers, wind, moving ice, water waves, produced the *denudation* of the land. They also transport the weathered material, the *detritus*, away from the areas where it is derived. The deposition of detritus produce transported soil deposits. In other circumstances, the weathered material can remain in position overlying the parent material as residual soil deposits.

From the genesis point of view, rocks are classified into three main groups; *igneous*, *metamorphic* and *sedimentary rocks*. Molten rock material which is generated within or below the Earth's crust reaches the surface from time to time, and flows out from volcanic orifices as *lava*. Similar material can be injected into the rocks of the crust, giving rise to a variety of *igneous intrusions* which cool slowly and solidify. The solidified lavas and intrusions constitute the igneous rocks. When the rock is formed on the crustal surface the material is called *extrusive rocks*, whereas when the materials in formed within the crust it is called *intrusive rocks*; the latter group can constitute huge masses of intrusive rock or small lenses or sheets of material, hence it can be divided into *major intrusive rocks* or *minor intrusive rocks*. Figure 6.1 shows the geological classification of igneous rocks used in this report, which is based on Blyth and de Freitas (1984).

The fragments, having been sorted by erosion agents and built up into deposits of sediments due to a complex process of *diagenesis* can become strong enough to constitute *sedimentary rock deposits*. The diagenesis process includes the deposition, consolidation and lithification of the sediment due to the action of high pressures and temperature. The geological classification of sediment deposits and sediment rocks is shown in Figure 6.2.

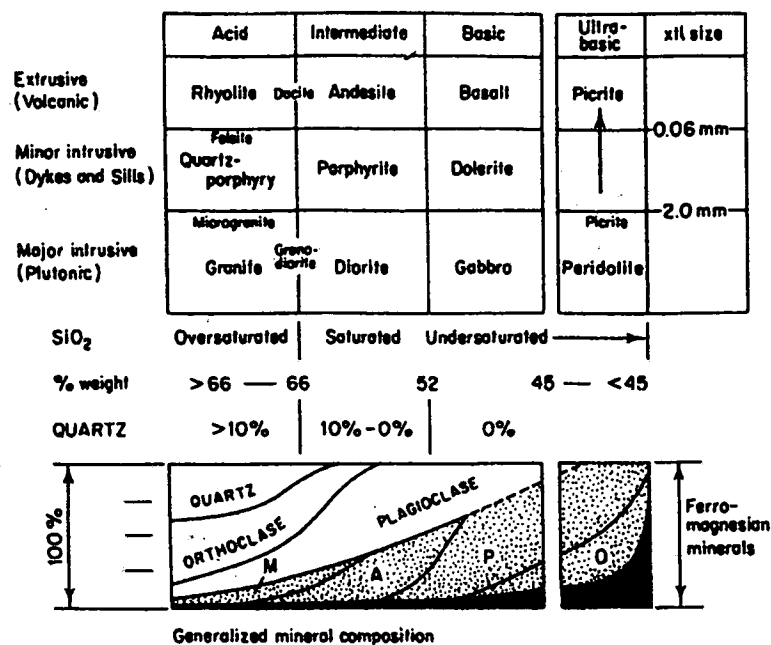


Figure 6.1 Classification of igneous rocks (Blyth and De Freitas, 1984)

Metamorphic rocks are derived from earlier igneous or sedimentary rocks, but transformed from their original state by heat or pressure, so as to acquire new characteristics. *Metamorphism* is the term used to denote the transformation of the rocks into new types by the recrystallization of their constituents. Figure 6.3 shows the geological classification of metamorphic rocks.

Besides the origin of natural material deposits it is important to define which processes have influenced and changed the original condition of these materials. In slope stability analysis, the most important features of a specific deposit related to the changes during its geological history are the changes in the stress condition, and the effects in properties due to environmental agents such as climate and human intervention.

PARTICLES (or GRAINS) size (mm)	name	DETRITAL (terigenous)			DETRITAL (calcareous)		DETRITAL (pyroclastic)	
		> 50% GRAINS OF ROCK & SILICATE MINERALS			> 50% GRAINS OF CALCIUM CARBONATE		> 50% GRAINS OF VOLCANIC ROCK	
		Unconsolidated raw material	Geological Term	Consolidated as rock	Unconsolidated raw material	Consolidated as rock	Unconsolidated raw material	Consolidated as rock
200	Boulder	Storm beach Colluvium (scree, or talus)	RUDACEOUS (mainly rock fragments)	Conglomerate Breccia	Storm beach Colluvium Coral reef debris	Conglomerate Breccia	Volcanic bombs & ejected rock blocks	Agglomerate Volcanic breccia
60	Cobble	Glacial boulder beds		Tillite				
4	Pebble	Coarse alluvium (e.g. wadi)		Conglomerate		Carbonate gravel		
2	Granule	Fine alluvium Deltaic grits and sands		Grit	Carbonate gravel: eg. coral reef debris. Shell rich beach gravel	Pisolithie grit Carbonate grit	Fine strands and droplets of lava ejected into the atmosphere	Lapilli tuff
0.06	Sand	Sand beach Desert sand & dust	ARENACEOUS (mainly mineral fragments)	Sandstone (arkose, grey- wacke, and other varieties) Loose	Carbonate sand, eg. shell and beach	Carbonate sand oolitic limestone	Volcanic ash	
0.002	Silt	Estuarine silt	ARGILLACEOUS (mainly mineral fragments)	Siltstone	Carbonate Silt, eg. lagoon silt	Carbonate silt		Tuff
	Clay	Glacial silts & clays clay oosa		Mudstone & Shale	Desert Dust Carbonate mud	Loose Cement-stone Chalk	Volcanic dust	

PARTICLE (or CRYSTAL)	CHEMICAL & BIOCHEMICAL (= organic)				
	SILICEOUS	CALCAREOUS	CARBONACEOUS	SALINE	FERRUGINOUS
Megeocrystalline		Coral reef, shell banks		(Some evaporite sediments become megacrystalline with diagenesis)	
— 2 mm —		Coral (reef), crinoidal and shelly limestones	Peat & drilled vegetation Lignite (brown oal)	Mineral crust precipitated in salt lakes and marine mud-flats	
Mesocrystalline		Pisolithie grit	Humic (bituminous) ooids		
— 0.06 mm —		Oolitic limestone	Anthracite	Evaporites, anhydrite, halite, potash-salts, Gypsum	
Microcrystalline	Silica gel & colloids, radiolaria, diatoms	Algal, coccolithic and other forams	Organic muds		Algal precipitates of iron-rich carbonates and silicates colloidal feric hydroxides, siderite, chamosite and bog-iron ore
	Flint, chert, jasper, radiolarite, diatomite	Algal and ferrominiferous limestones CaCO ₃ precipitated from solution, & CaMg (CO ₃) ₂ , replacement Travertine (tuffs) & Dolomite	Oil shale		

Figure 6.2 Classification of sediments and sedimentary rocks (Blyth and de Freitas, 1984).

The movement of the crustal plates induces the folding, faulting or fracturing of the original material, and produces discontinuities into the solid material. The macro-structural characteristics produced in this process such as joints, fissures and other types of discontinuity are called the *secondary structures* and are differentiated from the *primary structures* that are those structural features that appeared at the moment of material formation, such as bedding, foliation or deposit intrusion in

rocks. Table 6.1 shows a list of primary and secondary structures considered in the geological setting description of deposits affected by earthquakes and in zones where slides were triggered.

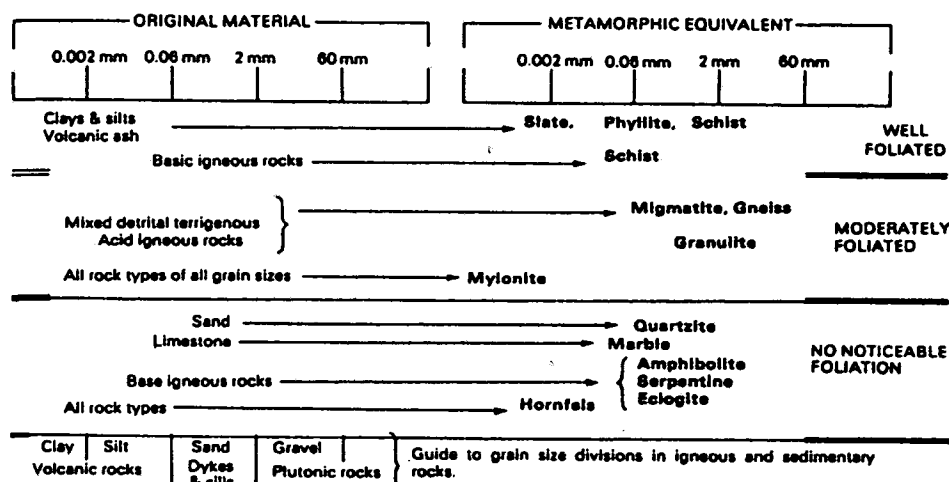


Figure 6.3 Classification of metamorphic rocks (Blyth and de Freitas, 1984).

Changes in material properties are due mainly to *weathering* from atmospheric agents. *Chemical weathering*, or *decomposition*, is the breakdown of minerals into new component by action of chemical agents. *Mechanical weathering*, or *disintegration*, breaks down rocks into small particles by action of physical agents. *Biological weathering* describes those mechanical and chemical changes of the ground that are directly associated with the activities of animals and plants.

Table 6.1 Geologic structures (Adapted from Blyth and de Freitas, 1984).

Main Structures	Secondary Structures
1. Igneous rocks - Dykes - Sills - Ring Structures - Laccoliths - Phacolith - Plutons - Stocks - Batholiths - Sheets	1. Folds - Anticline - Syncline - Monocline - Valley bulges - Salt domes
2. Sedimentary rocks - Normal bedding - Crossed bedding	2. Faults - Normal - Reverse - Thrust
3. Metamorphic rocks Foliation	3. Joints - Joints in young sediments as shrinkage joints - Joints of folded sediments - Strike joints - Dip joints - Tension joints - Oblique joints

Several schemes have been devised for the description of granitic rock weathering and these can be generalised to be applicable to a wide range of rock types. The three main criteria for classification of weathering grades along a weathering profile are: degree of colour decolouration, ratio of soil/rock, and presence or absence of original rock structure. The scheme proposed by Dearman (1995), and used here in the geological setting description, is shown in Table 6.2.

The ground water condition has clearly been identified as one of the most important factors in slope instability (Crozier, 1986; Small and Clark, 1982), but the influence of regional water condition has not been taken into account in seismic analysis and it is very uncommon find in earthquake reports a description of the regional ground water conditions and the effects associated with it. For the description of the geological setting of the regional ground water condition, only a brief description of the position of the water level relative to the height of the slopes in the zone is used. Besides the water level position a more complete description of ground water condition must be considered in seismic risk analysis, which would include hydrological data of the zone and the regional water circulation.

Table 6.2 Scale of weathering grades of rock mass. (Dearman, 1995)

Term	Description	Grade
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	I
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.	II
Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock present either as a continuous framework or as corestones.	III
Highly weathered	More than half of the rock material is decomposed or disintegrated to soil. Fresh or discoloured rock present either as a continuous framework or as corestones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

6.1.5 Earthquake effects

Natural events such as earthquakes are capable of causing deaths, injuries, and property damage. Hazards associated with earthquakes are commonly referred to as *seismic hazards*. The practice of earthquake engineering involves the identification and mitigation of seismic hazards. The most important seismic hazards that can be the cause of severe damage are: a) ground shaking, b) structural hazards, c) liquefaction of soil deposits, d) landslides, e) retaining structures failures, f) damage on lifeline systems, and g) tsunamis and seiches.

The main aim of this survey is the identification of the seismic hazard due to landslides, although some landslide phenomena are due to liquefaction such as lateral spread of soil deposits. In this study, only lateral spreads due to liquefaction were considered. Other liquefaction phenomena were recorded in the databases as complementary information only. Different landslide mechanisms reported due to each earthquake were compiled and recorded in database.

The geographical co-ordinates of landslide locations are taken from the appropriate maps, and the distance of each from the corresponding earthquake epicentre is calculated using spherical geometry. For locations at relatively short distances from moderate to large earthquakes that can be associated with extensive fault ruptures, the use of epicentral distance can overestimate the distance of the site from the source of energy release. For this reason, following Keefer, the distance of each landslide is also measured from the fault-rupture zone. In this study, the distance has been measured from the closest point on the surface projection of the fault rupture, as used in many strong-motion attenuation studies. The surface projection of the fault in some cases has been extracted directly from individual studies, where available; in other cases, it is defined on the basis of surface ruptures, aftershock locations and focal mechanism solutions. The definition of the source-to-site distance used in this study is slightly different from that adopted by Keefer, which was to use the distance to the closest point on the fault rupture, which in many cases was a slant distance to a sub-surface point.

For each earthquake event, when the available data are sufficient, the database also includes the total area affected by landslides defined as the area that extends to the farthest observed landslides,

even if these were isolated, and the area affected by high density of landsliding defined by concentrations of observed slides.

6.1.6 Antecedent climatic conditions

Antecedent short-term (1 month) and long-term (six months and annual) rainfalls were determined, when possible, for areas where landslides were triggered for each earthquake. Based on landslides distribution the nearest pluviometric station was located and historical records retrieved. Most of the rainfall records were obtained from the world-wide data available from the NOAA Climatic Research Center (web site www.noaa.gov.us). General identification of pluviometric station used in each case is also included in database.

From these records, average values of monthly, seasonal and yearly rainfalls were calculated. In some cases, actual records preceding earthquakes were available, which were used to calculate the climatic factors used for the model development; these are: total monthly, six months, and yearly rainfalls, rainfall excess or deficits for the same periods and the event and cyclic coefficients defined Guidicini and Iwasa (1977). Excesses or deficits were defined as the difference between recorded and average values, whereas cyclic and event coefficients were defined as the percentage of total annual rainfall that fell during the previous six months and previous month, respectively. These values are also compiled in this database.

6.2 Earthquake-induced landslides database

The analysis of earthquake-induced landslides is based on the study of the slope characteristics of past slides triggered during historical earthquakes. The slide characteristics are divided into six main groups: a) identification and location of the slide, b) slope and slide morphometric characteristics, c) failure conditions, d) material properties, e) seismic parameters, and f) climatic parameters.

6.2.1 Identification and location of landslides

The initial identification of a slide is by its name that usually correspond to the nearest town or village in which the slide occurred. The location of the slide has been defined by the geographical coordinates.

In order to link both databases, the earthquake that produce each slide was included. Although all the earthquakes considered caused slides, a description of particular slides was not reported in all cases, and important information is not included in the spreadsheet. This complementary information was compiled in Rodríguez (2001).

The general description of the slide is based on the classification of the slide and the description of the material that is included within the slide mass. The material description is based on the same classification of natural materials used for the description of the regional geology in the previous section.

During earthquakes it is common to have a cluster of slides rather than a isolated slide in a particular location. The description in these cases has been based on typical characteristics of predominant landslides. The present database identified those cases of cluster events and differentiated them from individual cases. On the other hand, for model development purposes, landslides were classified into different groups according to typical failure mechanisms, as shown in Table 6.3. This classification allows the development of hazard assessment models for different geomorphologic environments.

6.2.2 Morphometric characteristics

In this study morphometric characteristics of slopes and slides have been recorded to define the most common features that earthquakes-induced slides have shown, in order to identify some slope characteristics that allow a seismic zonation of slides.

Table 6.3 Lithology classification based on failure mechanism commonly reported in each one.

Group Code	Lithology	Failure mechanisms
S1	Alluvial, deltaic, lacustrine and coastal (beach, continental) deposits	Slumps, lateral spreads.
S2	Colluvial, talus, and old landslide deposits	Slumps, disrupted slides.
S3	Residual	Disrupted slides, flows
S4	Volcanic soils including tuffs	Soil falls, disrupted slides, flows
S5	Clayey soils	Slumps, disrupted slides, flows
R1	Igneous rocks	Rock falls, disrupted slides, slumps, avalanches
R2	Metamorphic rocks	Rock falls, disrupted slides, slumps, avalanches
R3	Sedimentary rocks	Rock falls, disrupted slides, slumps, avalanches

From the field observation of slides, a group of morphological features have been defined, mainly to create new classification systems. These factor may be separated into two groups, *morphology of deposited material* and *morphology of failed slope*.

Morphological studies introduce their own specific requirements in terminology, so that the precise meaning of each measurement is known. Figure 6.4 briefly summarises the important features of landslides and some of their most commonly measured dimensions, which were proposed by the IAEG Commission on Landslides (1990) and are explained in Table 6.4.

It was pointed out that topography can influence the site effects of an earthquake, so a clear description of regional topographic setting is convenient, mainly in slope instability analysis where more specific information about the site slope topography is required. It is easy to make an intuitive comparison between the slopes of areas of low, moderate and high relief, but the detailed description and explanation of the topography is more difficult. The study of slope can be based on a single variable (slope inclination is the most common) or to be multivariate (i.e., angle, form in profile and plan).

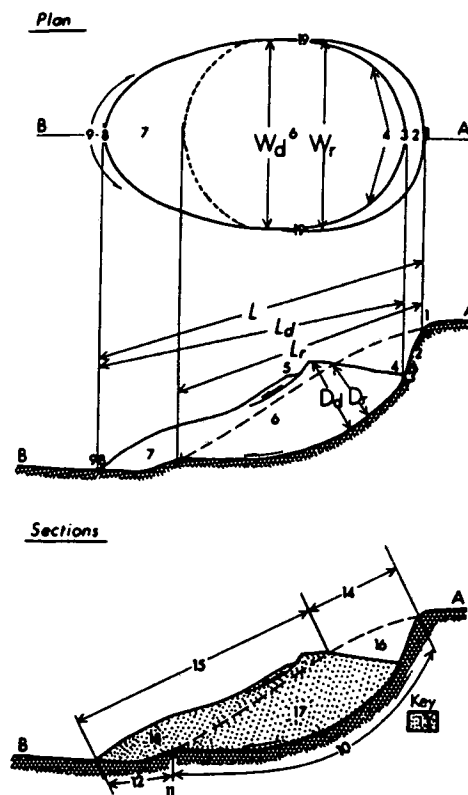


Figure 6.4 Suggested nomenclature for landslides proposed by the IAEG (1990). Terms explained in Table 6.4.

In this study a multivariate description of the slope has been used. Slope angle was not classified but actual values were reported. The slope form includes both profile form: convex, concave or rectilinear, and plan form: curvature indicated by weather lines running directly down the steepest slope would be divergent as on a spur, convergent as in a valley head or parallel. Figure 6.5 summarises the nine types of possible slopes by the plan and profile form combination (Clark and Small, 1982).

Other parameters that have been proposed for landslide description are summarised in Table 6.5. Although they were considered during data collection, for most of the cases they were very difficult to determine.

Table 6.4 Nomenclature for landslides description suggested by the IAEG Commission on landslides (1990). Numbers refer to those in Figure 6.4.

Landslide feature	Description
Crown (1)	The practically undisplaced material still in place and adjacent to the highest parts of the main scarp.
Main Scarp (2)	A step surface on the undisturbed ground at the upper edge of the landslide, caused by movement of the slide material away from the undisturbed ground.
Top (3)	The highest point of contact between the displaced material and the main scarp.
Head (4)	The upper parts of the landslide along the contact between the displaced material and the main scarp.
Minor scarp (5)	A steep surface on the displaced material of the landslide, produced by differential movements within the sliding mass.
Main body (6)	The part of the displaced material of the landslide that overlies the surface of rupture between the main scarp and the toe of the surface of rupture
Foot (7)	The position of the landslide that has moved beyond the toe of the surface of rupture and overlies the original ground surface.
Tip (8)	The point of the toe farthest from the top of the landslide.
Toe (9)	The lower, usually curved margin of the displaced material of a landslide, it is the most distant from the main scarp
Surface of rupture (10)	The projection of the main scarp surface under the displaced material of a landslide
Toe of surface of rupture (11)	The intersection (sometimes buried) between the lower part of the surface of rupture of a landslide and the original ground surface.
Surface of separation (12)	The part of the original ground surface overlain by the foot of the landslide.
Displaced material (13)	Material displaced from its original position on the slope by movement in the landslide.
Zone of depletion (14)	The area of the landslide within which the displaced material lies below the original ground surface.
Zone of accumulation (15)	The area of the landslide within which the displaced material lies above the original ground surface.
Depletion (16)	The volume bounded by the main scarp, the depleted mass and the original ground surface.
Depleted mass (17)	Part of the displaced material which overlies the rupture surface but underlies the original ground surface.
Accumulation (18)	The volume of the displaced material which lies above the original ground surface.
Landslide dimensions	
L_r	The length of the rupture surface: the distance from the toe of the surface of rupture to the crown.
L_d	Length of the displaced mass: the distance from the toe of the landslide surface to its crown.
L	Total length: the distance from the tip of the landslide to its crown
W_r	Width of the rupture surface: the maximum width between the flanks of the landslide, perpendicular to the length, L_r .
W_d	Width of the displaced mass: the maximum breadth of the displaced mass perpendicular to the length, L_d
D_r	Depth of the rupture surface: the maximum depth of the rupture surface below the original ground surface measured perpendicular to the original ground surface.
D_d	Depth of the displaced mass: the maximum depth of the displaced mass, measured perpendicular to the surface of the displaced material.

Table 6.5 Additional landslide descriptive parameters

Parameter	Description
H_s	Slope height: vertical height between the divide and the stream. Also referred to as maximum altitudinal difference.
β	Slope angle: Mean slope angle from the divide to stream; the mean angle between landslide crown and toe.
A	Area of mass movement: Surface area of the failed mass in plan.
V	Volume of slide: volume of failed mass above the failure surface.
L/D_r	Shape ratio: relation between landslide length and depth.
β_s	Slope angle at which failure takes place.
β_r	Repose angle: Slope at which the failed mass comes to rest.
S_p	Slope position: distance from the divide to the slide crown.
N_i	Inflection number: number of inflections along the slope surface. Inflections are the points of angle change between two adjacent concave and convex segments.
I_i	Inflection index: denotes the position of inflection point. Defined as the ratio between profile length above the inflection point and profile length below inflection point
α	Knick point angle: difference between angle in the knick point (inflection point) and slope angle.
R_r	Relative relief: altitudinal difference per area.

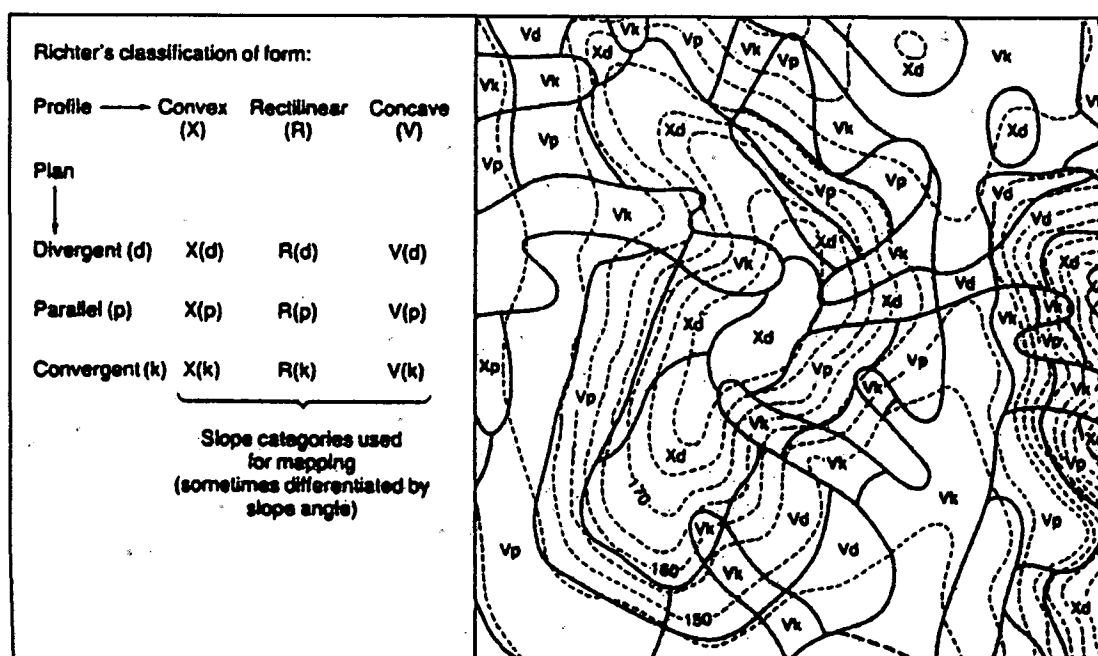


Figure 6.5 Profile and plan shape classification (Clark and Small, 1982).

6.2.3 Failure conditions

The more common term used to describe the slip surface is referred to the shape in a cross section of this surface. Some of the most general groups of slides in the Varnes classification scheme have implicit the slip surface shape, so a slump is associated with a circular or spiral surface, the translational movements usually are associated with a planar failure as well the falls, topples and wedge failures in rock deposits. The term used to describe a complex slip surface shape is applicable in those cases where more than one of the different slide types occur and in cases where the slip surface is partly controlled by structural features of the material such as bedding planes or any other structural discontinuity.

Owing to the variations in speed of movement of displacements within the same group it becomes easier and more meaningful to express rate of movement in strictly relative terms. Soil creep can be describe as extremely slow, and at the other end of the scale, the free fall of rock and debris can be extremely rapid.

The description of slide velocity has been included by Varnes in his classification, and it is shown in Table 6.6. New intervals were suggested by the International Union of Geological Sciences Working Group on Landslides (1995), which are also shown in the same table.

Table 6.6 Velocity classes suggested by the International Union of Geological Sciences Working Group on Landslides (1995).

OLD CLASSES (Varnes, 1978)		NEW CLASSES (WP/WLI, 1994)				
Velocity	Value in mm/sec	Vel Class	Description of velocity	Examples (Table 2)	Velocity limits	Value in mm/sec
3 m/sec 600 ¹⁾ 0.3 m/min 288 1.5 m/day 30 1.5 m/month 12 1.5 m/year 25 0.06 m/year	$3 \cdot 10^3$ 5 $17 \cdot 10^{-3}$ $0.6 \cdot 10^{-3}$ $48 \cdot 10^{-6}$ $1.9 \cdot 10^{-6}$	7 ----- 6 ----- 5 ----- 4 ----- 3 ----- 2 ----- 1	Extremely rapid ----- Very rapid ----- Rapid ----- Moderate ----- Slow ----- Very slow ----- Extremely slow	1-8 ²⁾ ----- 8 ²⁾ 9 ----- 10-11 ----- 12-14 ----- 15-17 ----- 18-22 ----- 23-24	5 m/sec 100 ¹⁾ 3 m/min 100 1.8 m/hour 100 13 m/month 100 1.6 m/year 100 16 mm/year	$5 \cdot 10^3$ 50 0.5 $5 \cdot 10^{-3}$ $50 \cdot 10^{-6}$ $0.5 \cdot 10^{-6}$

¹⁾ Multiplication factor between lower and higher velocity limit

²⁾ Two estimated values are available (see Table 2)

The ground water condition has been identified as one of the most important factors that contributes to slope instability. Firstly, the unit weight of the soil mass increases with the increase in water content for a given volume of soil, hence a fluctuation in water level can increase the shear stress along the failure surface, and secondly, an increase in water level produces an increase in pore water pressure within the soil mass and consequently a reduction in the soil strength.

In stability analysis, a precise determination of the water level is required in order to assess the pore water pressures along the failure surface, but this information is only available in those cases where a field investigation has been carried out. For the purposes of this work only a general definition of the position of the water level compared with the slip surface was considered. The presence of other features such as springs points and ponds were also included in the description.

Activity involved different aspects of landslides which can be grouped into three different headings: state of activity, which describes what is known about the timing of movements; distribution of activity, which describes broadly where the landslide is moving; and style of activity, which indicates how different movements within the landslide contribute to its overall movement. A glossary of descriptions for landslide activity are summarised in Table 6.7, whereas Figures 6.6 to 6.8 illustrate schematically these definitions.

Table 6.7 Descriptive terms for landslides activity (UNESCO Working Party on World Landslide Inventory (1993).

State of activity	Distribution of activity	Style of activity
Active	Retrogressing	Complex
Reactivated	Advancing	Composite
Suspended	Widening	Multiple
Inactive:	Confined	Successive
Dormant	Enlarging	Single
Abandoned	Diminishing	
Stabilised	Moving	
Relict		

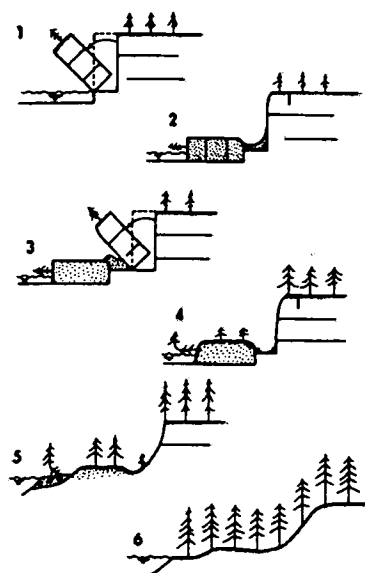


Figure 6.6 Section through topples in different states of activity. 1) Active, 2) Suspended, 3) Reactivated, 4) Dormant, 5) Abandoned, and 6) Relict. (From UNESCO Working Party on World Landslide Inventory, 1993).

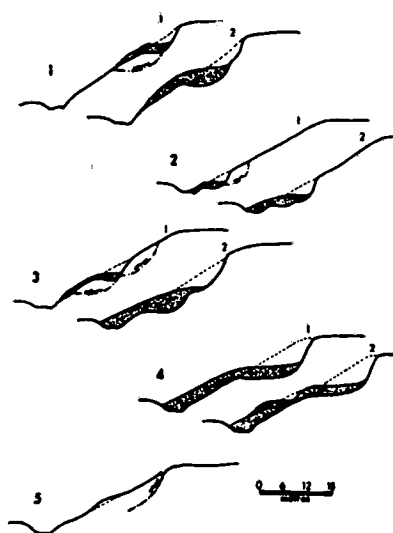


Figure 6.7 Sections through slides showing distributions of activity: 1) Advancing, 2) Retrogressing, 3) Enlarging, 4) Diminishing, and 5) Confined. Subdivisions 1 and 2 represent slide conditions prior and after the movement. (UNESCO Working Party on World Landslide Inventory, 1993).

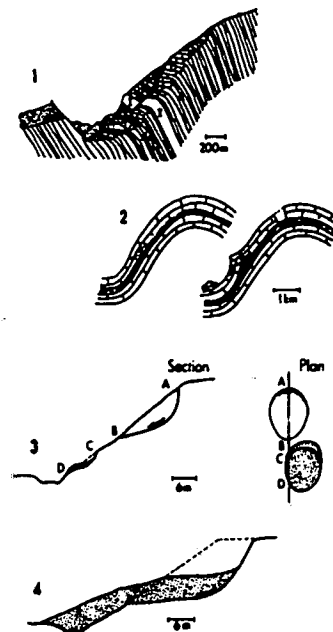


Figure 6.9 Sections through landslides showing styles of activity: 1) Complex, 2) Composite, 3) Successive, and 4) Single.
(UNESCO Working Party on World Landslide Inventory, 1993).

6.2.4 Material properties

The properties which reflect the characteristics of the moving mass are of greatest importance, but as yet there are no comprehensive classifications based on such properties. If suitable data banks can be compiled for a wide enough variety of mass movement types, then steps towards a more engineering-based classification may be achieved such as that proposed by Vaunat *et al.* (1994). The material properties have been reported only for a minor proportion of the slides studied in this report, so a comprehensive analysis of the data is not possible at this moment.

Slope material and its structure are important because they determine the tolerance of the ground to all other destabilising factors. Their combined influence is represented by the *inherent strength* of material. The greater the inherent strength, the more difficult it is for the other destabilising factors to produce failure. In engineering terms, this means that the material with greater inherent strength can more safely withstand steeper and higher cut slopes than can weaker material.

Theoretically, inherent strength can be represented by *internal friction* and *cohesion*, which are properties of the material. However, structural factors determine the extent to which these strength parameters can be mobilised, as well as determining the location, within the slope, of the material with the least resistance.

A landslide may occur so quickly that, given a fine-grained material, water has little chance to drain from the site; this type of slides are known as *short-term slide*. If one is anticipating or analysing such a failure, then the strength testing in the laboratory or the field must try to replicate these conditions by using what are referred to as *total stress* and *undrained conditions*.

Long-term slides, on the other hand, are not triggered by a rapid increase in shear stress or more commonly by an increase in pore water pressure which has the effect of reducing resistance within the slope. In this case the effect of pore water pressure generated during the test must be eliminated, or measured and subtracted from the total stress in order to obtain the *effective strength parameters*.

Material properties that were included into database are: unit weight, moisture content, liquid limit, plasticity index, fines content, saturation, specific gravity, peak and residual cohesion and friction angle of shearing resistance.

6.2.5 Seismic parameters

The intensity level in areas where particular landslides were recorded, and epicentral distance and distance to fault projection for each landslide were calculated. Using these distances and applying local or regional attenuation relationships, ground-motion parameters at the site were also calculated. Parameters included in present data set are: PGA, PGV, PGD, Arias Intensity, energy density, and duration. These parameters and their determination were explained in Chapter 2.

6.2.6 Climatic parameters

Climatic parameters calculated for different earthquakes were considered uniform throughout the total area affected by landslides. Although this assumption is not always valid, especially in mountainous areas, the lack of data for specific sites makes this simplification necessary.

Short-term and long-term parameters compiled in landslide-inducing earthquakes database were adopted for individual landslides and reported in the present data set.

6.3 Descriptive statistics of parameters included in databases

Comparison of data within each field of the databases may be carried out through descriptive statistic parameters. A description of the distribution of data should include its shape and numbers describing its centre and spread (Moore and McCabe, 1999). This section presents a brief summary of the parameters used for data description, and those values for variables included in the databases are also summarised and analysed.

6.3.1 Descriptive statistic parameters

A description of a distribution almost always includes a measure of its centre or average. The two most common measures of the centre are the *mean* and *median*. The mean is the average value, and the median is the middle value. The mean value is very sensitive to extreme values. The mean and the median of a symmetric distribution are close together. If the distribution is exactly symmetric, the mean and the median are exactly the same. In a skewed distribution, the mean is farther out in the long tail than is the median. A simpler measure of the centre is the *mode*, or normal value, which is the value that occurs most frequently in the set of data. This parameter is not always a very useful measure since any specific value may not occur more than once. For nominal data (descriptive parameters), however, the mode is the most appropriate measure of the central tendency. When dealing with nominal data, the class which contains the most items is defined as the *modal class*.

Measures of dispersion are concerned with the spread of values in a data set. The spread or variability of a distribution can be described by percentiles. The p th percentile of a distribution is the value such that p percent of the observations fall at or below it. The median is the 50th percentile. The most commonly used percentiles other than the median are the *quartiles*. The first quartile is the 25th percentile, and the third quartile is the 75th percentile. Difference between quartiles and the median gives an idea of the skewness of data. The distance between the quartiles is a simple measure of spread that gives the range covered by the middle half of data. This distance is called the *interquartile range*. This range can be used to analyse outliers.

The total range of data is the difference between the smallest and largest values in the data, range is strongly affected by outliers, which should be considered with care. A comparison of range and interquartile range gives an idea of spread of the data particularly of outliers. One disadvantage of range and interquartile range as measures of the spread is that they both take into account only a few values in the data set. They do not consider every single value. A number of possible measures of dispersion which do take into account every value are based on the deviations of the values with respect to the mean.

The *mean deviation* is the mean value of absolute deviations of values from the mean. Although this value is a very simple measure to calculate and to understand, it is not widely used, although it is used for two more common measures, the *variance* and *standard deviation*. Variance is the mean of squared deviations, and standard deviation is the square root of variance. The standard deviation measures spread by looking at how far the observations are from their mean. Standard deviation is a measure affected by outliers values. For a skewed distribution with few observations, the single long tail will have a large standard deviation, which does not give much helpful information about such a distribution.

Standard deviation is concerned with spread of values giving information with respect to dispersion but it does not provide any information about the shape of the distribution. When the median of the distribution is displaced to one side of the mean, the distribution is said to be skewed, and skewness

measures the extent to which the bulk of the values in a distribution are concentrated to one side or the other of the mean. If the bulk of the values are less than the mean, the distribution is said to be negatively skewed. If there are more values greater than the mean, the distribution is positively skewed. Symmetrical distributions have no skewness.

Kurtosis measures the extent to which values are concentrated in one part of a frequency distribution. If one class, or group of adjacent classes, in a frequency distribution contains a large proportion of all the values, then the distribution has a high degree of kurtosis, and is very peaky. A normal distribution has a kurtosis of 3.0, whereas a peaky distribution has a value greater than 3.0, and a flat distribution less than 3.0. Kurtosis is also defined as the moment coefficient of the kurtosis value minus 3.0, then peaky graphs have positive values and flat distributions have negative values. Most of the data in the database obtained during the present study are highly skewed and highly peaky, then when applied to these the mean and standard deviation may give misleading impressions, making them unsuitable for parametric tests. Moore and McCabe (1999) suggest for these cases to use median, quartiles, total range and inter-quartile range as descriptive parameters.

6.3.2 Descriptive statistics for the landslide-inducing earthquakes database

Descriptive statistics for data compiled in the landslide-inducing earthquakes are reported in Table 6.8. As pointed out by Ebdon (1985), geographic data tend to show frequency distributions different from the normal trend, and these kinds of distribution show very high skewness values. This is observed for most of the variables included into the landslide-inducing earthquakes database, except for magnitudes. Some variables, such as distance to fault projection for landslides within high concentration zone, show low skewness values but in those cases it is due to lack of data. Cyclic coefficient and total coefficients show low skewness values, but they also show low kurtosis values; this means that the data are distributed relatively uniform within the total range.

Based on this observation it seems reasonable to analyse data trends by the median and quartiles values rather than by mean and standard variation. A brief description for each variable follows, giving special attention to outliers and limit values.

6.3.2.1 Earthquake magnitude

A central distribution with large spreads can be seen for magnitudes. Worth noting are the low magnitude values that are capable of inducing landslides, depending on the susceptibility of areas affected by the earthquake. Although very small earthquakes can induced landslides it seems that a critical value of $M_S = 5.0$ can be used for hazard analysis purposes, since about 95% of data is larger than this value. Equal values of magnitude may be considered for other magnitude scales.

It looks strange to report very high values of M_S , and m_b , knowing the saturation effect on magnitude scales. This is mainly due for pre-instrumental data, for which magnitudes reported have been defined from macroseismic effects. However, only 5 to 6%, and 1 % of data lies above the M_S and m_b saturation values, respectively. Saturation values were consider as $M_S = 7.5$, and $m_b = 7.0$.

Table 6.8a Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable			
	M_S	m_b	M_W	M_L
Count	603	221	473	33
Sum	4009.17	1325.2	3175.15	207.4
Maximum	9.5	7.6	9.2	8.7
Minimum	2.9	4.6	3.5	3.3
Range	6.6	3.0	5.7	5.4
Mean	6.65	6.0	6.71	6.28
Mode	6.5	6.1	7.0	-
Median	6.7	6.0	6.8	6.5
Lower quartile	6.0	5.6	6.0	5.4
Upper quartile	7.2	6.7	7.4	7.3
Inter-quartile range	1.2	1.1	1.4	1.9
Standard deviation	0.91	0.57	0.94	1.29
Variance	0.83	0.32	0.90	1.67
Kurtosis	0.45	0.23	-0.37	-
Skewness	-0.27	-0.04	-0.27	-

6.3.2.2 Area affected by landslides

Area affected by landslides, defined by the farthest landslides reported, range between very low values to extremely high areas. As explained previously, this value does not depend on magnitude only but also on precedent climatic conditions, although extreme values are related to extreme

values of magnitude. Upper values may be consider cases where, independent of antecedent climatic conditions, the earthquake is so strong that large areas affected by landslides must be expected. This is landsliding controlled by ground-motion rather than by slope susceptibility. The opposite extreme corresponds to small areas, which are generally related to small earthquakes; in this case landsliding is controlled by susceptibility, including antecedent rainfall conditions, rather than by strong motion. Magnitude values that defined these limiting behaviours are determined in next chapter.

Area affected shows a left trend distribution or negatively skewed, which means that most of data are low values. In the case of low concentration landsliding area, 75% of data have values lower than 10,000 km², and 95% of data have values lower than 50,000 km². For high concentration of landslides area, only 25% of data have values higher than 5,000 km², and only one case has a value higher than 45,000 km². Although areas have been reported to increase with magnitude, values higher than 50,000 km² are very exceptional and this value may be taken as a upper limit for hazard analysis.

Table 6.8b Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable			
	Area affected by landslides (km ²)	Area affected by landslides (km ²)	Epicentral distance to coherent slides (km)	Epicentral distance to coherent slide (km)
	Low concentration	High concentration	Low concentration	High concentration
Count	297	50	120	3
Sum	3896863.5	444295	12737	464.5
Maximum	270000	173000	720	327
Minimum	1.5	2	2.5	7.5
Range	269998.5	172998	717.5	319.5
Mean	13120.75	8885.9	106.14	154.83
Mode	1500	17500	40	-
Median	3500	1175	70	130
Lower quartile	1000	100	30	-
Upper quartile	10000	5000	140	-
Inter-quartile range	9000	4900	110	-
Standard deviation	31813.74	25619.41	118.48	-
Variance	1012114319	656354417.4	14037.32	-
Kurtosis	35.57	35.79	8.83	-
Skewness	5.49	5.66	2.63	-

6.3.2.3 Epicentral distance

As it was found for the area affected by landsliding, maximum epicentral distance shows a negatively skewed distribution with a wide spread, although as indicated by the kurtosis and quartile values, the distribution is more uniform across the total range. Large values may also be considered as shaking controlled, whereas lower values as susceptibility controlled.

As expected, higher maximum distances are related to disrupted slides, and lower values to lateral spreads and flows. Comparison of mean values of distance to different failure mechanisms is not useful without taking into account earthquake magnitude, as was shown in Chapter 4.

Maximum epicentral distance to coherent slides shows a left distribution with 50% of data lower than 70 km, and only 5% of data are distances higher than 300 km. Values for disrupted and lateral spreads are: 50 km and 300 km, and 75 km and 350 km, respectively. Based on this, a value of 300 to 350 km can be consider as an upper limit of the epicentral distance to landslides.

Lack of data about epicentral distance to landslides within areas of major landslides concentration makes conclusions very difficult. In general, these data tend to show lower maximum distances, and kurtosis and skewness values that represent a more uniform and narrowed distribution. However, this parameter also shows a left distribution with 50% of data lower than 130 km, 30 km and 100 km for coherent, disrupted and lateral spreads, respectively. Insufficient data was available for additional analyses.

6.3.2.4 Distance to fault projection

Similarly to epicentral distances, distances to fault projection show a left distribution, although kurtosis and skewness values are lower, showing a more uniform and narrowed distribution. The ratio between total range and inter-quartile range corroborates this observation. As expected, modal values tend to be very small, and for lateral spreads this is zero, which means a high concentration of landslides within the epicentral area and particularly around and along fault projection and scarp.

As for previous parameters, higher values may be referred to as slope susceptibility controlled, whereas lower values are shaking controlled.

Maximum distance to fault projection values lie within a wide range for all different failure mechanisms, however, most of data lie in a narrow interval. For coherent slides, 50% of data are lower than 30 km and 95% of data lower than 180 km; for disrupted slides, 50% of data are less than 40 km and 0.5% lower than 200 km; finally, for lateral spreads and flows, 50% is lower than 40 km and 95% lower than 150 km. In this case it is also possible to suggest an upper limit, 200 km seems to apply for all different mechanisms.

Table 6.8c Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable			
	Epicentral distance to disrupted slides (km)	Epicentral distance to disrupted slide (km)	Epicentral distance to lateral spreads and flows (km)	Epicentral distance to lateral spreads and flows (km)
	Low concentration	High concentration	Low concentration	High concentration
Count	327	26	111	8
Sum	28904.5	2179	13520.9	1228
Maximum	900	465	620	465
Minimum	1	2	7	11.5
Range	899	463	613	453.5
Mean	88.39	83.80	121.81	153.5
Mode	20	3	15	
Median	50	30.5	75	100
Lower quartile	20	5	30	40
Upper quartile	100	95	170	150
Inter-quartile range	80	90	140	110
Standard deviation	116.38	115.92	123.32	153.97
Variance	13544.50	13438.54	15207.25	23705.92
Kurtosis	15.90	3.82	3.86	1.53
Skewness	3.49	1.97	1.86	1.46

6.3.2.5 Climatic parameters

Areas where earthquakes have induced landslides range from very dry regions to very rainy zones. Minimum precipitation values of 1.1 mm per year show that rainfalls are not a prerequisite to landslides to be triggered. As pointed out previously, rainfall influence on susceptibility and extension of damage due to landslides is controlled not only by precedent rainfall conditions but also

by earthquake shaking. These relationship were explored in order to define the model proposed in following chapters.

Table 6.8d. Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable			
	Distance to fault projection from coherent slides (km)	Distance to fault projection from coherent slide (km)	Distance to fault projection from disrupted slides (km)	Distance to fault projection from disrupted slide (km)
	Low concentration	High concentration	Low concentration	High concentration
Count	45	5	78	6
Sum	2643.9	166	4800.2	182.2
Maximum	400	95	260	95
Minimum	0	0	0	0
Range	400	95	260	95
Mean	58.75	33.2	61.54	30.37
Mode	25	-	30	0
Median	30	10	40	21.1
Lower quartile	10	-	20	0
Upper quartile	80	-	90	45
Inter-quartile range	70	-	70	45
Standard deviation	75.31	42.46	62.39	37.74
Variance	5671.27	1803.7	3892.29	1424.41
Kurtosis	9.06	-1.21	1.53	0.59
Skewness	2.62	0.94	1.45	1.09

Although the maximum annual average rainfall in the areas studied is very high, most of data lie within a relatively narrow interval, this is shown by a inter-quartile range of 1000 mm between 400 and 1400 mm that represents moderately rainy areas. Moreover, 90% of data are lower than 2000 mm. These values lead to a left distribution, which means that most of the cases are reported in semiarid to arid zones, according to the Holdridge classification, which is also confirmed by the fact that around 60% of cases were in areas with annual rainfalls lower than 1000 mm. Average six monthly and monthly rainfalls confirm the previous conclusions, although the lower the time interval the higher the kurtosis and skewness values.

Available actual rainfall data show similar trends as those for average values, and confirming the occurrence of landslides within areas between a wide range of climatic conditions, from completely dry zones, including some reporting zero previous rainfall, to very rainy areas. Inter-quartile values and the 90 percentile and 60 percentile values reported for average rainfalls are almost equal to actual values. This is an important finding considering that lack of rainfall stations or lack of actual

data for many areas makes it possible to implement the proposed models with average values rather than actual values. This may also suggest that most of the cases are related to normal rainfall conditions rather than exceptional events, which in turn validates the assumption of taking rainfall as a susceptibility factor rather than a triggering factor for earthquake-induced landslide hazard assessment purposes, and consequently extreme cases may be analysed separately.

Table 6.8e Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable			
	Distance to fault projection from lateral spreads and flows (km)	Distance to fault projection from lateral spreads and flows (km)	Average yearly rainfall (mm)	Average six months rainfall (mm)
	Low concentration	High concentration		
Count	52	5	851	785
Sum	2781.3	185	886704.53	420812.23
Maximum	230	75	5629.2	3484.0
Minimum	0	0	1.1	0.03
Range	230	75	5629.1	3483.97
Mean	53.49	37	1041.96	536.07
Mode	0	0	231.5	176.1
Median	40	50	757.2	388.2
Lower quartile	10	0	400	150
Upper quartile	80	60	1400	750
Inter-quartile range	70	60	1000	600
Standard deviation	51.58	34.92	921.80	517.28
Variance	2660.86	1220	849709.48	267579.74
Kurtosis	1.60	-2.84	2.87	4.77
Skewness	1.27	-0.31	1.58	1.87

From these descriptive statistical parameters it is not possible to define if long-term rainfalls are more important than short-term rainfalls for seismically-induced landslides, which will also be defined in the following chapters.

Rainfall excesses, defined as the difference between actual values and average values, show central distributions with mean values around zero. This shows that high precedent rainfall values do not necessarily represent the most critical conditions, and on the other hand, lower rainfall values do not mean safer conditions, confirming that a combination of seismic triggering and susceptibility may be used for hazard assessment. Negative median values indicate that most of the data are related to a deficit in rainfalls compared with average values.

Table 6.8f Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable			
	Average monthly rainfall (mm)	Actual yearly rainfall (mm)	Actual six months rainfall (mm)	Actual monthly rainfall (mm)
Count	785	248	248	248
Sum	72574.81	252206.3	121990.1	21354.3
Maximum	1138.7	6282.1	3306.0	1442.2
Minimum	0	0	0	0
Range	1138.7	6282.1	3306.0	1442.2
Mean	92.45	1016.96	491.89	86.11
Mode	0	251	366.1	0
Median	58.3	668.3	363.6	35.55
Lower quartile	10	300	150	10
Upper quartile	120	1400	650	110
Inter-quartile range	110	1100	500	100
Standard deviation	113.31	1002.30	524.30	153.34
Variance	12840.11	1004604.38	274894.94	23513.44
Kurtosis	14.75	5.46	8.32	36.12
Skewness	2.89	2.07	2.45	5.13

Table 6.8g Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable		
	Yearly rainfall excess (mm)	Six months rainfall excess (mm)	Monthly rainfall excess (mm)
Count	241	241	241
Sum	-1669.03	-3057.6	-1124.8
Maximum	2502.7	835.1	303.5
Minimum	-717.4	-460.4	-184.1
Range	3220.1	1295.5	487.6
Mean	-6.92	-12.68	-4.66
Mode	-550.7	-206.8	-0.5
Median	-29.3	-10.8	-3.4
Lower quartile	-150	-90	-25.0
Upper quartile	100	35	5.0
Inter-quartile range	250	125	30
Standard deviation	307.92	164.68	53.51
Variance	94818.19	27121.41	2863.51
Kurtosis	20.31	6.2	9.50
Skewness	2.89	1.24	1.81

The cycle coefficient shows that 50% of cases are related to seasonal rainfalls (taken as six monthly rainfalls) lower than the 50% of the annual value. This confirms that precedent climatic conditions alone do not define the hazard but that shor-term rainfalls and seismic shaking are also important in the instability process.

Most of the data are related to small event coefficients, 92% of the data lie below event values of 0.18, and 63% of data have a value lower than 0.08, which represents a uniform rainfall annual distribution and proposed by Guidicini and Iwasa (1977) as critical. For rainfall-induced landslides independent of precedent long-term rainfalls, the present data show that around 65% of cases are related to lower values, which suggests the strong influence of seismic shaking on landsliding.

Table 6.8h Descriptive statistics for data in landslide-inducing earthquakes data base

Statistic parameters	Variable		
	Cycle Coefficient	Event Coefficient	Total Coefficient
Count	241	241	241
Sum	125.55	17.32	142.86
Maximum	1	0.34	1.19
Minimum	0	0	0
Range	1	0.34	1.19
Mean	0.52	0.07	0.59
Mode	1	0	1
Median	0.51	0.06	0.61
Lower quartile	0.35	0.03	0.40
Upper quartile	0.70	0.10	0.80
Inter-quartile range	0.35	0.07	0.40
Standard deviation	0.24	0.07	0.27
Variance	0.06	0.004	0.07
Kurtosis	-0.72	1.71	-0.63
Skewness	-0.11	1.31	-0.13

6.3.3 Descriptive statistics for the earthquake-induced landslides database

Descriptive statistics for earthquake-induced landslides are presented in Table 6.9. These data also show a left trend for morphometric parameters of landslides induced by historical earthquakes. Extreme values in this case are due mainly to structural controls rather than to seismic shaking. On a regional scale identification of these structural controls is difficult, hence a regional hazard analysis may not consider those values, pointing out the necessity of making hazard analyses at different scales. The distribution of the data for particular parameters included in this data set are discussed in this section.

Data about individual landslides may be biased since reports tend to be available only for those cases that have been shown to be important, either due to their catastrophic effects or their size

causing considerable environmental changes. For this reason it is difficult to establish general conclusion about trends of morphometric and contributing parameters.

6.3.3.1 Epicentral and fault distance

Although landslides have been reported to be triggered or reactivated at large epicentral distance, as seen by the maximum distance in the data set of 670 km, most of the recorded cases lie within a range between 15 and 85 km, with a median value of 35 km. Furthermore, 95 % of data were reported at epicentral distances lower than 200 km. Epicentral distance distribution shows a left trend with a very high kurtosis and high skewness.

Distance to fault projection for cases in the present data set are concentrated into a very narrow range with only 2.5 % of data farther than 100 km. Possibly the applicability of proposed model based on these data may be restricted to epicentral distances lower than 200 km and distances to the fault projection lower than 100 km.

6.3.3.2 Strong ground-motion parameters

Landslides may be found to be related to very small seismic parameters: in the present data minimum values reported to landslide events during historical earthquakes are: PGA, 0.001 g; PGV, 0.01 m/s; PGD, 0.001 m; Arias Intensity, 0.004 m/s, energy density, $0.0003 \text{ m}^2/\text{s}^2$, and duration, 0.39 s. The opposite extreme shows very high values. It can be presumed that small values are related to very susceptible conditions, whereas high values are related to cases very near to seismic sources.

PGV distribution shows lower spread and higher kurtosis than PGA and PGD, which may suggest a better relationship of landsliding with PGV than with other parameters. However, from this analysis, no conclusions can be made.

Similarly a narrower and peakier distribution is observed for Arias Intensity than for energy density. Particularly the distribution of Arias Intensity is seen to be very concentrated around 4.5 m/s with an

inter-quartile range between 1.5 and 12.5 m/s, which also comprises the range of values suggested by Wilson and Keefer (1985) as critical values, and the range of limit values found from distances to the farthest slides in Chapter 4. Further discussion of which strong-motion parameter correlate the best with landslides is presented in Chapter 8 as a result of model development.

As discussed in Chapter 2, duration predictions were very difficult since this parameter show erratic behaviour during cases where strong motion records were available, hence for this reason analysis of this parameter is not included.

Table 6.9a Descriptive statistics for data in earthquake-induced landslides data base

Statistic parameters	Variable				
	Epicentral distance (km)	Distance to fault projection (km)	PGA (g)	PGV (m/s)	PGD (m)
Count	352	284	333	180	132
Sum	22230.47	5240	116.55	113.82	14.01
Maximum	670	380	1.34	5.23	2.05
Minimum	2.2	0	0.001	0.01	0.001
Range	667.8	380	1.34	5.22	2.04
Mean	63.15	18.45	0.35	0.63	0.11
Mode	5.6	0	0.68	0.03	0.005
Median	35	5.6	0.30	0.32	0.08
Lower quartile	15	0	0.16	0.20	0.03
Upper quartile	85	15	0.56	0.75	0.11
Inter-quartile range	70	15	0.40	0.55	0.08
Standard deviation	77.72	35.93	0.24	0.74	0.19
Variance	6041.07	1290.91	0.06	0.55	0.04
Kurtosis	15.17	38.37	0.55	11.47	88.74
Skewness	3.06	4.93	0.52	2.84	8.64

6.3.3.3 Morphometric parameters

In general morphometric characteristic distributions show left trends, which confirms that most of the cases tend to be small landslides rather than large cases. Consequently the extreme cases must be consider separately. Contrarily, slope characteristics such as slope angle, relative relief and height show a more uniform distribution. This means that hazard assessment of earthquake-induced landslides based only on morphometric parameters, without including the seismic shaking level and/or precedent rainfalls, have a higher associated uncertainty.

Table 6.9b Descriptive statistics for data in earthquake-induced landslides data base

Statistic parameters	Variable		
	Arias Intensity (m/s)	Energy Density (m ² /s ²)	Duration (s)
Count	284	108	104
Sum	5009.88	79.19	318.40
Maximum	166.11	35.73	15.61
Minimum	0.004	0.0003	0.39
Range	166.11	35.73	15.23
Mean	17.64	0.73	3.06
Mode	140.45	0.002	1.19
Median	4.48	0.19	1.74
Lower quartile	1.5	0.05	1.5
Upper quartile	12.5	0.37	3.0
Inter-quartile range	11.0	0.32	1.5
Standard deviation	38.38	3.45	3.18
Variance	1473.36	11.89	10.10
Kurtosis	8.44	101.83	6.66
Skewness	3.11	9.95	2.66

Table 6.9c Descriptive statistics for data in earthquake-induced landslides data base

Statistic parameters	Variable					
	Lr (m)	Wr (m)	Dr (m)	Hr (m)	A x10 ⁶ (m ²)	β (°)
Count	187	201	136	171	111	237
Sum	125303.1	126376.5	3353.7	28310.1	156.09	7863.19
Maximum	16700	20000	500	1600	100	90
Minimum	4.5	0.3	0.5	0.9	0.000016	0
Range	16695.5	19999.7	499.5	1599.1	99.99	90
Mean	670.07	628.74	24.66	165.55	1.41	33.17
Mode	100	500	3.0	6.0	0.3	90
Median	200	183	10.0	60.0	0.1	25
Lower quartile	50	50	4.0	15.0	0.015	12.5
Upper quartile	750	500	27.5	210.0	0.415	45.0
Inter-quartile range	700	450	23.5	195.0	0.40	32.5
Standard deviation	1539.32	1899.46	50.26	252.72	9.60	26.97
Variance	2369521.76	3607963.88	2525.90	63868.91	92.11	727.28
Kurtosis	65.92	66.17	60.44	9.56	103.91	-0.31
Skewness	7.07	7.61	6.87	2.77	10.07	0.83

For hazard assessment purposes a morphometric parameter may be used as a landslide magnitude parameter; looking at the area and volume distributions it may be suggested that these two characteristics can be used for this purpose since they are very peaky around low values, which leads to smaller variability that allows them to be used as reference parameter.

Table 6.9d Descriptive statistics for data in earthquake-induced landslides data base

Statistic parameters	Variable				
	Dr/Lr	Maximum altitudinal difference (m)	Rr (m/km ²)	Volume X10 ⁶ (m ³)	V/V _i
Count	111	51	16	160	134
Sum	12.09	9859.5	5694	4668.42	24591.58
Maximum	1	1150	970	2000	13333.33
Minimum	0.0019	2	58	0.000001	0.000003
Range	0.998	1148	912	2000	13333.33
Mean	0.11	193.32	355.87	29.18	183.52
Mode	0.1	300	400	0.3	93.77
Median	0.067	80	354.5	0.578	0.67
Lower quartile	0.025	20	150	0.01	0.02
Upper quartile	0.135	295	450	6.01	6.9
Inter-quartile range	0.11	275	300	6.0	6.88
Standard deviation	0.16	262.79	240.68	192.54	1315.03
Variance	0.02	69061.05	57930.12	37073.93	1729316.94
Kurtosis	18.92	4.37	1.43	85.41	82.20
Skewness	3.93	2.06	0.97	9.13	8.83

Table 6.9e Descriptive statistics for data in earthquake-induced landslides data base

Statistic parameters	Variable					
	Average yearly rainfall (mm)	Average six months rainfall (mm)	Average monthly rainfall (mm)	Actual yearly rainfall (mm)	Actual six months rainfall (mm)	Actual monthly rainfall (mm)
Count	379	379	379	310	310	310
Sum	414250	206651.04	33155	313479.2	160804.4	22120.4
Maximum	5629.2	2630.5	2070	4790.3	2569.3	540.9
Minimum	16.7	6.1	0.2	1.0	1.0	0.0
Range	5612.5	2624.5	2069.8	4789.3	2568.3	540.9
Mean	1093.00	545.25	87.48	1011.22	518.72	71.35
Mode	1674.9	972.8	97.1	1455.0	970.8	46.1
Median	1007.9	495.5	77.0	774.8	409.2	46.1
Lower quartile	600	200	30	500	200	40
Upper quartile	1400	750	100	1400	750	90
Inter-quartile range	800	550	70	900	550	50
Standard deviation	874.45	400.71	122.58	823.35	418.18	73.70
Variance	764670.44	160566.82	15027.97	677903.58	174878.01	5432.24
Kurtosis	8.91	2.07	181.49	7.76	1.85	13.57
Skewness	2.52	1.09	11.48	2.46	1.14	3.05

6.3.3.4 Climatic parameters

As climatic factors for individual landslides were assumed to be the same for the whole area affected by the earthquake, similar distributions to those for the landslide-inducing earthquakes database are observed, hence similar conclusions may be obtained.

Table 6.9f. Descriptive statistics for data in earthquake-induced landslides data base

Statistic parameters	Variable					
	Yearly rainfall excess (mm)	Six months rainfall excess (mm)	Monthly rainfall excess (mm)	Cycle Coefficient	Event Coefficient	Total Coefficient
Count	310	310	310	310	310	310
Sum	-27076.7	-8784.44	-2874.7	160.10	23.31	183.42
Maximum	758.3	584.7	204.3	1.0	0.34	1.065
Minimum	-573.5	-217.0	-107.1	0.07	0.0	0.07
Range	1331.8	801.7	311.4	0.93	0.34	0.99
Mean	-87.34	-28.33	-9.27	0.52	0.08	0.59
Mode	-219.9	-2.0	-51.0	0.66	0.03	0.70
Median	-61.2	-16.1	-3.2	0.56	0.06	0.62
Lower quartile	-225	-90	-45	0.35	0.04	0.40
Upper quartile	50	20	20	0.70	0.10	0.70
Inter-quartile range	275	110	65	0.35	0.06	0.30
Standard deviation	194.17	99.38	42.33	0.23	0.06	0.25
Variance	37703.39	9877.01	1791.46	0.06	0.003	0.06
Kurtosis	2.15	4.99	5.47	-0.77	1.43	-0.56
Skewness	0.087	0.86	1.26	-0.22	1.31	-0.04

CHAPTER 7

REGIONAL HAZARD ASSESSMENT

A new landslide hazard model was derived from the data compiled during the present project. This model is based on the assumption that susceptibility, for a given geological environment, can be defined by short-term and long-term climatic conditions. The triggering factor is modelled by a strong-motion parameter. Plotting these three factors in a three-dimensional space, it is possible to define a boundary surface. The plane defined by the short-term and long-term climatic factors may represent the susceptibility function, whereas the strong-motion axis may represent the triggering factor. Using values of the area affected by landslides and maximum distance from epicentre and from fault projection, subdivision of the space was possible, which divided the three-dimensional space into zones that comprises different levels of hazard, defined by these regional parameters.

These kind of boundaries classify as hazard assessment models of Grade-1, and can be used in hazard analyses with the regional characteristics of seismic areas. Total area and maximum distance to seismic sources may be used to define area likely to be affected by a design earthquake.

As will be shown, these plots are an extension of Keefer (1984) and Rodríguez *et al.* (1999) graphs that relate earthquake magnitude with area and distances. As it was discussed in Chapter 4, these relations have the disadvantage that they do not consider rainfalls in defining the upper limits established by them. The resent model considers this factor and helps to analyse the apparent outliers, from the upper limit defined by these two studies.

7.1 Model description

Model was developed from the initial hypothesis schematised in Figure 7.1. From this graph the main characteristics of the model can be described. In this Figure, Eq represents the strong-motion parameter, whereas R and r represent the long-term and short-term rainfall parameters, respectively.

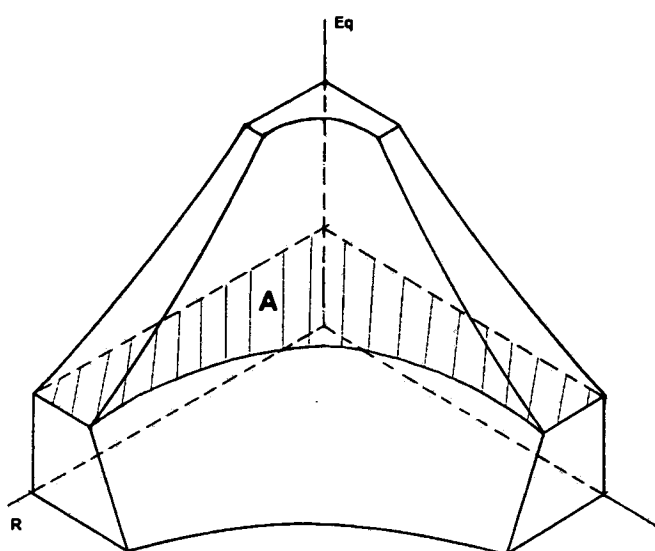


Figure 7.1 Schematic hypothesis of surface boundary shape

Threshold values may be established for the strong-motion, (Eq), long-term, (R) and short-term, (r) climatic parameters. Each of these values represent critical levels above which the other two factors are not necessary for landslides to be triggered. When seismic shaking is not considered into the analysis, this is projection on plane (R , r), these critical values must correspond to critical rainfall levels for rainfall-induced landslides. Determination of these levels was not within the scope of the present project. Similar values of critical rainfalls may be established for different earthquake shaking levels, as suggested by plane A. A critical value of the seismic parameter may also be determine above which climatic conditions are not important in landslides triggering.

Taking the seismic shaking intensity to be constant, the higher the long-term precedent climatic factor, the lower is the short-term climatic factor required to induce landslides. Similarly, the higher the short-term climatic factor, the lower the long-term climatic factor for landslides to be triggered.

Maintaining the long-term climatic factor constant, the larger the short-term climatic factor, the lower the earthquake shaking necessary to induced landslides. On the other hand, the lower the short-term climatic factor, the higher the earthquake shaking to induce landslides. Finally, maintaining the short-term factor constant, the larger the long-term climatic factor, the lower the earthquake shaking required, and the lower the long-term climatic factor the higher the earthquake shaking necessary to induce landslides. Curvilinear boundaries were expected based on observation of boundaries proposed by different rainfall-induced landslides assessment models, which were summarised in Chapter 3.

Although the expected general trend was confirmed by data, some variations were found. The general characteristics of the model obtained by the present data can be seen in Figure 7.2. As rainfall data are taken as climatic factors, short-term rainfall has a physical constraint, the maximum possible value that this factor can take is the long-term factor value. Then, a new limit may be established for the (R, r) planes, this is a line with a maximum inclination angle of 45 degrees. However, the data suggested that this line has an inclination of around 17° with respect to horizontal axis (r) when annual rainfall is taken as the long-term value, 38° when six months rainfall is used, and 45 degrees when event and cycle coefficients are used as climatic parameters.

Threshold boundaries of rainfalls were confirmed but no curvilinear relationship, as supposed, was found. Instead a bilinear boundary seems to be the most appropriate. Finally, it was possible to determine threshold values of seismic shaking, in this case defined by earthquake magnitude.

7.2 Model development

The method used to develop the model was based on the plotting of data on the three planes that generate the three-dimensional space. As an example of the model development, the procedure will be described for the case where earthquake surface-wave magnitude (M_s) was used as the seismic parameter (Eq), and monthly and annual rainfalls were used as the short-term (r) and long-term (R) climatic parameters.

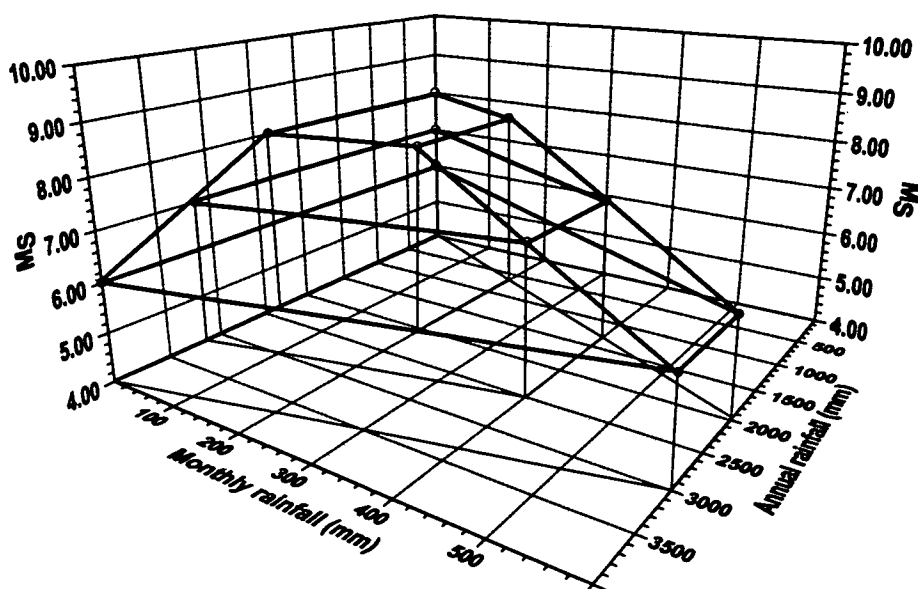


Figure 7.2 Surface boundaries for area affected by landslides

Projection of points, corresponding to the whole dataset, on the short-term vs. long-term climatic factors plane is shown in Figure 7.3. In this graph, the limits that enclose cases of earthquakes causing different levels of areas affected by landslides are shown. Levels of areas affected by landslides correspond to the first and third quartiles, which represent the 25 percentile and 75 percentile. As determined in previous chapter these values are 1000 km^2 and 10000 km^2 , respectively.

The boundaries shown in Figure 7.3 were established from graphs 7.4 to 7.6, which represent those points within the three different intervals defined by the quartile values of areas affected by landslides.

Threshold values and limits obtained from these graphs are summarised in Table 7.1. Values refer with subscripts 1 enclose cases with areas larger than 10000 km^2 , those referred with subscripts 2, enclose cases with areas between 1000 km^2 and 10000 km^2 , whereas subscripts 3 refer to cases with areas smaller than 1000 km^2 .

ALL DATA

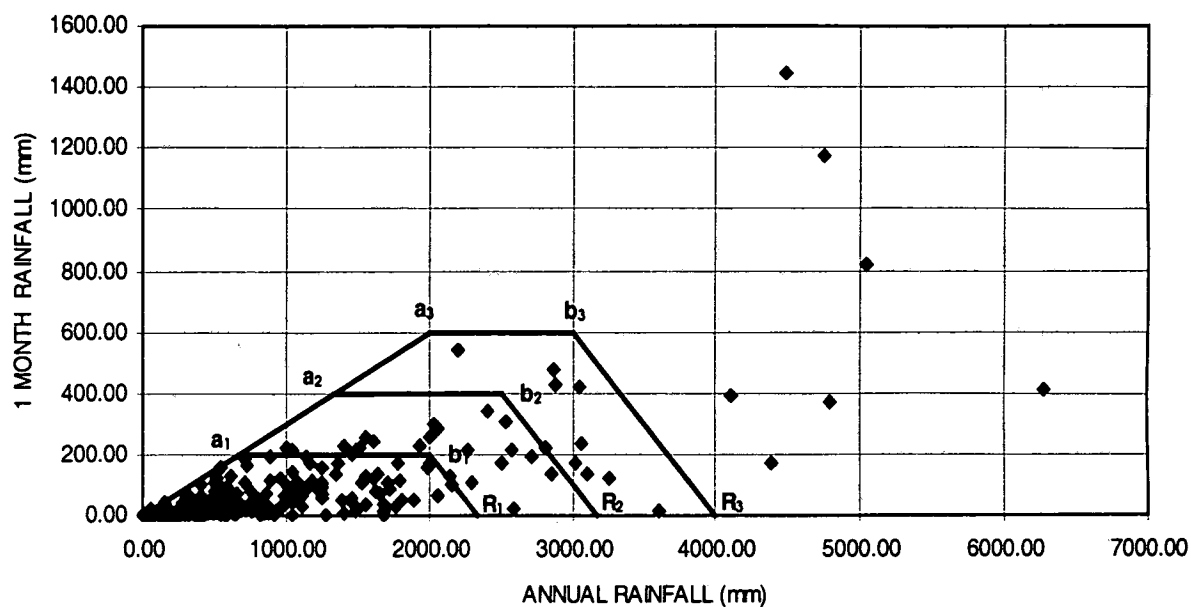


Figure 7.3 Annual and monthly rainfalls for areas affected by earthquake-induced landslides. Limits correspond to regions enclosing cases affecting areas lower than 1000 km^2 (external), between $1,000 \text{ km}^2$ and $10,000 \text{ km}^2$ (middle), and larger than $10,000 \text{ km}^2$ (internal).

Table 7.1 Coordinates of points that define the different areas established from Figures 7.4 to 7.6. Subscript 1 refers to region enclosing cases where area was larger than 10000 km^2 , subscript 2 refers to region enclosing cases with areas between 1000 km^2 and 10000 km^2 , and subscript 3 refers to region enclosing cases with areas smaller than 1000 km^2 . Points are named according to Figure 7.3.

Reference points	Coordinates (R, r, Eq) (mm, mm, M_s)
a_1	(666.7, 200, 8.0)
a_2	(1333.3, 400, 7.0)
a_3	(2000, 600, 6.0)
b_1	(2000, 200, 8.0)
b_2	(2500, 400, 7.0)
b_3	(3000, 600, 6.0)
c_1	(0.0, 0.0, 8.0)
c_2	(0.0, 0.0, 7.0)
c_3	(0.0, 0.0, 6.0)
R_1	(2333.3, 0, 8.0)
R_2	(3133.3, 7.0)
R_3	(4000, 0, 6.0)

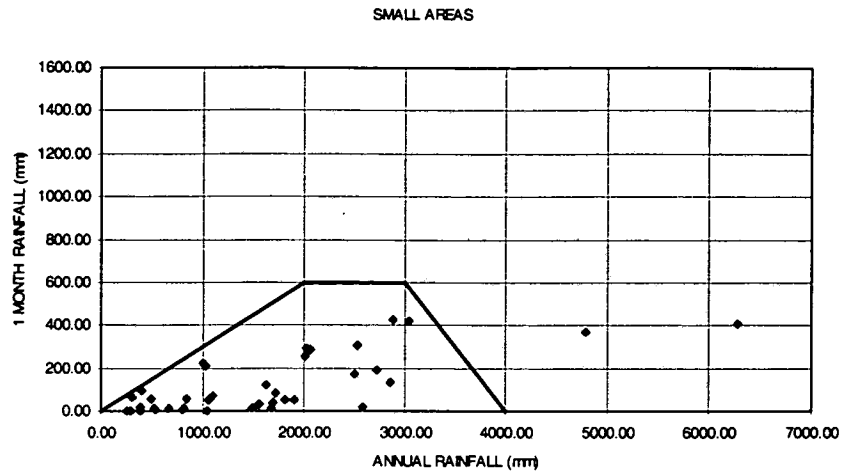


Figure 7.4 Annual and monthly rainfalls for earthquakes that affected areas lower than 1000 km² by landsliding. Boundary shown was used to define reference points in Table 7.1.

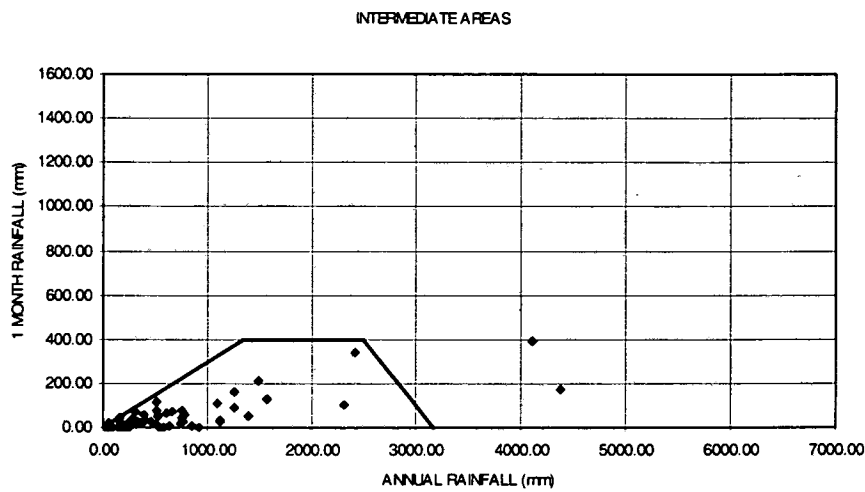


Figure 7.5 Annual and monthly rainfalls for earthquakes that affected areas between 1000 km² and 10000 km² by landsliding. Boundary shown was used to define reference points in Table 7.1.

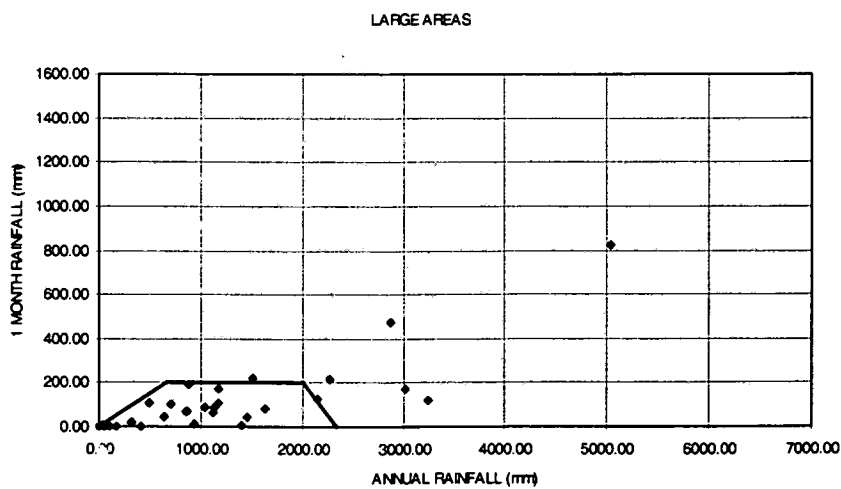


Figure 7.6 Annual and monthly rainfalls for earthquakes that affected areas larger than 10000 km² by landsliding. Boundary shown was used to define reference points in Table 7.1.

During the definition of the single boundary for the small areas, two cases lie outside of these boundaries, representing 5% of data; for intermediate areas, two cases were also excluded, representing 3.8% of data; and finally, for large areas 5 cases were excluded, constituting 16.7% of data. This means that in total 9 cases of 121 were excluded during the boundaries definition, which represent 7.4 % of data. There are another two cases that lie outside of the proposed boundaries, which could not be classified due to lack of information about extension of the area affected.

A similar procedure was followed in order to define boundaries enclosing data on the other two planes. In these cases, points defined on the (R, r) plane were taken into account to fix them as reference points.

A plot of data on the plane (R, Eq) is presented in Figure 7.7. Reference points for this graphs are values of the annual rainfalls, R_1 , R_2 , and R_3 defined in Figure 7.3. Projections of the other points that defined the surface boundary are also plotted. The lines that link these points are the projection of the surface boundaries on the (R, Eq) plane.

Data were divided into the three different groups based on the area affected by landslides. Figures 7.8 to 7.10 show the boundaries established for each group. Earthquakes affecting areas smaller than 1000 km^2 by landslides are commonly cases with M_s values between 5.0 and 7.0, although it is assumed that earthquakes with lower magnitude also produce small areas of landsliding. Intermediate areas are commonly produced by earthquakes with M_s between 6.0 and 8.0, and large areas are caused by earthquakes with M_s values higher than 7.0.

When defining the boundaries for small areas, only 5 points lie outside of these boundaries, representing 12.8 % of the data; for intermediate areas, 7 points were excluded, which represent 13.5%, and finally, boundaries of large areas excluded 10 cases representing 40% of the data. In general, 22 cases were excluded during the model development, which represent 18.8% of the total data.

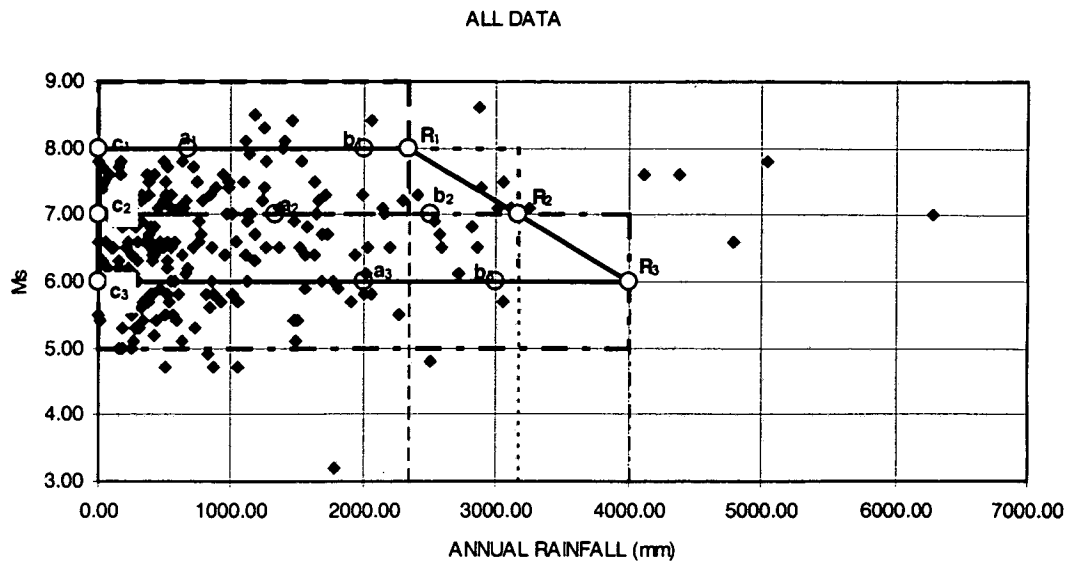


Figure 7.7 Annual rainfalls and surface wave magnitude for different areas affected by earthquake-induced landslides. Solid lines represent the external limit of the surface. Dashed lines delimit magnitude and annual rainfall ranges for regions enclosing cases affecting areas lower than 1000 km² (lower rectangle), between 1000 km² and 10000 km² (middle rectangle), and larger than 10000 km² (upper rectangle), these areas are shown separately in Figures 7.8 to 7.10. Reference points shown are those presented in Table 7.1.

Magnitude values of 6.0, 7.0 and 8.0 were taken as critical values that separate the three-dimensional space into four zones enclosing cases that produced different levels of areas affected by landslides. Earthquakes with magnitude lower than 6.0 produce affected areas smaller than 1000 km²; the opposite extreme are earthquakes with magnitude larger than 8.0, which produce affected areas larger than 10000 km². Earthquakes with magnitude between 6.0 and 7.0 may affect areas ranging between small and intermediate, that is between 1000 km² and 3500 km², and earthquakes with magnitudes between 7.0 and 8.0 may produce areas between 3500 km² and 10000 km². A value of 3500 km², assumed to divide intermediate zone, is the median or 50th percentile of the data.

Critical magnitude values define reference points c_1 , c_2 , and c_3 ; for the present example these points are also reported in Table 7.1. Having all these points, the three-dimensional surface may be plotted as shown in Figure 7.2.

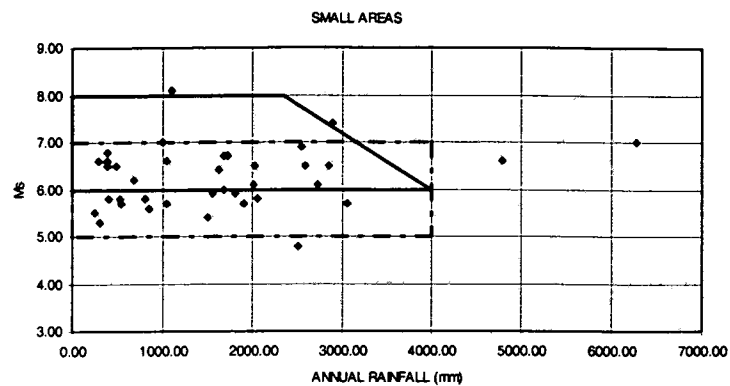


Figure 7.8 Annual rainfalls and surface-wave magnitude for earthquakes that affected areas lower than 1000 km^2 by landsliding. Solid lines represent the external boundary of the surface. Dashed lines delimit magnitude and annual rainfall ranges.

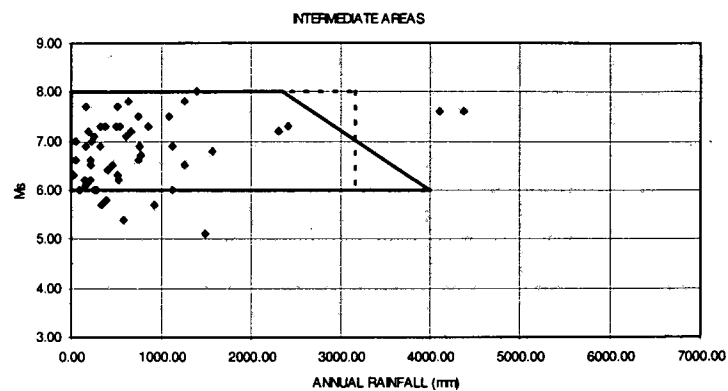


Figure 7.9 Actual annual rainfalls and surface wave magnitude for earthquakes that affected areas between 1000 km^2 and 10000 km^2 by landsliding. Solid lines represent the external boundary of the surface. Dashed lines delimit magnitude and annual rainfall ranges.

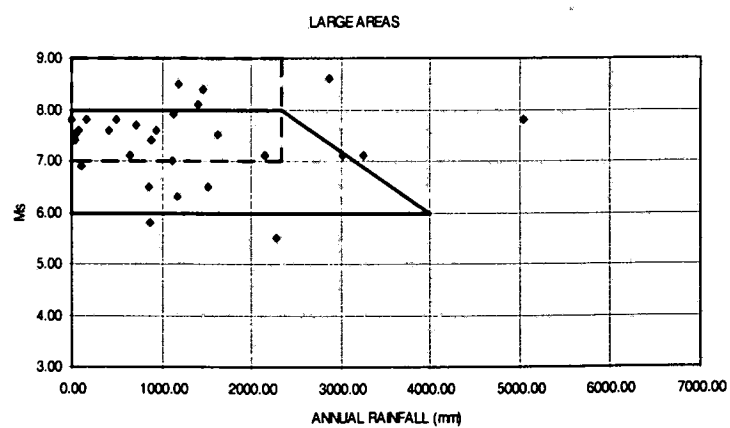


Figure 7.10 Actual annual rainfalls and surface wave magnitude for earthquakes that affected areas larger than 10000 km^2 by landsliding. Solid lines represent the external boundary of the surface. Dashed lines delimit magnitude and annual rainfall ranges.

Projection of data on the (r, E_q) plane is shown in Figure 7.11, confirming the critical values of earthquake magnitudes that separate different zones.

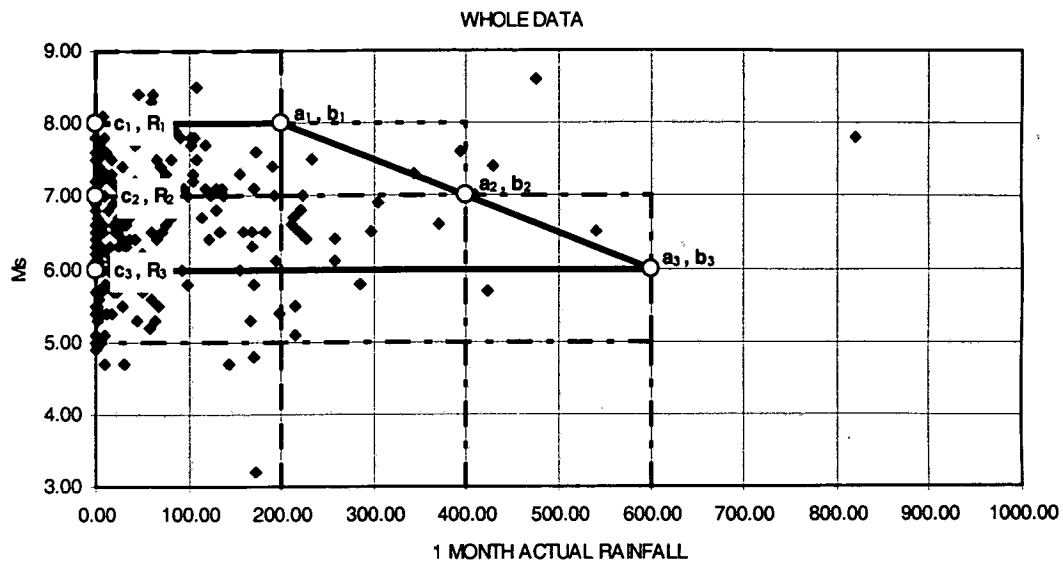


Figure 7.11 Monthly rainfalls and surface-wave magnitude for areas affected by earthquake-induced landslides. Solid lines represent the external limit of the surface. Dashed lines delimit magnitude and monthly rainfall ranges for regions enclosing cases affecting areas lower than 1000 km^2 (lower rectangle), between 1000 km^2 and 10000 km^2 (middle rectangle), and larger than 10000 km^2 (upper rectangle), these areas are shown separately in Figures 7.12 to 7.14. Reference points shown are those presented in Table 7.1.

Definition of boundaries on the (r, E_q) plane excluded 3 points for the small areas zone, which represent 7.7% of data; 6 cases were excluded from the intermediate zone corresponding to 11.3% of data; and 7 points were excluded from the large areas zone, which is 23.3% of the data. In total, 13.2% of the data were excluded during the definition of the boundaries.

The total number of points that do not follow the behaviour proposed by the model was 22, these are compiled in Table 7.2. From these 22 cases, 5 are cases related to annual rainfalls larger than 4000 mm; as expected from these rainfall levels, they were in tropical areas. These cases may be considered outliers due to the excessive rainfalls associated to them. For instance, one of them is also related to a monthly rainfall larger than 800 mm; this is the 1990 Luzon earthquake, which is related with rainfalls that reached an annual excess of 1900 mm, and a monthly excess of around 160 mm. Moreover, this earthquake produced extensive landsliding around the fault trace, which had a length of about 80 to 100 km.

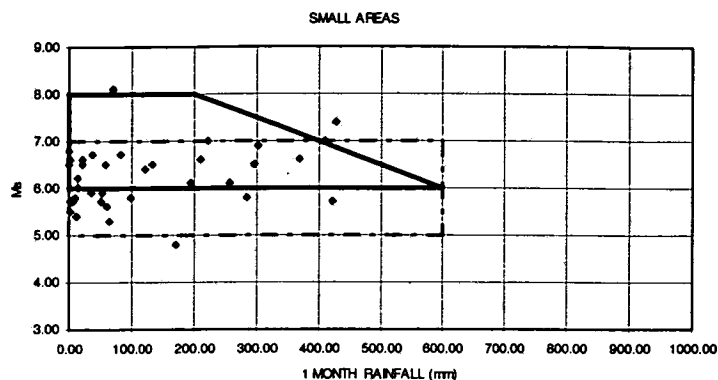


Figure 7.12 Monthly rainfalls and surface-wave magnitude for earthquakes that affected areas lower than 1000 km² by landsliding. Solid lines represent the external boundary of the surface. Dashed lines delimit magnitude and annual rainfall ranges.

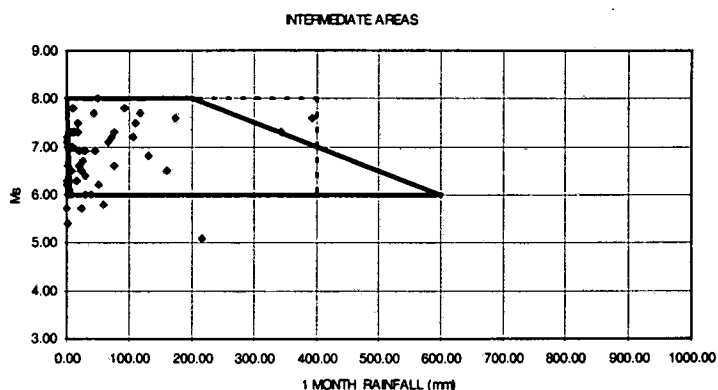


Figure 7.13 Actual monthly rainfalls and surface wave magnitude for earthquakes that affected areas between 1000 km² and 10000 km² by landsliding. Solid lines represent the external boundary of the surface. Dashed lines delimit magnitude and annual rainfall ranges.

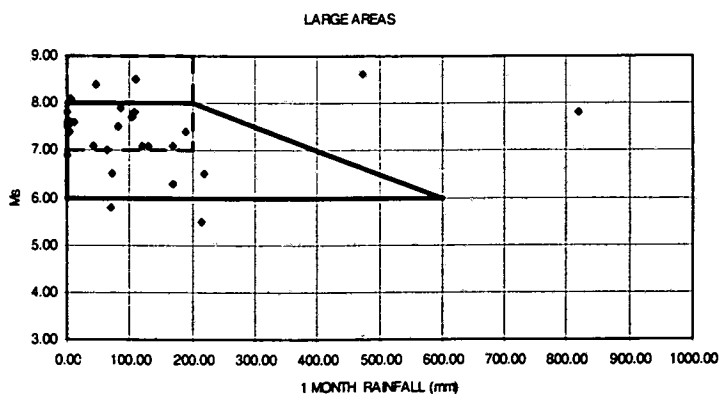


Figure 7.14 Actual monthly rainfalls and surface wave magnitude for earthquakes that affected areas larger than 10000 km² by landsliding. Solid lines represent the external boundary of the surface. Dashed lines delimit magnitude and annual rainfall ranges.

Table 7.2 Earthquakes discussed as outliers from boundaries proposed in the present model

Date	Country	M _S	M _W	Annual rainfall (mm)	Six months rainfall (mm)	Monthly rainfall (mm)	Annual excess (mm)	Six months excess (mm)	Monthly excess (mm)
1843-02-08	Antigua/Guadalupe	8.3	7.8	1247.3	653.3	59.3	-511.90	-353.30	-77.10
1906-04-18	USA/California	7.7	7.9	712.1	611.1	103.2	92.20	68.00	41.90
1907-01-14	Jamaica	6.5	-	2850.9	1802.3	132.9	592.60	346.50	-36.70
1914-05-28	Panama	6.4	-	1556.3	366.1	256.8	-550.70	-153.20	17.50
1927-11-04	USA/California	7.3	-	506.1	89.6	75.9	116.20	58.50	60.10
1929-03-09	New Zealand	7.1	-	3013.7	1467.3	169.7	168.60	4.70	-32.70
1934-01-15	India/Nepal	8.1	7.99	1400.5	679.0	6.9	-46.50	-139.70	0.70
1934-07-18	Panama/Costa Rica	7.5	7.59	3057.0	684.6	233.0	695.30	73.30	-4.60
1941-12-05	Costa Rica	7.6	7.35	4110.8	2854.8	392.6	-717.40	-376.00	-129.80
1946-06-23	Canada	7.10	7.56	3252.6	2104.6	118.4	2502.70	700.10	15.50
1948-06-28	Japan	7.1	6.97	1351.9	754.7	134.7	-587.20	-150.60	-122.30
1949-04-13	USA/Washington	7.0	6.5	1117.7	630.9	63.0	109.80	-101.10	-25.60
1950-08-03	Venezuela	6.9	6.8	1115.0	646.1	29.7	257.90	264.60	-14.90
1950-08-15	India	8.6	8.61	2868.5	1862.5	474.0	172.60	-26.00	-11.40
1958-07-10	USA/Alaska	7.9	7.78	1134.5	349.1	85.6	-204.40	-93.30	9.00
1959-08-18	USA/Montana	7.5	7.28	747.5	437.5	17.5	75.50	105.90	-24.70
1960-05-22	Chile	8.5	-	1181.3	333.7	109.3	-149.30	-154.40	-69.80
1961-04-14	China	6.5	6.8	1361.4	608.8	168.5	-246.50	45.30	-29.20
1964-03-28	USA/Alaska	8.4	9.2	1455.0	970.8	46.1	-219.90	-2.00	-51.00
1966-06-28	USA/California	5.5	5.3	255.4	51.9	2.5	-62.60	-167.90	1.70
1966-10-17	Peru	7.8	8.16	1.0	1.0	0.0	-15.70	-11.40	-1.50
1967-02-09	Colombia	7.1	7.16	2148.8	1499.9	129.0	114.50	414.90	-26.40
1967-07-30	Venezuela	6.3	6.5	1722.0	941.0	82.0	-176.80	21.10	-11.30
1968-02-19	Greece	7.5	7.18	503.8	351.5	108.9	-3.00	-10.60	35.60
1968-04-09	USA/California	6.5	6.5	860.6	625.2	71.9	-90.60	-162.00	-72.90
1968-05-16	Japan	8.10	8.26	1099.6	300.7	71.1	-28.50	-159.80	0.10
1968-05-23	New Zealand	7.4	7.16	2883.7	1413.3	428.7	38.60	14.10	185.50
1968-08-31	Iran	7.1	7.25	251.0	130.0	0.0	3.90	-8.70	-0.60
1970-09-26	Colombia	6.6	-	4790.3	2569.3	370.1	-61.20	-61.20	-107.10
1974-03-06	Nicaragua	5.7	-	1678.0	718.0	0.0	416.00	213.70	-0.80
1976-07-11	Panama	7.0	-	6282.1	1660.9	409.8	1430.60	-162.80	-50.90
1978-08-13	USA/California	5.7	5.7	917.2	458.6	0.2	496.50	292.30	-1.60
1979-08-06	USA/California	5.7	5.7	534.9	191.7	5.7	-9.80	-27.40	4.60
1979-10-15	USA/California	6.9	6.5	111.2	23.7	0.0	33.80	-1.50	-7.10
1979-11-23	Colombia	6.5	7.2	1515.7	909.8	219.2	-168.10	78.20	44.80
1980-01-24	USA/California	5.8	-	404.0	220.7	98.1	29.6	24.5	23.7
1981-04-26	USA/California	6.0	5.9	88.6	68.6	2.6	11.20	17.80	-1.00
1981-10-18	Venezuela	5.5	5.9	2267.0	1415.4	214.4	550.70	354.60	-22.40
1983-03-31	Colombia	5.1	5.6	1486.0	1084.0	215.0	169.30	174.60	89.20
1983-04-03	Costa Rica/Panama	7.2	7.47	2302.4	726.2	105.5	-259.30	-36.20	44.00
1983-10-28	USA/Idaho	7.3	6.9	372.8	213.4	17.4	120.50	67.50	1.50
1987-01-07	China	5.4	5.3	585.4	329.8	1.9	-68.40	-51.20	-1.40
1987-10-01	USA/California	6.0	5.9	257.7	27.8	6.9	-261.00	-39.80	-5.30
1988-08-20	Nepal	6.5	6.8	2197.2	1467.1	540.9	639.30	343.50	204.30
1988-11-25	Canada	5.8	5.8	869.4	525.0	70.0	-69.20	-28.60	-7.00
1989-04-30	Venezuela	6.0	5.9	1114.9	393.0	28.9	6.00	-110.70	-73.60
1989-10-18	China	5.8	5.6	393.7	371.9	58.8	22.70	32.80	29.40
1989-10-18	USA/California	7.1	6.8	643.4	67.6	42.2	-91.20	-16.10	16.70
1990-03-25	Costa Rica	7.1	7.3	3103.5	1309.6	134.9	237.20	137.00	61.20
1990-07-16	Philippines	7.8	7.6	5042.7	1770.5	819.7	1189.40	379.30	159.90
1990-10-20	China	5.7	5.7	334.4	281.3	22.5	-76.90	-86.50	-9.70
1991-04-22	Costa Rica	6.9	6.9	4382.5	1628.3	172.3	-492.80	-70.10	-56.30
1993-07-10	Costa Rica	5.7	5.8	3050.0	1554.7	422.6	319.00	526.00	112.20
1993-09-21	USA/California-Oregon	5.8	5.9	521.7	176.6	10.7	165.70	65.20	-3.20
1994-06-18	New Zealand	6.9	6.8	2538.2	1470.2	303.4	-306.90	90.80	68.00
1995-05-29	New Zealand	4.8	5.5	2501.3	1021.2	170.8	-343.80	-374.90	-74.90

The other four cases correspond to the 1941 and 1991 Costa Rica, 1970 Colombia, and 1976 Panamá earthquakes. It is worth noting that although these earthquakes are related to high

precedent annual rainfalls, the areas affected by them range between small and very large. The Panamá and Colombia earthquakes produced small areas of landslides, which may be explained by the fact that although they have high annual rainfalls, actual values are lower than the average value, that is that they actually are related to dry conditions compared to normal. This reinforces the importance of short-term rainfalls for landsliding in tropical environments. They are also related to moderate earthquakes of magnitudes 6.1 and 6.6, respectively. Based on critical values of magnitude defined previously, the areas produced by these earthquakes would be expected to lie within the lower intermediate range.

The Costa Rican earthquakes both have a magnitude of 7.6, and they are related to rainfall deficits. In spite of their high magnitude, the area affected by landslides was constrained by rainfall deficit compared with average values. These cases would be expected to classify within the upper intermediate zone, but the actual areas are within the lower intermediate zone. This result also confirms the importance of short-term rainfalls for landsliding in tropical areas.

Analysis of these five points would suggest that for tropical areas, the total area affected by landslides during seismic events is a function not only of the total long- and short-term rainfalls but also of the rainfall excess or deficit with respect to normal conditions. For these cases, the use of the critical magnitude to define potential area affected by landslides gives reasonable results if the analysis takes into account the short-term rainfall excess.

The 1995 Arthur's Pass earthquake in New Zealand lies outside the of boundaries which were used to define the surface; however, this case affected a small area, which should be expected given the earthquake magnitude. Consequently, this case actually lies within the boundaries established by the model, as it was considered that earthquakes with magnitudes lower than 5.0 produce small areas.

Two large earthquakes producing small areas were also excluded, these are the 1968 Japan and the 1968 New Zealand earthquakes. For these earthquakes there is no apparent reason to cause such small effects; in contrast the model would predict high to very high areas for them. The

Japanese earthquake is related to annual rainfall deficit but normal monthly rainfalls, which suggest high short-term and medium-term rainfalls. On the other hand, the New Zealand earthquake was related to annual rainfall excess and very large monthly rainfall excess.

Most of the points excluded during the analysis correspond to cases that lie in the intermediate zone, where more scatter of the data is observed, as can be seen from Figures 7.8 to 7.10. Some cases of earthquakes with magnitudes between 5.0 and 6.0 produce intermediate areas, a zone which was considered to enclose cases of small areas. Similarly, some cases that produced large areas were excluded because they lie in zone considered to enclose intermediate areas, defined by magnitudes between 6.0 and 7.0.

Cases producing intermediate areas and classified within the small area zone are the 1987, 1989, and 1990 China earthquakes, and the 1978 California and the 1983 Colombia earthquakes. The Chinese cases are earthquakes that produced landsliding on loess deposits, and are related to normal rainfall conditions in the 1989 case, and to rainfall deficits for the 1987 and 1990 cases. Although these were moderate magnitude earthquakes occurring in normal or dry conditions they produced extensive areas affected by landslides apparently due to the fact that they affected very susceptible materials. The Colombian earthquake triggered landslides in volcanic, residual and alluvial deposits, and although the magnitude was relatively low ($M_S=5.1$), it occurred in a very wet season associated with large annual rainfall excess and monthly rainfall excess. As stated previously, the susceptibility of residual soils increases dramatically with high short-term rainfalls. Finally, the 1978 California earthquake, with a moderate magnitude ($M_S=5.7$), was related to high annual rainfall excess and normal monthly rainfall conditions. This earthquake affected alluvial, colluvial and artificial fills, which are very susceptible under high water levels that may be controlled by excessive long-term rainfalls.

Those cases that produced large areas but lie within the intermediate and small areas zones are the 1968 and 1979 California earthquakes, the 1967 Venezuela and the 1979 Colombia earthquakes. The 1968 California earthquake was related to dry conditions, defined by annual and monthly rainfall deficit. This case produced slides on alluvial, coastal, lacustrine, fractured metamorphic, and

sedimentary rock deposits. The 1979 California earthquake was related to almost normal rainfall conditions, and produced slides in similar deposits to the 1968 case. Both cases were located in the same area, and actual areas may be greater due to lack of information of damage beyond the Mexico border. There is no evident reasons for these cases to produce such large areas.

The 1967 Venezuela and the 1979 Colombia earthquakes produced landslides in residual soils, and extension of areas affected seems to be controlled by large monthly rainfall excess observed in both cases, in spite of they being related to annual rainfall deficits. The area of high concentration of landslides is markedly smaller than the total area affected by landslides, in both cases this being around 30 to 40% of the total area. However, these high concentration areas also lie within the large areas range.

Two relatively small earthquakes triggered landslides over large areas, which were the 1981 Venezuela earthquake and the 1988 Canada earthquake. The Canadian case was discussed previously, and it was found that this case triggered landslides on very sensitive clays. On the other hand, the Venezuelan earthquake was related to very high annual and monthly rainfall excess, additionally, it caused damage in a region where residual soils and fractured sedimentary rocks dominate, which are very susceptible, especially under high short-term rainfalls.

There exist another three cases that were not included within the boundaries used to define the model surface. These are the 1950 India, the 1946 Canada and the 1929 New Zealand earthquakes. Although the Indian earthquake lies outside the proposed boundaries, area affected by landslides during that earthquake was very large, as could be predicted by the model. This case was related to high annual rainfall excess that, as shown in other cases seems, to influence landsliding in alluvial deposits, which were the most affected deposits during this earthquake.

The 1946 Canadian earthquake produced landslides across a very large area, but high concentration of landslides were present in an intermediate area. Although this case also lies outside the boundaries, the model would correctly classify the area affected by the earthquake from the earthquake magnitude. In this case, the excessive annual and monthly rainfalls seem to be the

cause of the point to lie outside. Finally, the 1929 New Zealand earthquake also produced a large area, which is congruent with its magnitude but lies out of the boundaries due to excessive annual rainfalls.

Summarising from the 22 cases that were not included within the boundaries used to define the present model, only six are impossible to explain. From the other cases, some lie outside the boundaries due to excessive rainfalls, either long-term or short-term, others lie outside due to earthquakes occurring in zones covered by very susceptible deposits, reporting higher areas than expected. Although it is observed from the previous analysis that short-term and long-term rainfalls controlled the extension of damage due to landslides, it was also found that total rainfall values are not enough to explain the slope behaviour and the rainfall deficit also need to be considered.

7.3 Results

A similar procedure to that explained in the previous section was used to define the boundaries within which historical cases lie, in order to define surfaces similar to that shown in Figure 7.2 when using different parameters. The earthquake parameter was defined by the surface wave magnitude and the moment magnitude. The short-term rainfall was defined by the monthly rainfall, the monthly rainfall excess and the event coefficient, and finally, the long-term rainfall parameter was defined by the annual rainfall, the six months rainfall, the cycle coefficient, the annual rainfall excess, and the six months rainfall excess

7.3.1 Area affected by landslides

Results for area affected by landslides were presented in the previous section for the description of the model development. In that case results referred to data with annual and monthly rainfalls and M_s . For other cases results are presented in Table 7.3. Correlations with cyclic coefficient and event coefficient were very poor, hence no attempt was made to define the surface using these parameters.

Table 7.3 Coordinates of points, in three-dimensional space (R, r, M), that define the boundary surface for area affected by landslides. Subscript 1 refers to region enclosing cases where area was larger than 10000 km², subscript 2 refers to region enclosing cases with areas between 1000 km² and 10000 km², and subscript 3 refers to region enclosing cases with areas smaller than 1000 km². Points are named according to Figure 7.3.

Reference points	Coordinates (R, r, M)				
	Ann. Rainfall Monthly Rainfall M_s and M_w	Six Mon. Rainfall Monthly Rainfall M_s and M_w	CC EC M_s and M_w	Ann. Rainf. Excess Mont. Rainf. Excess M_s and M_w	Six Mont. Rainf. Excess Mont. Rainf. Excess M_s and M_w
a₁	(666.7, 200, 8.0)	(250,200,8.0)	-	(-500,100,8.0)	(-440,100,8.0)
a₂	(1333.3, 400, 7.0)	(500,400,7.0)	-	(-550,150,7.0)	(-420,150,7.0)
a₃	(2000, 600, 6.0)	(750,600,6.0)	-	(-500,200,6.0)	(-400,200,6.0)
b₁	(2000, 200, 8.0)	(1000,200,8.0)	-	(250,100,8.0)	(400,100,8.0)
b₂	(2500, 400, 7.0)	(1250,400,7.0)	-	(375,150,7.0)	(500,150,7.0)
b₃	(3000, 600, 6.0)	(1500,600,6.0)	-	(500,200,6.0)	(600,200,6.0)
c₁	(0.0, 0.0, 8.0)	(0,0,8.0)	-	(-1000,-300,8.0)	(-600,-300,8.0)
c₂	(0.0, 0.0, 7.0)	(0,0,7.0)	-	(-1000,-300,7.0)	(-600,-300,7.0)
c₃	(0.0, 0.0, 6.0)	(0,0,6.0)	-	(-1000,-300,6.0)	(-600,-300,6.0)
R₁	(2333.3, 0, 8.0)	(1166.7,0,8.0)	-	(650,-300,8.0)	(560,-300,8.0)
R₂	(3133.3, 0, 7.0)	(1583.3,0,7.0)	-	(825,-300,7.0)	(680,-300,7.0)
R₃	(4000, 0, 6.0)	(2000,0,6.0)	-	(1000,-300,6.0)	(800,-300,6.0)
Points excluded (%)	M_s 22/121(18.2) M_w 25/121(20.7)	M_s 23/121(19.0) M_w 25/121(20.7)	43/121 (35.5)	M_s 16/121(13.2) M_w 19/121(15.7)	M_s 15/121(12.4) M_w 15/121(12.4)

Cases excluded when using annual rainfall were discussed in the previous section. When six months rainfall was used, essentially the same cases were excluded except for the 1995 New Zealand and the 1979 California earthquakes, which are now included. However, three new cases were excluded, these being the 1907 Jamaica, the 1967 Colombia and the 1994 New Zealand earthquakes. The Jamaican case is related to very high precedent annual and six months rainfalls but moderate deficit of monthly rainfall. This case produced a small area for its magnitude due probably to physical constraints of the area susceptible to be affected. Landslides triggered were reported in alluvial, coastal and fill deposits which are susceptible under high long-term rainfalls, thus a large area would be expected.

The 1967 Colombia earthquake affected a high area by landslides, mainly disrupted landslides, ground cracks and rock falls in residual and volcanic soils. A moderate area affected would be predicted according to the earthquake magnitude, however, this case is excluded because of extremely high six months rainfalls, which also contributed to landsliding extension. The 1994 New Zealand earthquake is excluded because it affected a very small area for its magnitude. This case produced rock and soil falls, rock slides and debris slides in sedimentary rock deposits; additional soil slides were also triggered in colluvial deposits, after an annual rainfall deficit but a six months

moderate excess and a high monthly rainfall. These conditions may suggest an intermediate area affected by landslides and there is no explanation for this case to have affected a small area.

When annual rainfall excess and monthly rainfall excess are used, with M_s , to define the boundaries of area affected by landslides, only 16 cases were excluded. It is apparent that these parameters produce a better relationship, although, as presented in previous sections, neither total rainfalls nor rainfall excess alone are able to explain the extension of area affected by landslides, but a combination of both is necessary. Therefore pre-determination of the potential area affected by landslides should use at least two of the surfaces proposed. All earthquakes excluded in this case were also excluded in the previous cases, so no further discussion of them is necessary.

Six months rainfall excess, monthly rainfall excess and M_s were also used to define the limiting boundaries. In this case, 15 cases were excluded, which were also excluded in previous cases. When using rainfall excess, four cases of the six that were impossible to be explained in previous section, are excluded; these are the 1968 Japan, the 1968 New Zealand, and the 1968 and 1979 California earthquakes. However, the other two cases that correspond to the 1941 and 1991 Costa Rica earthquakes lie within the boundaries. This strengthens the idea of using more than one of the surfaces to study each case, especially in tropical areas associated with very high long-term rainfalls, where rainfall deficit has been shown to be the key factor to explain the extension of landsliding.

When using moment magnitude as the earthquake parameter some extra points are excluded from boundaries, these being the 1949 Washington, the 1981, 1987, and 1989 California, and the 1989 Venezuela earthquakes. All these earthquakes are cases that lie very close to the magnitude limits, where since M_w is smaller than M_s they move out of boundaries. Use of M_w underpredicts the area affected by landslides for these cases. This result may suggest different magnitude limits for M_w , however the differences are not sufficiently important to make this applicable. Instead it must be suggested that for magnitudes around the limits, the rainfall conditions should be used as prediction parameters.

7.3.2 Epicentral distance to disrupted slides

Reference points to define the surface boundaries for epicentral distance to disrupted slides are compiled in Table 7.4. In this case three different levels were defined: short distances were taken to be shorter than 20 km, the intermediate distance range between 20 km and 100 km, and distances larger than 100 km are considered as large.

Table 7.4 Coordinates of points, in the three-dimensional space (R, r, M), that define the boundary surface for epicentral distance and distance to fault projection to disrupted slides. Subscript 1 refers to region enclosing cases where epicentral distance, and distance to fault projection was larger than 100 km, and 90 km, respectively. Subscript 2 refers to region enclosing cases with epicentral distance and distance to fault projection between 20 km and 100 km, and between 20 km and 90 km, respectively; and subscript 3 refers to region enclosing cases with epicentral distance and distance to fault projection shorter than 20 km. Points are named in similar way to Figure 7.3.

Reference points	Coordinates (R, r, M)				
	Ann. Rainfall Monthly Rainfall M_s and M_w	Six Mon. Rainfall Monthly Rainfall M_s and M_w	CC EC M_s and M_w	Ann. Rainf. Excess Mont. Rainf. Excess M_s and M_w	Six Mont. Rainf. Excess Mont. Rainf. Excess M_s and M_w
a₁	(1333.3, 400, 8.0)	(333.3, 400, 8.0)	(0.4, 0.2, 8.0)	(-611.1, 50, 8.0)	(-444.4, 50, 8.0)
a₂	(1666.7, 500, 7.0)	(416.7, 500, 7.0)	(0.3, 0.2, 7.0)	(-555.5, 100, 7.0)	(-422.2, 100, 7.0)
a₃	(2000, 600, 6.0)	(500, 600, 6.0)	(0.2, 0.2, 6.0)	(-500, 150, 6.0)	(-400, 150, 6.0)
b₁	(3000, 400, 8.0)	(1500, 400, 8.0)	(0.6, 0.2, 8.0)	(250, 50, 8.0)	(100, 50, 8.0)
b₂	(3250, 500, 7.0)	(1625, 500, 7.0)	(0.7, 0.2, 7.0)	(375, 100, 7.0)	(150, 100, 7.0)
b₃	(3500, 600, 6.0)	(1750, 600, 6.0)	(0.8, 0.2, 6.0)	(500, 150, 6.0)	(200, 150, 6.0)
c₁	(0.0, 0.0, 8.0)	(0.0, 8.0)	(0.2, 0, 8.0)	(-1000, -300, 8.0)	(-600, -300, 8.0)
c₂	(0.0, 0.0, 7.0)	(0.0, 7.0)	(0.1, 0, 7.0)	(-1000, -300, 7.0)	(-600, -300, 7.0)
c₃	(0.0, 0.0, 6.0)	(0.0, 6.0)	(0, 0, 6.0)	(-1000, -300, 6.0)	(-600, -300, 6.0)
R₁	(3333.3, 0, 8.0)	(1666.7, 0, 8.0)	(0.8, 0, 8.0)	(638.9, -300, 8.0)	(255.6, -300, 8.0)
R₂	(3666.7, 0, 7.0)	(1833.3, 0, 7.0)	(0.9, 0, 7.0)	(819.4, -300, 7.0)	(327.8, -300, 7.0)
R₃	(4000, 0, 6.0)	(2000, 0, 6.0)	(1.0, 0, 6.0)	(1000, -300, 6.0)	(400, -300, 6.0)
Points excluded (%)	Epicentral M_s 22/123(17.9) M_w 20/123(16.3) Fault projection M_s 9/45(20.0) M_w 7/45(15.5)	Epicentral M_s 21/123(17.1) M_w 20/123(16.3) Fault projection M_s 9/45(20.0) M_w 7/45(15.5)	Epicentral M_s 37/123(30.1) M_w 39/123(31.7) Fault projection M_s 14/45(31.3) M_w 12/45(26.7)	Epicentral M_s 21/123(17.1) M_w 21/123(17.1) Fault projection M_s 9/45(20.0) M_w 7/45(15.5)	Epicentral M_s 23/123(18.7) M_w 23/123(18.7) Fault projection M_s 10/45(22.2) M_w 8/45(17.8)

When annual and monthly rainfalls were used with M_s as parameters, 22 earthquakes were excluded, however, only four cases have no apparent reasons for being outliers. 14 of these cases were also excluded from boundaries for area affected by landslides, and the remaining cases are: the 1914 Panamá, 1948 Japan, 1968 Greece, 1966, 1979 (Aug.), 1980 California, 1988 Nepal and 1993 Oregon earthquakes. Four of these cases have no data about their area affected by landsliding so they were not included within these boundaries. Cases with data about the area affected by landsliding correspond to cases of very small areas, which usually are concentrated in

epicentral areas; this means that unusually large distances to disrupted slides compared with most of the cases may be due to a very high susceptibility of slopes where failure was reported. The eccentricity of the area affected by landslides is another factor that contributes to discrepancies between area affected by landslides and maximum distances to landslides, particularly for small areas, as in these cases.

Cases without data regarding the area affected by landslides and that lie outside the distance boundaries are the Nepal, Greece, Japan and Panamá earthquakes. The Nepal earthquake was related to very large rainfalls, well above the average values. The 1968 Greece earthquake has a very short distance considering its high magnitude. This earthquake is related to almost normal long-term rainfall conditions, although a small rainfall deficit for annual and six months periods were observed. However, monthly excess is relatively large (about 30% larger than average), which may lead one to assume a large area and long distances of landsliding. No explanation for this case to lie outside the boundaries could be found.

The 1948 Japan earthquake had a high magnitude but it produced landslides only within a short distance from its epicentre. In this case very high rainfall deficits were associated, which may constrain the landsliding extension. Failure mechanisms were also limited to ground cracks and lateral spreads. Assuming low water levels, the key factor in these failures may be the ground shaking. Finally, the 1914 Panamá earthquake produced landslides along the Panamá Canal whose slopes have been shown to be very susceptible. Although the specific location of landslides were not reported the distance was indeed very large since this earthquake has an off-shore epicentre very far from the Canal area. Long-term rainfalls associated with this earthquake were well below the average values showing large rainfall deficits, and although monthly rainfall shows a small excess this is not significant, and for clayey deposits, such as those affected, short-term rainfalls are not very important, particularly after a dry season. Although there is no apparent reason for this case to be an outlier, it must be noted that the landslides were actually triggered on man-made cuts.

When using six months and monthly rainfalls, 21 cases lie outside the limit boundaries, which are also excluded in the previous cases. 13 of them were also excluded from the area affected by

landslides surfaces and the remaining 8 cases are the same ones explained in previous paragraphs. In this case also only four cases cannot be explained as outliers.

Event and cycle coefficients were used to define boundaries, and although in Table 7.4 reference points were obtained, it is evident, from the number of points that lie outside, that these factors do not correlate well with maximum distance. From those 37 cases that lie outside the boundaries, only 18 have also been excluded from other surfaces. Due to the high scatter shown by these data, in this case, the remaining 19 cases are not considered to be genuine outliers.

The use of annual and monthly rainfall excess leaves 21 cases outside the boundaries, from which 11 were also excluded from the area affected surfaces, 8 were the same new cases excluded in previous distance surfaces, and only two new cases are excluded. Only 2 of the 21 cases cannot be considered as outliers.

The two new cases excluded are the 1934 Panamá/Costa Rica, and the 1990 Costa Rica earthquakes. Both cases have no data for area affected by landslides, but due to their high magnitude and high antecedent rainfall conditions, it is expected that they are also outliers of surfaces defined for areas. The 1934 earthquake has an intermediate distance, which is lower than expected. Although annual and six months rainfalls exceeded the average values, a monthly rainfall deficit was recorded, which, as explained previously, may constrain landslides in residual soils. The mechanisms associated with this earthquake were slides, slumps, ground cracks and lateral spreads on alluvial and volcanic soils. The 1990 earthquake, which although it triggered landslides to a large distance from epicentre, as expected from the magnitude, lies outside of boundaries due to excessive rainfalls.

When six months rainfall excess is used instead of annual rainfall excess, 23 cases were excluded, from which 11 were also excluded from surfaces defined for area affected by landslides, 9 cases were excluded in previous surfaces for epicentral distance and three new cases were excluded: the 1950 Venezuela, 1974 Nicaragua and 1993 Costa Rica earthquakes. The area affected by landslides due to the 1950 Venezuela earthquake may be predicted by the model, but this case lies

outside the boundaries for epicentral distance to disrupted slides due to excessive monthly rainfalls. Although for the 1974 Nicaragua earthquake, M_S and M_W values were not found, an m_b value of 5.7 was recorded. For this magnitude value a short distance may be expected, but this was intermediate probably due to high precedent annual and six month rainfalls that affected mostly the alluvial and volcanic deposits where landslides were triggered. Particularly six months rainfall excess causes this point to lie outside the boundaries. As for the Venezuelan earthquake, the area affected by landslides and distance to landslides for the 1993 Costa Rica earthquake may be predicted by the model; however, this case outliers due to excessive long-term rainfalls. From these cases it is apparent that boundary could be enlarged to include them, but, from the three points analysed in this section, two may be predicted correctly, from earthquake magnitude value, although they are out of limits, so this expansion is not useful.

When M_W is used instead of M_S , in general, the same points are excluded, although three cases are excluded because they lie near the limits defined by magnitude and with M_W they move slightly out the of boundaries. Two of these cases are the 1987 California and 1949 Washington earthquakes that were also excluded for the same reason in surfaces defined for area affected by landslides. Then only one case further was excluded, the 1961 China earthquake, for which, according to its magnitude, landslides may be expected at a short distance from epicentre; however, it produced landslides up to 100 km from the epicentre and was related to almost normal rainfall conditions. No description of the material where landslides were triggered was found but it can be assumed that it was a loess, which are very susceptible slopes independent of precedent rainfalls, and are very common in China. No data of area affected by landslides nor a value M_S were found for this case, so it may be expected that this is also an outlier from previous boundaries.

7.3.3 Distance to fault projection from disrupted slides

Reference points to define the surface boundaries for distance to fault projection of disrupted slides are also compiled in Table 7.4. In this case, three different levels were also defined, short distance were determine to be shorter than 20 km, intermediate distance range between 20 km and 90 km, and distances larger than 90 km are considered as large.

Annual and monthly rainfalls were used as climatic parameters, resulting in a surface that excluded 9 cases, from which 7 were also excluded in other surfaces and similar reasons for their exclusion can be stated. However, two new cases were excluded, the 1960 Chile and 1968 Iran earthquakes. The Chile earthquake produced a large area affected by landslides and long epicentral distances but the area of fault rupture and consequently its projection is so large that it includes almost the whole area affected by landslides. The Iran earthquake may be expected to produce landslides up to distances ranging from intermediate to large but the distance was very short. In this case, due to commonly very dry conditions, the landslides were constrained to the epicentral area and particularly along the fault trace.

For other relationships, the same points were excluded and the only different points from relations of epicentral distance were the two explained above. This leads one to consider the limits that separate the different zones to be the same, although actual values of distance that represent each range are different. For fault distance fewer data could be found and the applicability of these surfaces may be questioned, although the percentage of data excluded for different cases is almost the same as that those found for epicentral distance and areas, which leads one to assume that the general trend will be conserved if extra data is included. On the other hand, it was shown that the data excluded actually were due to particular conditions that make these cases exceptional. Only one of the cases excluded from relations for fault distances cannot be explained, the 1968 California earthquake, which as stated previously produced larger damage due to landslides than would be expected from its magnitude.

When M_w is used instead of M_s only the 1968 Iran earthquake is additionally excluded compared to those excluded from previous relationships. As observed from Table 7.4, the total number of points excluded in this case are less than that excluded for M_s relationships, and none of the cases that lie near the boundaries and were excluded in previous relations were excluded in this one.

7.3.4 Epicentral distance to coherent slides

Reference points to define the surface boundaries for epicentral distance to coherent slides are compiled in Table 7.5. In this case three different levels were defined, short distance were obtained to be shorter than 30 km, intermediate distance range between 30 km and 140 km, and distances larger than 140 km are considered as large.

Table 7.5 Coordinates of points, in the three-dimensional space (R, r, M), that define the boundary surface for epicentral distance and distance to fault projection to coherent slides. Subscript 1 refers to region enclosing cases where epicentral distance, and distance to fault projection was larger than 140 km, and 80 km, respectively. Subscript 2 refers to region enclosing cases with epicentral distance and distance to fault projection between 30 km and 140 km, and between 10 km and 80 km, respectively; and subscript 3 refers to region enclosing cases with epicentral distance and distance to fault projection shorter than 30 km and 10 km respectively. Points are named in similar way to Figure 7.3.

Reference points	Coordinates (R, r, M)				
	Ann. Rainfall Monthly Rainfall M_s and M_w	Six Mon. Rainfall Monthly Rainfall M_s and M_w	CC EC M_s and M_w	Ann. Rainf. Excess Mont. Rainf. Excess M_s and M_w	Six Mont. Rainf. Excess Mont. Rainf. Excess M_s and M_w
a_1	(500, 200, 8.0)	(250,200,8.0)	(0.4,0.2,8.0)	(-611.1,50,8.0)	(-444.4,50,8.0)
a_2	(750, 300, 7.0)	(375,300,7.0)	(0.3,0.2,7.0)	(-555.5,100,7.0)	(-422.2,100,7.0)
a_3	(1000, 400, 6.0)	(500,400,6.0)	(0.2,0.2,6.0)	(-500,150,6.0)	(-400,150,6.0)
b_1	(2000, 200, 8.0)	(1000,200,8.0)	(0.6,0.2,8.0)	(250,50,8.0)	(100,50,8.0)
b_2	(2500, 300, 7.0)	(1250,300,7.0)	(0.7,0.2,7.0)	(375,100,7.0)	(150,100,7.0)
b_3	(3000, 400, 6.0)	(1500,400,6.0)	(0.8,0.2,6.0)	(500,150,6.0)	(200,150,6.0)
c_1	(0.0, 0.0, 8.0)	(0,0,8.0)	(0.2,0,8.0)	(-1000,-300,8.0)	(-600,-300,8.0)
c_2	(0.0, 0.0, 7.0)	(0,0,7.0)	(0.1,0,7.0)	(-1000,-300,7.0)	(-600,-300,7.0)
c_3	(0.0, 0.0, 6.0)	(0,0,6.0)	(0,0,6.0)	(-1000,-300,6.0)	(-600,-300,6.0)
R_1	(2500, 0, 8.0)	(1250,0,8.0)	(0.8,0,8.0)	(638.9,-300,8.0)	(255.6,-300,8.0)
R_2	(3125, 0, 7.0)	(1625,0,7.0)	(0.9,0,7.0)	(819.4,-300,7.0)	(327.8,-300,7.0)
R_3	(4000, 0, 6.0)	(2000,0,6.0)	(1.0,0,6.0)	(1000,-300,6.0)	(400,-300,6.0)
Points excluded (%)	Epicentral M_s 4/42(13.8) M_w 4/36(11.1) Fault projection M_s 4/29(13.8) M_w 3/25(12.0)	Epicentral M_s 4/42(9.5) M_w 4/36(11.1) Fault projection M_s 5/29(17.2) M_w 4/25(16.0)	Epicentral M_s 12/42(28.6) M_w 12/36(33.3) Fault projection M_s 8/29(27.6) M_w 8/25(32.0)	Epicentral M_s 7/42(16.7) M_w 7/36(19.4) Fault projection M_s 7/29(24.1) M_w 7/25(28.0)	Epicentral M_s 9/42(21.4) M_w 9/36(25.0) Fault projection M_s 7/29(24.1) M_w 7/25(28.0)

As can be observed in Table 7.5, the number of available data points for coherent slides is much more limited than those for area affected and for disrupted slides. Only four points were excluded when annual and monthly rainfall and M_s were used as parameters. Three of these points were also excluded from the surface defined for area affected by landslides and one from the surface defined for disrupted slides, and the same reasons can be stated to explain them as outliers. When using six months and monthly rainfalls exactly the same points were excluded.

Once again cycle coefficient and event coefficient give the least reliable results, as observed by the number of data excluded and the scatter of the data. Only when rainfall excesses were used new cases were excluded, these being the 1906 and 1927 California and the 1958 Alaska earthquakes. The distance to coherent slides produced by the 1927 California earthquake may be correctly predicted by the model although it lies outside of boundaries, from magnitude value. This case is related to very high monthly rainfall excess, which makes it an outlier, the boundary may be moved slightly in order to this point to lie in, but, as it is predicted correctly this change is not necessary.

The 1906 California and 1958 Alaska earthquakes produced disrupted slides at very large distances, which would be predicted as intermediate by the model. These two points lie outside the boundaries due to high monthly rainfall excess. Greater distances than expected, such as during the 1906 California earthquake, may occur due to it affecting areas with slopes in residual and dune deposits, which are susceptible to these short-term rainfalls. Alaska has been shown to be very prone to landslides during seismic events mainly due to the slope susceptibility, particularly sensitive clays, which may be the reason for the 1958 earthquake to lie out of boundaries. Furthermore, earthquakes in this region usually are related to large areas of fault rupture that make the distance to fault rupture more significant than epicentral distance.

When M_w is used instead of M_s , exactly the same points are excluded, and when the rainfall excess is used the same three new points, explained previously, are excluded. Although less cases are included, due to lack of data, a similar percentage of excluded data as in previous cases was found.

7.3.5 Distance to fault projection from coherent slides

Reference points to define the surface boundaries for distance to fault projection from coherent slides are also compiled in Table 7.5. In this case three different levels were defined, short distance were obtained to be shorter than 10 km, intermediate distance range between 10 km and 80 km, and distances larger than 80 km are considered as large.

Data of distance to fault projection from coherent slides were more difficult to obtain, and only 29 cases were compiled to define the limit boundaries. All points excluded in these cases were also excluded in previous surfaces, and the same reason can be argued for them to out lie.

When using annual and monthly rainfalls four cases were excluded, the 1946 Canada, 1960 Chile, 1978 and 1980 (Jan.) California earthquakes. The 1946 Canada earthquake is excluded due to large annual and six months rainfalls, although the model predicts correctly the distance to the fault projection from coherent slides. The two California earthquakes have magnitudes slightly lower than 6.0 but they produced distances larger than 10 km, which may be explained by large annual and six months rainfall excess that alter the susceptibility of alluvial and fills where landslides were triggered. Finally, the 1960 Chile earthquake lies outside the of limits for distance to fault projection due to the large extension of the fault rupture.

The same four points were excluded when using six months and monthly rainfalls, but an additional case was also excluded, which is the 1994 New Zealand earthquake, excluded by having excessive monthly rainfalls, although, the model predicts correctly the distance to fault projection.

As in previous cases, the use of cycle and event coefficients produced large scatter of the data and a larger number of cases to out lie. In this case 8 cases were excluded, and as explained before, not all of them are actual outliers, hence no additional analysis was carried out for them.

Seven cases were excluded when rainfall excesses were used. These are the five cases analysed for total rainfalls and the three cases excluded when using rainfall excess for defining boundaries of epicentral distance in previous section; these are the 1906 and 1927 California earthquakes and the 1958 Alaska earthquakes, which were analysed previously.

Using M_w instead of M_s gives basically the same outliers as before, except the Chile earthquake which is not included due to lack of M_w , although it must be expected to out lie due to its large fault rupture. The 1966 Perú earthquake is also excluded in this case. This earthquake is related to very dry but normal conditions, and magnitude is the key factor in landsliding occurrence. Change of M_s

for M_w in this case moves the point out of the boundaries. With a M_w larger than 8.0 it would be expected to produce large distances from the fault projection, but the dry climatic conditions tend to constrain the landsliding around the epicentral area and an intermediate distance was actually recorded.

7.3.6 Epicentral distance to lateral spreads and flows

Reference points to define the surface boundaries for epicentral distance to lateral spreads and flows are compiled in Table 7.6. In this case three different levels were defined, short distance were obtained to be shorter than 30 km, intermediate distance range between 30 km and 170 km, and distances larger than 170 km are considered as large.

Table 7.6 Coordinates of points, in the three-dimensional space (R, r, M), that define the boundary surface for epicentral distance and distance to fault projection to lateral spreads and flows slides. Subscript 1 refers to region enclosing cases where epicentral distance, and distance to fault projection was larger than 170 km, and 80 km, respectively. Subscript 2 refers to region enclosing cases with epicentral distance and distance to fault projection between 30 km and 170 km, and between 10 km and 80 km, respectively; and subscript 3 refers to region enclosing cases with epicentral distance and distance to fault projection shorter than 30 km and 10 km respectively. Points are named in similar way to Figure 7.3.

Reference points	Coordinates (R, r, M)				
	Ann. Rainfall Monthly Rainfall M_R and M_W	Six Mon. Rainfall Monthly Rainfall M_R and M_W	CC EC M_R and M_W	Ann. Rainf. Excess Mont. Rainf. Excess M_R and M_W	Six Mont. Rainf. Excess Mont. Rainf. Excess M_R and M_W
a_1	(666.7, 200, 8.0)	(333.3,200,8.0)	(0.4,0.2,8.0)	(-375,0,8.0)	(-450,0,8.0)
a_2	(833.3, 250, 7.0)	(416.7,250,7.0)	(0.3,0.2,7.0)	(-437.5,50,7.0)	(-425,50,7.0)
a_3	(1000, 300, 6.0)	(500,300,6.0)	(0.2,0.2,6.0)	(-500,100,6.0)	(-400,100,6.0)
b_1	(1500, 200, 8.0)	(1000,200,8.0)	(0.6,0.2,8.0)	(250,0,8.0)	(0,0,8.0)
b_2	(2250, 250, 7.0)	(1125,250,7.0)	(0.7,0.2,7.0)	(375,50,7.0)	(100,50,7.0)
b_3	(3000, 300, 6.0)	(1250,300,6.0)	(0.8,0.2,6.0)	(500,100,6.0)	(200,100,6.0)
c_1	(0.0, 0.0, 8.0)	(0,0,8.0)	(0.2,0,8.0)	(-1000,-300,8.0)	(-800,-300,8.0)
c_2	(0.0, 0.0, 7.0)	(0,0,7.0)	(0.1,0,7.0)	(-1000,-300,7.0)	(-800,-300,7.0)
c_3	(0.0, 0.0, 6.0)	(0,0,6.0)	(0,0,6.0)	(-1000,-300,6.0)	(-800,-300,6.0)
R_1	(2166.7, 0, 8.0)	(1166.7,0,8.0)	(0.8,0,8.0)	(625,-300,8.0)	(150,-300,8.0)
R_2	(3083.3, 0, 7.0)	(133.3,0,7.0)	(0.9,0,7.0)	(812.5,-300,7.3)	(275,-300,7.3)
R_3	(4000, 0, 6.0)	(1500,0,6.0)	(1.0,0,6.0)	(1000,-300,6.0)	(400,-300,6.0)
Points excluded (%)	Epicentral M_S 10/43(23.2) M_W 9/35(25.7) Fault projection M_S 5/28(17.9) M_W 4/24(16.7)	Epicentral M_S 10/43(23.2) M_W 9/35(25.7) Fault projection M_S 5/28(17.9) M_W 4/24(16.7)	Epicentral M_S 16/43(37.2) M_W 15/35(42.9) Fault projection M_S 8/28(28.6) M_W 7/24(29.2)	Epicentral M_S 8/43(18.6) M_W 8/35(22.9) Fault projection M_S 4/28(14.3) M_W 3/24(12.5)	Epicentral M_S 9/43(20.9) M_W 8/35(22.9) Fault projection M_S 5/28(17.9) M_W 4/24(12.5)

Similar to coherent slides, data for lateral spreads and flows was much more limited than data for area affected by landslides and disrupted slides, although, boundary limits can be defined in a similar way. Comparison of Tables 7.4 to 7.6 shows that for lateral spreads and flows the

relationships are much less reliable, from observing the total number of data analysed and number of data excluded for defining the boundaries.

From ten cases excluded when using annual and monthly rainfalls, six were previously excluded in other relations and the same reasons may be given for them to out lie. The other four cases are the 1843 Guadalupe, 1959 Montana, 1983 Costa Rica and 1983 Idaho earthquakes. The 1843 Guadalupe earthquake produced flows up to a distance of 150 km, which is slightly lower than the limit between intermediate and large distances (170 km). This earthquake was related to high annual, six months and monthly rainfall deficits, which may constrain distances, although, geographical constraint may also be a cause for this point to out lie.

Epicentral distance to flows due to the 1959 Montana and 1983 Idaho earthquakes may be predicted correctly by the model although they lie outside the boundaries. These points out lie due to excessive annual and six month rainfalls. This result confirms that lateral spreads and possibly flows are related to high precedent rainfall conditions. It has been found that flows result after failure of slopes on brittle material under high water contents that increases their mobility; previously this mechanism was associated with residual soils that failed after high short-term rainfalls, but for slopes affected during these two earthquakes they are not only associated with residual soil but also with colluvial and talus deposits. For the 1983 Costa Rica earthquake, an intermediate to large epicentral distance would be expected, but this was short probably due to high rainfall deficits.

The use of six months and monthly rainfalls excluded the same points as the previous surface, whereas cycle and event coefficients were not considered because the boundaries are related to the larger scatter of the data, which limited their applicability as climatic parameters. The same four different cases excluded in previous surfaces for epicentral distance to lateral spreads and flows were excluded when rainfall excesses were used as climatic factors.

Change of M_s to M_w as seismic factor includes two of the points excluded in previous. These are the 1843 Guadalupe and 1983 Idaho earthquakes, reducing the number of data excluded. In this case, the magnitude of the Guadalupe earthquake becomes smaller than 8.0 and the points lies

within boundaries for intermediate distances. The 1983 Idaho earthquake is a case that produced epicentral distance to lateral spreads and flows of 30 km, which is just the limit between short and intermediate distances, and a small difference in the rainfall limits makes it to lie within or outside the boundaries. Except for these two points, all the other points excluded are the same as those excluded when M_S is used.

7.3.7 Distance to fault projection to lateral spreads and flows

Reference points to define the surface boundaries for distance to fault projection from lateral spreads and flows are also compiled in Table 7.5. In this case three different levels were defined, short distance were taken to be shorter than 10 km, intermediate distance range between 10 km and 80 km, and distances larger than 80 km are considered as large.

For all combinations of parameters, most of the points excluded were also excluded for surfaces defined for epicentral distance to lateral spreads and flows. When using the M_S value the exceptions are the 1960 Chile and 1990 Costa Rica earthquakes, which were also reported to be excluded from previous relationships. The 1966 Perú earthquake was excluded when using M_W .

As stated previously, the 1960 Chile earthquake produced a large fault rupture whose projection cover most of the area affected by landslides and consequently distances to the fault projection were constrained. For the 1990 Costa Rica earthquake, although lateral spreads and flows were extensive, massive and up to large epicentral distance, most of the area affected by landslides lies on the fault projection, which constrained the distance to fault projection; on the other hand, this earthquake is related to very high rainfall excess that also causes the point out lie. As stated before, the 1966 Perú earthquake is related to very dry but normal conditions, and magnitude is the key factor in landsliding occurrence. Change of M_S to M_W in this case moves the point out of the boundaries. With a M_W larger than 8.0, it would expected that this case would produce landslides to a large distance from the fault projection; however, the dry climatic conditions tend to constrain the landsliding around the epicentral area and an intermediate distance was actually recorded.

7.3.8 Summary

In the previous sections, the cases excluded from each of the relationships that define boundaries for different levels of area affected by landslides, and epicentral distance and distance to fault projection, were identified and possible reasons for them to out lie investigated. In total 56 different cases were identified, for all relationships, to out lie, however, for only six of them, no apparent reason to out lie was found, which is the total number of cases that actually do not follow the model proposed.

Cases excluded from different relationships can be divided into groups according to possible explanatory reason. In very rainy areas as Costa Rica, Colombia, Panamá and New Zealand where high annual and monthly rainfalls are common, cases out lie since boundaries for the model were limited to the intermediate rainfall condition, which includes most of the regions where earthquake-induced landslides have been reported. For regions with higher rainfalls, the model predicts values as a function of earthquake magnitude and rainfall excess or deficit.

Earthquakes in Canada, Alaska, and China have been shown to lie outside of boundaries, even for normal climatic conditions, due to earthquakes occurring in regions of very susceptible materials such as sensitive clays and loess.

Some earthquakes are excluded because they produce areas and distances smaller than would be predicted. For those cases it was identified that dry conditions constrained these values. In areas where shallow residual soils are common, dry short-term conditions are the key factor that controls the landsliding extension. In areas where colluvial, alluvial, and talus deposits are the most common, dry long-term conditions are those which control the slope response.

A third group are those cases that produced areas or distances larger than expected. In this case rainfall conditions are also the key factor that controls landsliding extension. In areas of shallow residual soils, high short-term rainfall increases the slope susceptibility, whereas for areas of colluvial, alluvial or talus deposits, high levels of long-term rainfall are the controlling factor.

Many cases that produced areas and distances according with levels defined by the model were excluded because one of the rainfall parameters is larger than normally defined by boundaries. Then although they out lie, the model would predict these values correctly basically because the earthquake magnitude was the controlling factor in landsliding. Usually these cases out lie from one of the surfaces proposed and within the others, which suggests that application of the model must be based on a combination of the different surfaces and not only on one of them, in order to identify as accurately as possible the actual exceptional cases. Particularly, it is advisable to verify a possible outlier with boundaries defined from rainfall excess.

Similarly it was shown that some cases are moved out of the boundaries when moment magnitude is used. These cases are usually those that lie near the boundaries of M_S values and slightly lower M_W values. In this case it is also advisable to use boundaries based on both magnitude scales to verify possible exceptional cases.

7.4 Model applicability

Definition of areas affected by landslides and distances to different failure mechanisms can be used as a preliminary hazard assessment tool. Although the value of area affected by landslides alone is not a conclusive parameter due to possible eccentricity with respect to earthquake source, this parameter is useful for comparison purposes of the potential susceptibility of seismic areas. On the other hand, epicentral distance and distance to fault projection may be used to define the possible area affected by landslides within which a more detailed study must be carried out. Application of this model for such purposes is presented in Chapter 8.

The present results can be used to re-evaluate those relationships, classified as methods of Grade-1 in Chapter 4, between earthquake magnitude and areas or distances. Figure 7.15 shows the relationship defined by Keefer (1984) between area affected by landslides and magnitude. The limits defined during the present study are also shown.

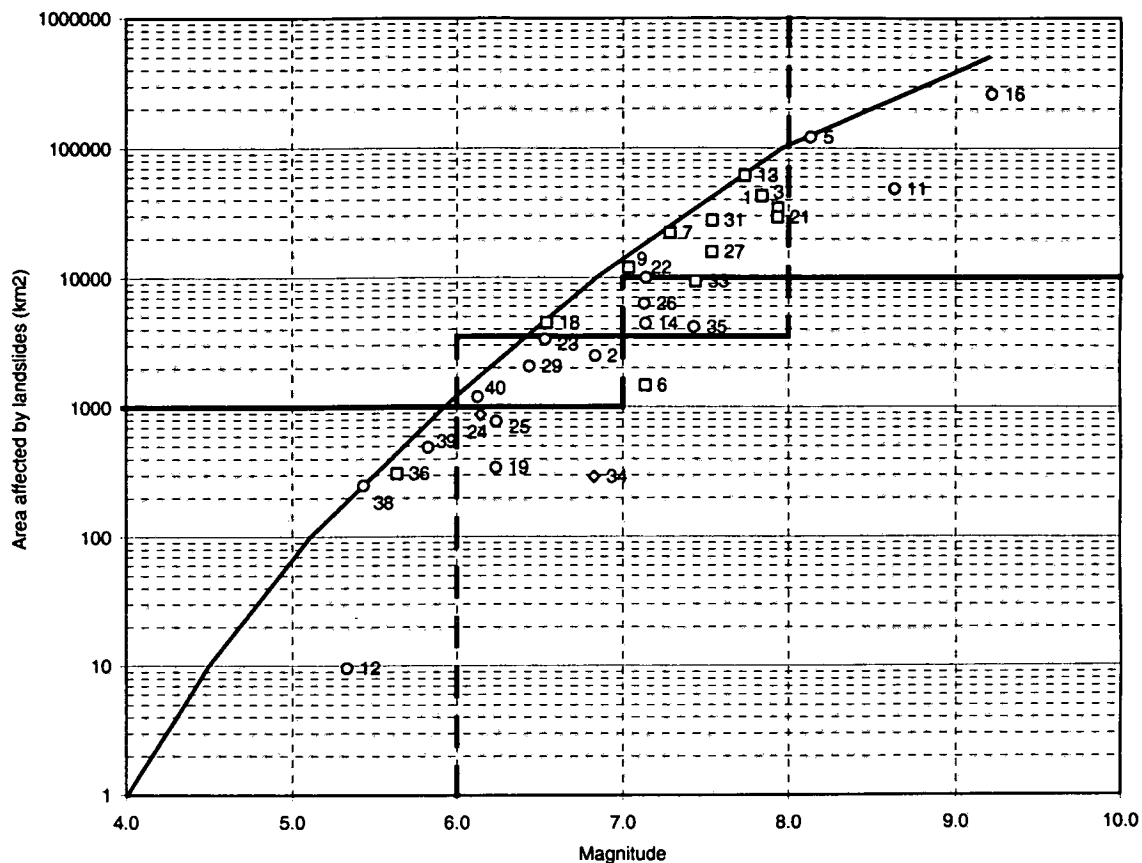


Figure 7.15 Magnitude and area affected by landslides relationship. Points represent data from Keefe (1984). Thin line is Keefe's envelope, and thick lines enclose the limits obtained in present study. Solid horizontal lines are the limits established for the four different area affected categories: small, lower intermediate, upper intermediate and large areas. Dashed vertical lines represent the limits established by magnitude for different area affected categories as before. Numbers refer to Keefe earthquake list, shown in Table 5.17. Circles are points for which the area is predicted correctly by present model, squares are cases with areas larger than expected and diamonds represent cases with areas smaller than predicted.

From the 30 cases included in Keefe's graph only 16 lie within the boundaries established for the present model, 5 show areas smaller than would be predicted and 9 show areas larger than would be predicted. The first conclusion that may be stated is that although all cases lie below the upper limit line, not all of them are normal cases; exceptional cases can also lie below these kinds of limits.

Those cases with areas smaller than predicted are the 1940 Imperial Valley, 1966 Parkfield, 1973 Hawaii, 1974 Pakistan, and 1978 Izu-Oshima earthquakes. For the Imperial Valley earthquake, Keefe (1984a) reported an area affected by landslides of about 1500 km², which classifies within the lower intermediate zone; however, the present study defined the area affected by landslides during this earthquake as 8000 km² which classifies within the upper intermediate zone which is in

accord with its moment magnitude. The difference may be due to Keefer only considering the area within the USA, whereas landsliding was also reported in Mexico. If surface wave magnitude is considered, this case lies above the limit giving a larger area than expected. No actual data of rainfall was recorded for this case, so it was not reported in the previous section; however, the region affected by the earthquake is characterised by relatively small rainfall (77 mm annual and 1.5 mm monthly values), which may constrain landsliding extension. Landslides triggered by this earthquake were slumps and lateral spreads on alluvial, deltaic and fill deposits; based on the model results it can be concluded that for this earthquake to produce a relatively large area affected by these kind of landslides, high long-term precedent climatic conditions would have been required.

The 1966 Parkfield earthquake was reported by Keefer as having a moment magnitude of 6.2; however, data compiled for the present study reported a moment magnitude of 5.3, and a surface wave magnitude of 5.5. This difference causes the case to move toward the left, lying within the limit proposed, given that the actual area affected by landslides was small. This case was excluded from the epicentral area to disrupted slides relationship due to eccentricity of area affected by landslides with respect to the epicentre. Due to high rainfall deficits, landsliding induced by this earthquake was constrained around the fault scarp.

The 1973 Hawaii earthquake, with a surface-wave magnitude of 6.1 would have been expected to produce an area affected by landslides between 1000 and 3500 km²; however, the actual area was just 880 km² due to physical constraint of the area available to be affected, and secondly, the earthquake was related to very dry antecedent conditions.

The 1974 Pakistan earthquake was reported by Keefer to produce an area affected by landslides of 775 km²; however, during the present study an area of 2300 km² was established, which moves the case within the proposed limits.

The 1978 Izu-Oshima earthquake was reported to have affected an area of 290 km² by landslides, which is smaller than expected. Although this earthquake was related to a small monthly rainfall deficit, which may constrain the area because of the presence of residual soils, it is most likely that

the cause for this case to give so small area was that it was a off-shore event and area affected was geographically constrained.

Cases that reported areas larger than those that would be expected are the 1811-1812 New Madrid, 1906 San Francisco, 1946 Canada, 1949 Washington, 1958 Alaska, 1965 Washington, 1970 Perú, 1976 China and 1976 Guatemala earthquakes.

The New Madrid earthquake was a multiple event with three main events with magnitudes higher than 7.5; this characteristic may be the reason for the case to have triggered slides over a very large area, although it is worth noting that in Keefer's plot this case was given a M_s magnitude between 7.3 to 7.5, whereas during the present study its maximum magnitude was found to be 8.7. This change in magnitude moves the case within boundaries, and the same result is obtained if moment magnitude is considered.

The San Francisco earthquake in 1906 was one of the cases discussed in the previous section due to it lies outside of boundaries. Based on magnitude, an area between 3500 and 10000 km² would be expected, whereas the actual area was reported to be 35000 km². With a moment magnitude of 7.9 this case lies very close to boundary between intermediate and large areas, so the analysis must be carried out with care. This earthquake was related to very high rainfall excesses, particularly very high monthly rainfall excess, and landslides were reported in residual and dune deposits that have been shown to be very susceptible under this climatic condition. This earthquake was also excluded from the surface for epicentral distance to coherent slides; in this case long-term, rainfall also plays an important role in slope stability given that landslides were reported in alluvial deposits and fills.

The 1946 Vancouver, Canada, earthquake was related to extremely high rainfalls, particularly very high long-term rainfalls were recorded (about 2500 mm above average annual and 700 mm above average six months rainfalls), which explains the large area affected, given that most of the slides were triggered in alluvial deposits.

The Puget Sound, Washington, earthquake in 1949 was one of the cases that moved out of boundaries when different magnitude scales are used. When surface wave magnitude is used, this case moves well above the Keefer upper limit, which shows the inconvenience of mixing scales in defining these kind of boundaries. Keefer reported an area due to this earthquake of about 12000 km², but an area of 28500 km² was found during the present study, which moved the case well above Keefer's limit. Independent of the magnitude used for analysis, this case produced a higher area affected by landslides than expected; although annual rainfall was larger than average, six months and monthly rainfall were well below average, consequently no explanation for this point to out lie can be found. It may be presumed that the lithologies affected are very susceptible, but no evidence for this was obtained.

The Alaska earthquake in 1958 is another case that lies very close to the magnitude limit between intermediate and large areas. The actual area affected by landslides was about 60000 km². Alaska has been shown to be very prone to landslides during seismic events mainly due to the slope susceptibility, particularly on sensitive clays, which may be the reason for the 1958 earthquake to lie outside the boundaries. Furthermore, earthquakes in this region usually are related to large areas of fault rupture that makes the distance to fault rupture more significant than epicentral distance. High short-term rainfalls may also be stated as a possible cause for this event producing a large area.

The Puget Sound earthquake in 1965 affected the same region that was affected by the 1949 earthquake, and again a larger area than expected was affected. Susceptibility of slopes may be the main reason for earthquakes in this region to cause exceptional damage by landslides. Keefer gives an area of about 4500 km², which lies slightly above present limits, but this study determined an area of 20700 km², which causes the case to lie well above the Keefer line. The earthquake is related to rainfall excesses but they are not higher than 5% for annual and six months rainfalls and 22% for monthly rainfalls; however, for alluvial deposits these precedent rainfalls may play an important role in inducing landslides.

The 1970 Perú earthquake is another case that lies near the magnitude boundary between intermediate and large areas. For this study, a moment magnitude of 7.96 was found, which actually

moves the case within the boundaries. For the region, dry conditions are normal and for this earthquake rainfall conditions were slightly drier than normal, which suggests that earthquake shaking is the controlling factor in landsliding.

The 1976 Guatemala earthquake was associated with very large areas affected by landslides, especially in volcanic soils. This earthquake was related to very dry conditions and the controlling factor of landslides seems to be the extension of fault rupture along which landslides were concentrated, and most importantly, the triggering of secondary shocks due to reactivation of sub-perpendicular faults which extend the area affected for this earthquake. Similar large extension of landsliding on volcanic soil have been observed after dry conditions during the recent El Salvador earthquake, which suggests that for this kind of deposits the ground shaking and its characteristics are the controlling factor rather than precedent climatic conditions.

Finally the 1976 Tangshan, China, earthquake triggered landslides over a very large area. Although rainfall conditions were very dry for that year, the extension of landslides may be so large due to high slope susceptibility. It has been reported that slides and flows in loess deposits were the main mechanism. As for volcanic soils, seismic response of loess deposits may be control by ground shaking rather than by climatic conditions.

The San Juan, Argentina, earthquake in 1977, that is within boundaries actually must out lie since the actual area affected by landslides is slightly higher than reported by Keefer. Antecedent dry conditions, which are normal in the region, were present for this case; however, the most common failure mechanisms were lateral spreads and liquefaction on alluvial deposits, which usually are related to high water levels. No explanation for this earthquake to produce larger area affected by landslides than expected was found.

Another case of the points presented by Keefer that lies within boundaries shown in Figure 7.15 but actually is out of limits is the 1978 Santa Barbara earthquake. The area affected by landslides was reported by Keefer as 310 km² but this study defined an area of 2250 km², which makes the point to

move out of the graph boundaries. Large annual and six months rainfall excesses may be the cause of the Santa Barbara earthquake to lie outside the boundaries.

Differences in areas proposed by Keefer and those found in this study may be due to Keefer in some cases concentrating only on areas where high concentration of landslides were reported, whereas this study considers the total area that encloses the farthest cases reported. As discussed in Chapter 4, relationships based on those areas of high landslides concentration tend to be more uniform and boundaries lie well below those reported. It was impossible to develop the present model using areas of high concentration due to lack of data.

Finally, the 1957 Daly City earthquake that was reported by Keefer to have produced an area of about 9.5 km^2 , was also found to have affected a larger area; this study recorded an area of 320 km^2 , which moves the point beyond the Keefer upper limit but stay within the present proposed boundaries.

As observed from the previous analyses, the upper limit proposed by Keefer (1984) presents some limitations especially given by the lack of analysis of rainfall influence on the extension and mechanism of landslides. Comparison of Keefer's limit with present results shows that some cases presented by Keefer have larger areas than reported and most of these cases move above the Keefer boundary. Some other cases have larger magnitudes than reported by Keefer and this moves the cases within the present boundaries, making them to past from being consider as exceptional cases to be normal cases. Figure 7.16 shows the points, included in the Keefer's plot, considering the corrected values found in the present study.

Although all cases considered by Keefer lie below his upper limit, some of them are actually exceptional cases because the area affected by landslides is larger or smaller than expected. On the other hand, some points that lie near the Keefer upper limit are actually cases with exceptionally small areas, and finally there are some cases that lie well below this upper limit which represent cases where landsliding was constrained either by rainfall conditions or geographical limitations.

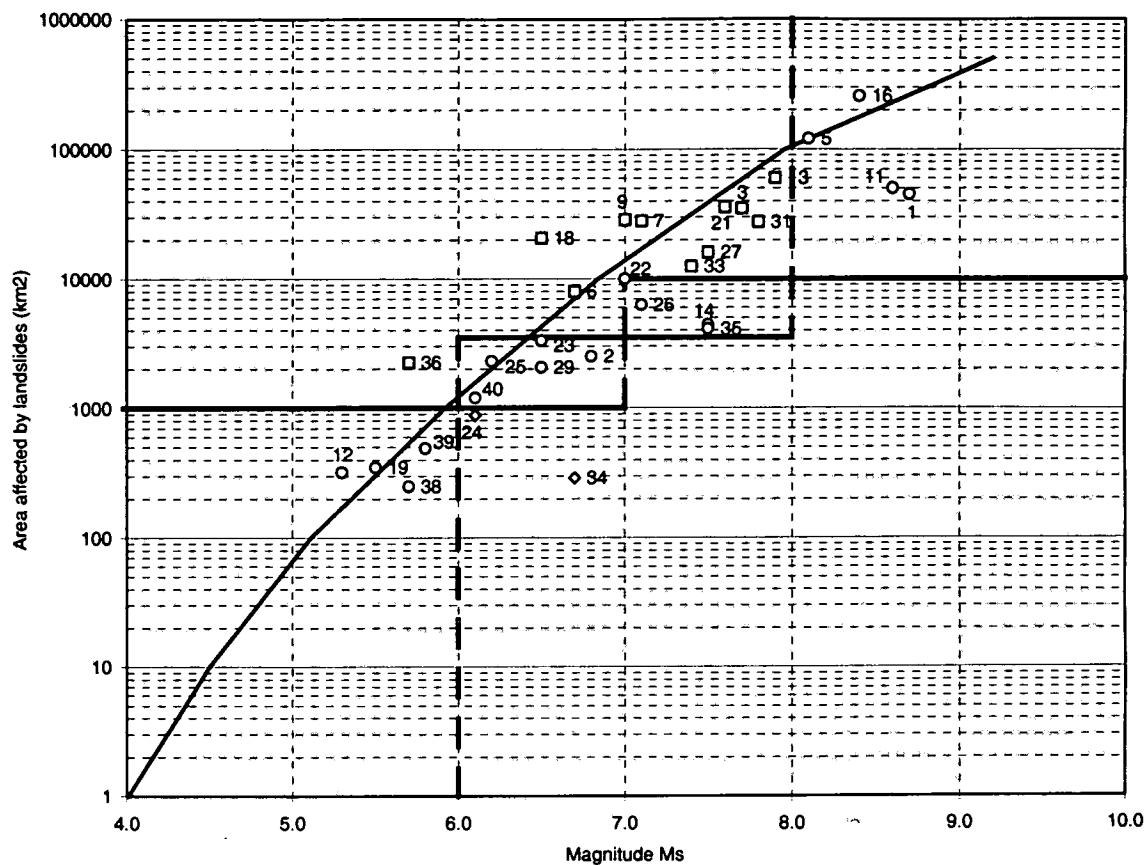


Figure 7.16 Magnitude and area affected by landslides relationship. Points represent corrected data for cases considered by Keefer (1984). Thin line is Keefer's envelope, and thick lines enclose the limits obtained in present study. Solid horizontal lines are the limits established for the four different area affected categories: small, lower intermediate, upper intermediate and large areas. Dashed vertical lines represent the limits established by magnitude for different area affected categories as before. Numbers refer to Keefer earthquake list, shown in Table 5.17. Circles are points for which the area was predicted correctly by present model, squares are cases with areas larger than expected and diamonds represent cases with areas smaller than predicted.

It is also worth noting that 67% of the data included in the Keefer plot are cases out of the inter-quartile range. Most of them are cases above the third quartile, which may indicate that cases included by Keefer are biased by the earthquake effects. Keefer bound may be a very conservative relationship, which is confirmed by the fact that the proposed zones, obtained with a much larger database, lie below the Keefer's line.

It has been suggested by Crozier (1991) and Jibson (1996) that relationships between area affected by landslides or distances and magnitude, such as those presented by Keefer, can be used in paleoseismicity in order to obtain the magnitude of historical earthquakes. Crozier (1991) presented the analysis of three historical earthquakes in New Zealand for which magnitude were defined based

on Keefer's relationships, and other relationships were also used for comparative purposes. Table 7.7 shows these results.

Table 7.7 Magnitude of historical earthquakes in New Zealand (Modified from Crozier, 1991)

Criteria	Source	Magnitude of earthquakes		
		A	B	C
Area affected by landslides	Adams (1981)	7.4	7.3	7.5
	Keefer (1984)	5.9	5.7	6.2
	Present study	6.0-7.0	5.5-6.0	6.0-7.0
Epicentral distance to liquefaction sites	Tinsley et al. (1985)	6.6	6.5	6.8
	Kuribayashi & Tatsuoka (1975, 1977)	6.5	6.4	6.8
	Present study	6.0-7.0	6.0-7.0	6.0-7.0
Parameters				
Distance of the furthestmost landslide to epicentre (km)		23	20	28
Area affected by landslides (km ²)		1015	825	1855

Since the present study does not give an exact value of magnitude, this model is not directly applicable for paleoseismic purposes); However, from Table 7.7 it can be seen that the Adams predictions are well above the other cases, and for earthquake A, Keefer's model gives a smaller magnitude to that expected. A more detailed analysis could be made if rainfall data were available, but no specific date was given for those cases that may allow further studies.

CHAPTER 8

LANDSLIDE HAZARD ASSESSMENT

A second hazard model was obtained from the data compiled for individual landslides. This model is based on the general definition of landslide hazard as a product of a triggering function and a susceptibility function. Triggering in this case was defined in terms of strong-motion parameters described in previous chapters, which are PGA, PGV, and Arias intensity. Only these parameters were considered because attenuation equations are required for the model to be applicable. Slope susceptibility can be defined by a combination of intrinsic factors such as slope inclination and height, and extrinsic factors such as precedent climatic conditions. Climatic short-term and long-term, total rainfall and rainfall excess conditions were considered simultaneously.

Hazard intensity may be defined by landslide density in a homogeneous area or by a single landslide characteristic such as the velocity, volume or depth. Although Hungr (1997) argued that the maximum movement velocity is the most important intensity parameter, this one is not applicable in this case due to the fact that it is not measured currently in historical cases and its definition based on witnesses; description is very subjective.

Landslide density for earthquake-induced landslide case histories is not well defined in most of the cases, and a detailed map of the affected areas would be required, which is not commonly available. As a result, for analysis purposes, landslide morphometric characteristics may be taken to define the intensity, as suggested by Hurg (1997). Since landslide volume has been used as analysis parameter in long-term erosion studies (Keefer, 1994), for paleoseismicity analysis (Jibson, 1996), and for natural dams analysis (Perrin and Hancox, 1991; Moon, 1997), it would be the most appropriate parameter to use as intensity parameter, however; as stated in Chapter 3, the volume of seismically-induced landslides is a function of geologic structural features rather than of the seismic

shaking or susceptibility. In order to use morphometric characteristics as the intensity parameter a statistical analysis of positive against negative evidence cases is required. In the present project no negative evidence cases were included so the hazard assessment must be defined qualitatively in terms of descriptive relative grades.

8.1 Model description and development

The model was developed from the basic definition of landslide hazard presented in Chapter 1 and expressed by:

$$H = T * S \quad (8.1)$$

where H is the landslide hazard, T is a triggering function and S is a susceptibility function.

As discussed previously, the present study assumes that the triggering is the seismic event and the triggering factor is defined in terms of strong-motion parameters. In Chapter 7, the triggering factor was defined in terms of the earthquake magnitude, which is a source parameter that does not take into account path and site effects on ground motion. The present model uses site parameters to define triggering: PGA, PGV and Arias Intensity.

The susceptibility function is defined by a combination of slope geometry, particularly slope angle and height, and climatic precedent conditions. As discussed in Chapter 7, not only long-term and short-term total rainfalls are important during the process but rainfall deficit or excess with respect to average values also play an important role.

For the triggering function a normalised value of each strong-motion parameter was obtained based on the maximum and minimum values of those parameters within the area potentially affected by landslides. This function was expressed as:

$$T = \frac{(SP - SP_{\min})}{(SP_{\max} - SP_{\min})} \quad (8.2)$$

where SP is the seismic parameter (PGA, PGV or AI) for the slope location, $SP_{\min}$ and $SP_{\max}$ are the maximum and minimum values of the seismic parameter related to the area where landslides were triggered. $SP_{\max}$ corresponds to the seismic parameter for an epicentral or fault projection distance equal to zero. $SP_{\min}$ corresponds to the seismic parameter for the maximum epicentral distance or the maximum distance to fault projection to which landslides are expected to be triggered. This maximum distance can be estimated from the boundaries proposed in Chapter 7, as a function of earthquake magnitude and rainfall conditions. The denominator in Equation 8.2 then represents the maximum range of the strong-motion parameter within the area affected by landslides. As boundaries obtained in Chapter 7, for regional hazard assessment, were obtained from the present database, this range is normalised over the historical cases compiled during the present study.

Value of the function T ranges between 0 to 1. Slopes out of the area expected to be affected by landslides will have values lower than zero, in this case T will be taken as zero, which represents the minimum ground shaking necessary for landslides to be triggered. For slopes near or just above the seismic source the function T will take a value of 1, which represents the maximum value of the seismic parameter that the earthquake may produce. Values of SP can be estimated by using attenuation relationships, then this function also considers the earthquake magnitude, distance from slope to source, and path and site effects on strong motions if appropriate attenuation equations are available for the area under study.

As discussed in Chapter 7, the maximum distance to which slides can be triggered is a function not only of the magnitude but also of the precedent rainfall conditions and failure mechanisms. The model was developed by grouping data into the three different mechanisms proposed by Keefer (1984) and analysed in Chapter 7, these are Coherent slides, disrupted slides and falls, and lateral spreads and flows. This allows the model to consider hazard style variability. Function T for a given

earthquake may vary depending on the period of time when the earthquake occurs, then temporal variation of T can also be analysed by using the present model.

The slope susceptibility function S was defined by a combination of slope geometry and climatic conditions. In this case the function may be expressed as:

$$S = \frac{1}{2}(G + C) \quad (8.3)$$

where G represents a function of the slope geometry and C represents a function of the precedent climatic conditions. Summation of the two factors was preferred instead of the product in order to allow either of the components to be zero, which is possible, without producing a susceptibility function equal to zero.

The functions G and C were also expressed as normalised values with respect to the maximum range of the geometric or climatic parameters in order to obtain a susceptibility function ranging between 0 and 1. To define the geometric function, G , slope height, slope angle and the product of these two parameters were used independently. These factors were chosen for the model development since they are the most common parameters reported for individual slides, although, slope height is difficult to use when hazard zonation of large areas is carried out because this factor is very difficult and inaccurately defined from small scale maps. For individual slope analysis both factors are available and then different G functions can be applied. The general form of the G function is:

$$G = \frac{GP - GP_{\min}}{GP_{\max} - GP_{\min}} \quad (8.4)$$

where GP is the geometric parameter for the slope under study, and $GP_{\max}$ and $GP_{\min}$ are the maximum and minimum values of the geometric parameter for the slides included in the database compiled in this project. As the geometric characteristics of slopes that failed during historical seismic events have been shown to vary with failure mechanism, as presented in Chapter 3,

different G functions are obtained for the three different mechanisms in which the data were grouped. As G depends on maximum and minimum values compiled by this study, these do not define an absolute range, however, all values were included without excluding extreme cases in order to consider the widest range available. If new data are included into the database, the total range of values may change and consequently function G may be redefined, although the relative position between different cases remains, and consequently the intervals of susceptibility function defined in this model will also remain unchanged. This characteristic of the model is a consequence of using relative values instead of absolute ones.

The function C is a combination of short-term and long-term, total and excess rainfalls. Then, as the climatic function an average value of rainfall values was used and expressed as:

$$C = \frac{1}{4}(LTR + STR + LTE + STE) \quad (8.5)$$

where LTR and STR represent the long- and short-term total rainfalls and LTE and STE represent the long- and short-term excess or deficit. As long-term parameters annual and six months rainfalls were used, whereas monthly rainfall was used as the short-term parameter. Each of the elements in Eq. (8.5) were also defined as relative values in terms of maximum and minimum values within the database for each of the different mechanisms analysed. Each element was expressed as:

$$RP = \frac{RP_{act} - RP_{min}}{RP_{max} - RP_{min}} \quad (8.6)$$

where RP_{act} represents the actual rainfall parameter for the studied condition, and RP_{max} and RP_{min} are the maximum and minimum values of the rainfall parameters for all cases included in the database. As for the geometric function, the maximum range defined is not absolute so with new data this parameter and C function may be redefined, changing the width of the range within historical cases lie but the relative position of different cases remains unchanged.

Expressing each element of Eq (8.5) as relative values over the entire database, the climatic function also becomes relative ranging between 0 and 1. Summation of factors was also preferred in this case in order to allow any of the elements to be zero. The arithmetic average was used because with the available data it was not possible to carry out multivariate analysis in order to determine if any of the elements is more important than the others, thus equal weighting factors were assumed for each one, which lead to this simple average value. When the actual parameter is smaller than the minimum value used to define the functions, a negative value is obtained. In this case the minimum value can be replaced and new equations proposed or a minimum value of zero can be assumed for the element without changing the final result.

A bi-dimensional space with axes T and S, as presented in Figure 8.1, is proposed. This space contains any combination of triggering and susceptibility values within the ranges established by the present database.

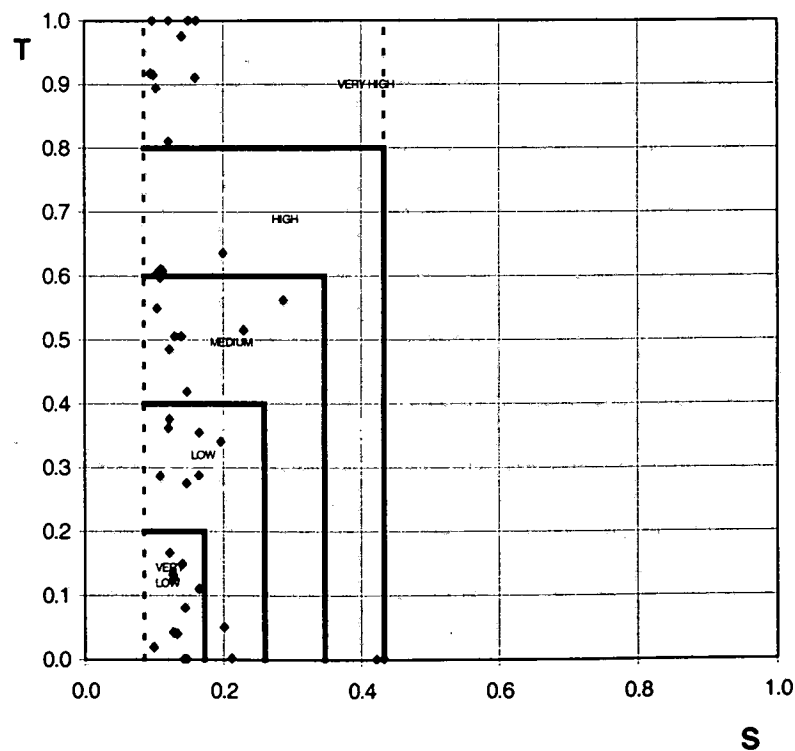


Figure 8.1 Hazard assessment space. Dashed lines represent the lower and upper limits for disrupted slides. Thick continuous lines define boundaries of different hazard levels.

Each point within the space represents an individual slope. All points of the present database lie between the minimum and maximum values which are used to define the upper and lower limits. Points to the right of the upper limit are considered to have a very high hazard due to exceptionally large susceptibility. Points to the left of the lower limit have very low hazard. This zone actually does not contain any case of landslide, so the hazard within it may actually be considered as negligible due to the extremely low susceptibility.

All the historical cases lie within the upper and lower limits, so this space is divided into four zones, which are arbitrarily assumed to have different hazard levels by different combination of triggering and susceptibility values. The zones were obtained by dividing the triggering axis into five levels of 0.2 each, and the susceptibility axis, within lower and upper limits, into four equal intervals.

The zone of Very Low hazard represents slopes far from the seismic source with a very low susceptibility function, which may be the result of low slope angle, low slope height and dry climatic conditions. The Low hazard zone contains slopes closer to the seismic source, thus with higher strong-motion shaking, and low susceptibility. Within this zone are also included slopes far away from the source but with higher susceptibility. Hazard levels increase as the distance to source decreases and the susceptibility of the slope increases. Those slopes very close to seismic source experience high ground shaking and hazard within this zone is Very High.

As the triggering function is obtained from attenuation equations for each of the strong-motion parameters analysed, taking equal intervals of triggering to define different levels does not represent concentric and equidistant areas around the source but areas that increase with distance, which represents a reduction of landslide density with distance, as observed in actual cases.

In Figure 8.1 slopes that failed as disrupted slides, during historical earthquakes are included. It can be seen from those points that values of the susceptibility function vary within a relatively small range. However, this is an apparent interval since values are normalised with respect to extreme values within database, which means that although the susceptibility function shows a maximum value below 0.5, for instance, this value may represent very high susceptibility conditions given by

high and steep slopes or very wet climatic conditions or an unfavourable combination of them. This characteristic is also given by assuming relative rather than absolute values. If extreme values were excluded a more uniform distribution would be found, however; this may exclude some actual cases or restrict the model to regions with climatic conditions below any arbitrary established limit.

8.2 Results

The procedure explained in the previous section was used to defined the boundaries of different hazard levels, considering different seismic, geometric and climatic parameters for the three different general mechanisms used to classify the data. The earthquake parameter was defined by the PGA, PGV and AI. The short-term rainfall was defined by the monthly rainfall, the monthly rainfall excess, and finally, the long-term rainfall parameter was defined by the annual rainfall, the six months rainfall, the annual rainfall excess, and the six months rainfall excess. Limits of each hazard zone for each of the possible combinations are summarised in this section.

8.2.1 Disrupted slides

Based on the mechanism description, data of individual slides were classified into three general mechanism: disrupted slides, coherent slides, and lateral spreads and flows. For each case combinations of triggering and susceptibility functions were analysed in order to establish limits of different hazard levels as discussed in the previous section. For disrupted slides limits are compiled in Table 8.1.

In this case the susceptibility function was defined by using the minimum and maximum values compiled in Table 8.2. These values are the minimum and maximum values of each parameter for cases included in the database compiled for the present study.

As can be seen from Table 8.2, the range of parameters used to define the geometric and climatic functions are very wide including most of the possible geometric and climatic conditions. This makes the proposed model of very general applicability.

Table 8.1 *S value limits for different hazard levels for disrupted slides. Numbers in parenthesis are the total number of data used to define susceptibility intervals. H is the slope height, β the slope angle, $H\beta$ is their product.*

Strong-motion parameter	Climatic parameter	Geometric parameter	Landslide Hazard					
			Negligible	Very Low	Low	Medium	High	Very High
T Values				0.2	0.4	0.6	0.8	
PGA	Annual Rainfall	H (59)	0.085	0.172	0.259	0.345	0.432	
		β (91)	0.079	0.289	0.5	0.711	0.921	
		$H\beta$ (50)	0.087	0.187	0.287	0.387	0.487	
PGV	Annual Rainfall	H (59)	0.085	0.172	0.259	0.345	0.432	
		β (91)	0.079	0.289	0.5	0.711	0.921	
		$H\beta$ (50)	0.087	0.187	0.287	0.387	0.487	
AI	Annual Rainfall	H (59)	0.085	0.172	0.259	0.345	0.432	
		β (91)	0.079	0.289	0.5	0.711	0.921	
		$H\beta$ (50)	0.087	0.187	0.287	0.387	0.487	
PGA	Six months Rainfall	H (59)	0.054	0.168	0.281	0.395	0.508	
		β (91)	0.078	0.286	0.493	0.700	0.908	
		$H\beta$ (50)	0.054	0.159	0.265	0.370	0.475	
PGV	Six months Rainfall	H (59)	0.054	0.168	0.281	0.395	0.508	
		β (91)	0.078	0.286	0.493	0.700	0.908	
		$H\beta$ (50)	0.054	0.159	0.265	0.370	0.475	
AI	Six months Rainfall	H (59)	0.054	0.168	0.281	0.395	0.508	
		β (91)	0.078	0.286	0.493	0.700	0.908	
		$H\beta$ (50)	0.054	0.159	0.265	0.370	0.475	

Table 8.2 *Minimum and maximum values of geometric and climatic parameters used to define susceptibility functions for disrupted slides. N is the number of data used to define limits.*

Parameter	N	Minimum	Maximum
Slope height (m)	75	2.40	1600.00
Slope angle (°)	117	3.00	90.00
$H\beta$	63	125.00	40000.00
Annual Rainfall (mm)	138	1.00	4790.30
Six Months Rainfall (mm)	138	1.00	2569.30
Monthly Rainfall (mm)	138	0.00	540.90
Annual Rainfall Excess (mm)	138	-573.50	758.30
Six Months Rainfall Excess (mm)	138	-217.00	584.70

Monthly Rainfall Excess (mm)	138	-107.10	204.30
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8.2.2 Coherent slides

In Table 8.3 the limits that define the different hazard levels for coherent slides, are compiled. As for disrupted slides, different geometric and climatic parameters were used to define susceptibility functions. The Minimum and maximum values used to define the susceptibility functions are presented in Table 8.4. In this case, the range of values is also very wide and most of the possible geometric and climatic conditions are included in the boundaries.

Comparing minimum and maximum values for disrupted and coherent slides it can be seen that coherent slides have been reported in regions with lower total rainfalls and rainfall excess than disrupted slides, which may be related to the increase of material mobility due to water content increase.

8.2.3 Lateral spreads and flows

Limits for different hazard levels and the minimum and maximum values used to define susceptibility functions are compiled in Tables 8.5 and 8.6, respectively.

8.3 Model applicability

The hazard of individual landslides within the potential area affected by landslides during a seismic event can be assessed by using the present model if climatic and seismic conditions can be estimated for that site. Climatic and seismic parameters can be evaluated by using deterministic or probabilistic approaches. An application example of the model is presented in Section 8.4.

Similar to the model proposed for regional landslide hazard assessment, the present model can be used to evaluate the applicability of different available approaches; in this section as an exploratory exercise, the applicability of the Mora and Vahrson (1994) model is discussed.

Table 8.3 *S value limits for different hazard levels for coherent slides. Numbers in parenthesis are the total number of data used to define susceptibility intervals. H is the slope height, β the slope angle, $H\beta$ is their product.*

Strong-motion parameter	Climatic parameter	Geometric parameter	Landslide Hazard					
			Negligible	Very Low	Low	Medium	High	Very High
T Values			-	0.2	0.4	0.6	0.8	
PGA	Annual Rainfall	H (21)	0.178	0.313	0.448	0.582	0.717	
		β (19)	0.202	0.351	0.500	0.648	0.797	
		H β (10)	0.186	0.355	0.523	0.692	0.860	
PGV	Annual Rainfall	H (21)	0.178	0.313	0.448	0.582	0.717	
		β (19)	0.202	0.351	0.500	0.648	0.797	
		H β (10)	0.186	0.355	0.523	0.692	0.860	
AI	Annual Rainfall	H (21)	0.178	0.313	0.448	0.582	0.717	
		β (19)	0.202	0.351	0.500	0.648	0.797	
		H β (10)	0.186	0.355	0.523	0.692	0.860	
PGA	Six months Rainfall	H (21)	0.111	0.254	0.397	0.540	0.683	
		β (19)	0.124	0.296	0.467	0.639	0.810	
		H β (10)	0.109	0.304	0.499	0.694	0.889	
PGV	Six months Rainfall	H (21)	0.111	0.254	0.397	0.540	0.683	
		β (19)	0.124	0.296	0.467	0.639	0.810	
		H β (10)	0.109	0.304	0.499	0.694	0.889	
AI	Six months Rainfall	H (21)	0.111	0.254	0.397	0.540	0.683	
		β (19)	0.124	0.296	0.467	0.639	0.810	
		H β (10)	0.109	0.304	0.499	0.694	0.889	

Table 8.4 *Minimum and maximum values of geometric and climatic parameters used to define susceptibility functions for coherent slides. N is the number of data used to define limits.*

Parameter	N	Minimum	Maximum
Slope height (m)	24	1.20	300.00
Slope angle (°)	23	7.50	90.00
$H\beta$	13	27.00	11550.00
Annual Rainfall (mm)	55	1.00	1455.00
Six Months Rainfall (mm)	55	1.00	970.80
Monthly Rainfall (mm)	55	0.00	195.60
Annual Rainfall Excess (mm)	55	-573.50	196.00
Six Months Rainfall Excess (mm)	55	-217.00	82.00
Monthly Rainfall Excess (mm)	55	-69.8	41.90

Table 8.5 *S value limits for different hazard levels for lateral spreads and flows. Numbers in parenthesis are the total number of data used to define susceptibility intervals. H is the slope height, β the slope angle, $H\beta$ is their product.*

Strong-motion parameter	Climatic parameter	Geometric parameter	Landslide Hazard				
			Negligible	Very Low	Low	Medium	High
T Values			-	0.2	0.4	0.6	0.8
PGA	Annual Rainfall	H (64)	0.145	0.268	0.391	0.513	0.636
		β (87)	0.123	0.283	0.443	0.602	0.762
		H β (55)	0.136	0.264	0.392	0.519	0.647
PGV	Annual Rainfall	H (64)	0.145	0.268	0.391	0.513	0.636
		β (87)	0.123	0.283	0.443	0.602	0.762
		H β (55)	0.136	0.264	0.392	0.519	0.647
AI	Annual Rainfall	H (64)	0.145	0.268	0.391	0.513	0.636
		β (87)	0.123	0.283	0.443	0.602	0.762
		H β (55)	0.136	0.264	0.392	0.519	0.647
PGA	Six months Rainfall	H (65)	0.137	0.257	0.378	0.498	0.618
		β (87)	0.156	0.313	0.470	0.626	0.783
		H β (55)	0.146	0.267	0.387	0.508	0.628
PGV	Six months Rainfall	H (65)	0.137	0.257	0.378	0.498	0.618
		β (87)	0.156	0.313	0.470	0.626	0.783
		H β (55)	0.146	0.267	0.387	0.508	0.628
AI	Six months Rainfall	H (65)	0.137	0.257	0.378	0.498	0.618
		β (87)	0.156	0.313	0.470	0.626	0.783
		H β (55)	0.146	0.267	0.387	0.508	0.628

Table 8.6 *Minimum and maximum values of geometric and climatic parameters used to define susceptibility functions for lateral spreads and flows. N is the number of data used to define limits.*

Parameter	N	Minimum	Maximum
Slope height (m)	72	0.90	1330.00
Slope angle (°)	97	0.00	90.00
$H\beta$	61	0.15	51870.00
Annual Rainfall (mm)	118	58.10	4382.50
Six Months Rainfall (mm)	118	23.70	1628.30
Monthly Rainfall (mm)	118	0.00	195.60
Annual Rainfall Excess (mm)	118	-573.50	496.50
Six Months Rainfall Excess (mm)	118	-217.00	292.50
Monthly Rainfall Excess (mm)	118	-102.00	41.90

The Mora and Vahrson model described in Chapter 4 is the only available model that considers the climatic conditions for seismic-induced landslide hazard assessment. This model uses as geometric factor the relative relief, which is expressed as the maximum altitudinal difference within a grid cell of 1 km², so from this factor slope heights related to limits of different hazard levels can be estimated. Climatic factors in the Mora and Vahrson model are considered as a triggering factor by combining them with the seismic factor, which is represented by macro seismic intensity. Critical values of monthly and annual rainfalls considered to define different hazard levels by Mora and Vahrson are used for the present discussion.

Critical values of geometric and climatic factors used by Mora and Vahrson are compiled in Table 8.7, and the corresponding geometric, climatic and susceptibility functions from these values are also presented. As the model defined hazard levels by intervals of different parameters, their combination gives a band that determines the boundaries between the different levels. Functions were established by using the minimum and maximum parameters for disrupted slides, which are the most common mechanism of earthquake-induced landslides in Costa Rica from where the model was developed.

Table 8.7 Geometric, climatic and susceptibility functions corresponding to hazard levels proposed by the Mora and Vahrson (1984).

Hazard Level	H (m)	G	Monthly Rainfall (mm)	Annual Rainfall (mm)	Average Annual Rainfall (mm)	Annual Excess (mm)	C	Susceptibility Function
NEGIGLIBLE	0	0	125	0	0	0	0.165	0.083
	75	0.045	250	100	50	50	0.238	0.142
LOW	75	0.045	125	100	50	50	0.180	0.113
	175	0.108	250	200	90	110	0.254	0.181
MODERATE	175	0.108	125	200	90	110	0.196	0.152
	300	0.186	250	300	130	170	0.271	0.229
MEDIUM	300	0.186	125	200	90	110	0.213	0.200
	500	0.311	250	300	130	170	0.286	0.299
HIGH	500	0.311	125	300	130	170	0.228	0.270
	800	0.499	250	400	175	225	0.286	0.393
VERY HIGH	>800	>0.499	125	>400	>175	>225	>0.286	>0.393
			250					

Band boundaries and hazard zones obtained for the Mora and Vahrson model in Table 8.7 are plotted in Figure 8.2, where the present proposal is also presented for comparison purposes. The actual boundaries are simplified in Figure 8.3 in order to define the simple boundaries between each level.

In Figure 8.3 it can be seen that lower and upper limits of the present model and Mora and Vahrson model are almost coincident, which may be explained by the fact that those values are strongly influenced by the extreme values used to define susceptibility functions, which are almost the same for both models. The lower limit is related to low, gentle slopes within very dry regions, whereas the upper limit is related to high, steep slopes in very wet conditions. The range between both limits is controlled by the maximum value of total annual rainfall, which for the actual cases plotted was found for landslides in Costa Rica, which also explains the coincidence.

It is worth noting that Mora and Vahrson do not use the rainfall excess, thus when applying the equations this parameter does not contribute to the susceptibility function due to it becoming zero. This means that if the model considers the rainfall excess and deficits, it should give larger limits that move the boundaries to the right, but the maximum increase would be 0.082 for extremely wet conditions, and 0.043 for extremely dry conditions. As observed from Figure 8.3, these magnitudes are not large enough to make both models coincident.

The Mora and Vahrson model gives smaller susceptibility function limits for each hazard level to the present model, which suggests that the former model tends to underestimate the hazard for a given strong-motion value. Earthquake parameters as triggering factor for the two models are not directly comparable since they use different parameters and triggering definitions.

Mora and Vahrson use macroseismic intensity to represent the seismic factor. In Chapter 5 it was presented that most of the disrupted slides tend to lie within zones of MMI intensities higher than VI, and the lowest intensity to which disrupted slides have been reported was IV. These intensities values are considered by Mora and Vahrson to represent low and very low triggering levels, which seems to be reasonable. However, Mora and Vahrson give a very high triggering level to zones with

intensities larger than X, for which there are no actual cases of disrupted landslides reported mainly due to these intensity levels being very unlikely to occur. This means that slopes near the seismic source will be assigned an intermediate to high triggering level, which reduces their hazard level; such zones actually have a very high triggering level as proposed in the present model.

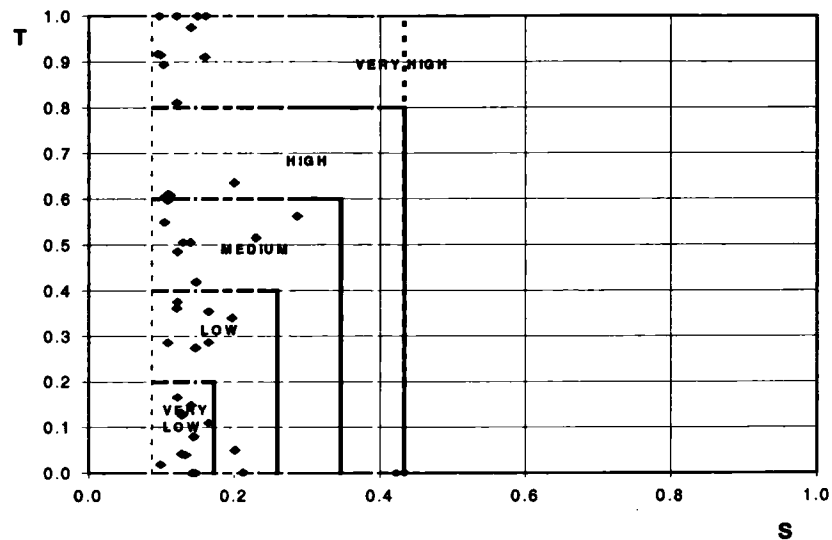


Figure 8.2 Limits of different hazard levels for the Mora and Vahrson (1994) model plotted as susceptibility functions within the proposed hazard space. Negligible, Very Low, Low, Medium, High and Very High levels for the Mora and Vahrson model are those defined by red, yellow, green, blue and brown vertical lines, respectively. Levels stated in this figure represent the zones proposed in the present model.

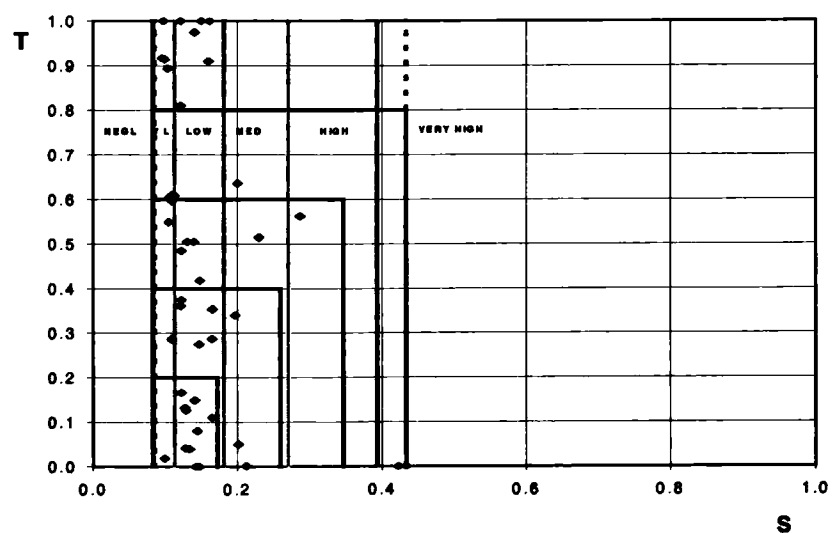


Figure 8.3 Limits of different hazard levels for the Mora and Vahrson model plotted as susceptibility functions within the proposed hazard space. Negligible, Very Low, Low, Medium, High and Very High levels for the Mora and Vahrson models are those defined by red, yellow, green, blue and brown vertical lines, respectively. These levels are shown in this figure.

Considering, for instance, the maximum limits that separate the very low and low hazard levels, the maximum susceptibility value given by the Mora and Vahrson model is 8, which is found by multiplying the triggering value obtained from the sum of the rainfall and seismic factors. The highest possible triggering value for low annual rainfalls is 8, which gives, for the range of very low susceptibility, a maximum hazard value of 64 that classifies within the moderate hazard level. This means that for the low susceptibility zone, the Mora and Vahrson will never give a hazard higher than moderate, even for slopes very close to the seismic source. Similar analyses for different susceptibility levels give similar results.

Taking the most critical geometric and climatic conditions, for a slope to classify within the very high hazard level, using the Mora and Vahrson model, it requires at least a triggering factor value of 5, which represents zones of medium or higher strong-motion shaking, or zones within intensity levels larger than VII, which means that slopes located at moderate to large distances from the seismic source will never be classified as having very high hazard regardless of the susceptibility, which does not coincide with historical observations even in Costa Rica.

8.4 Landslide hazard assessment in Boyacá (Colombia)

Models developed during the present study were presented in Chapter 7 and the previous section. The applicability of these models to evaluate advantages and disadvantages of other available models were also discussed therein. The present section is an exploratory exercise of both models to earthquake-induced hazard assessment.

The objective of this exercise is to show the steps to follow in order to use the models and how the results can be analysed. For this reason the data evaluation will be based on a deterministic approach; however, for actual application the probabilistic analysis of data is suggested in order to include parameter uncertainty in analysis.

The application was performed for slopes within the Tauramena (Casanare) in Colombia, where in 1995 a moderate earthquake, after very dry antecedent rainfall conditions, triggered many

landslides around the epicentral area. The earthquake magnitude of this event was used to define the strong-motion parameters required for the model, climatic factors were obtained from instrumental records within the area, and slope geometry was defined for a particular slope around village Santa Teresa (Boyacá, Colombia). A general map of the area is shown in Figure 8.4.

8.4.1 General Setting

The slope analysed is located about 2.5 km to the east of Santa Teresa on the left bank of the Upía River. The slope plan is shown in Figure 8.5 and cross section in Figure 8.6. As can be seen, this is a very long, uniform and rectilinear slope, limited by two well defined streams in the lower half, whereas the right-hand side of the upper half is limited by a divisor. This slope has a total height of 1000 m and slope angle of about 16°.

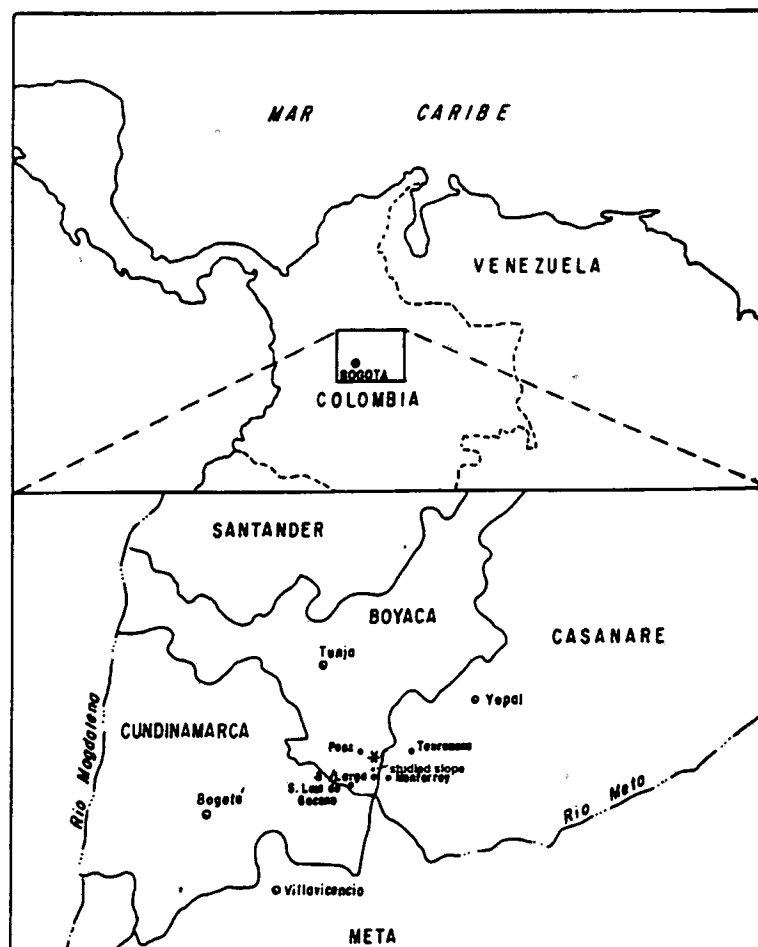


Figure 8.4 General location of the studied slope site. Asterisk is the epicentre of the earthquake considered. (From INGEOMINAS, 1995)

Slopes in this area have been formed by bisection due to fluvial erosion of extensively folded, fractured and interbedded deposits of sandstones, shales and siltstones. In some areas, mudstone layers are also present. In the slope zone these layers dip with angles steeper than 30° . The geology of the slope area is presented in Figure 8.7. It is shown that the slope is located along the axis of the Palmichal anticline, and is composed of interbedded layers, from top to bottom of the El Palmichal Group (Tkp), El Limbo Mudstone Formation (Tal), El Limbo Sandstone Formation (Tarl) and the San Fernando Formation (Tsf).

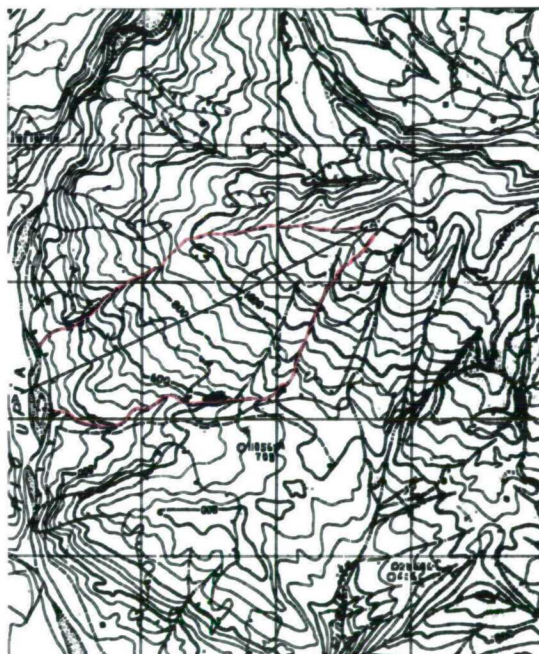


Figure 8.5 Plan view of the analysed slope. Slope is enclosed by red line. Profile in Figure 9.3 corresponds to section along line A-A'.

The Palmichal Group material is mainly composed of three sandstones layer interbedded with shales and siliceous siltstones. The sandstone units are fine grained materials showing a large porosity and permeability. Very high and steep slopes are usually associated with this lithology. The El Limbo Mudstone Formation is constituted by a sequence of interbedded mudstones and sandstones, which present gentle and wavy topographic shapes. Underlying the El Limbo Mudstone Formation is the El Limbo Sandstone Formation, which, lithologically, is composed of quartzite conglomerates and very fractured and friable sandstones. This unit is characterised by high porosity and permeability. Finally the older elements of the sandstone units are presented in the lower part

as the San Fernando Formation, which commonly is seen forming terraces strongly eroded by fluvial erosion.

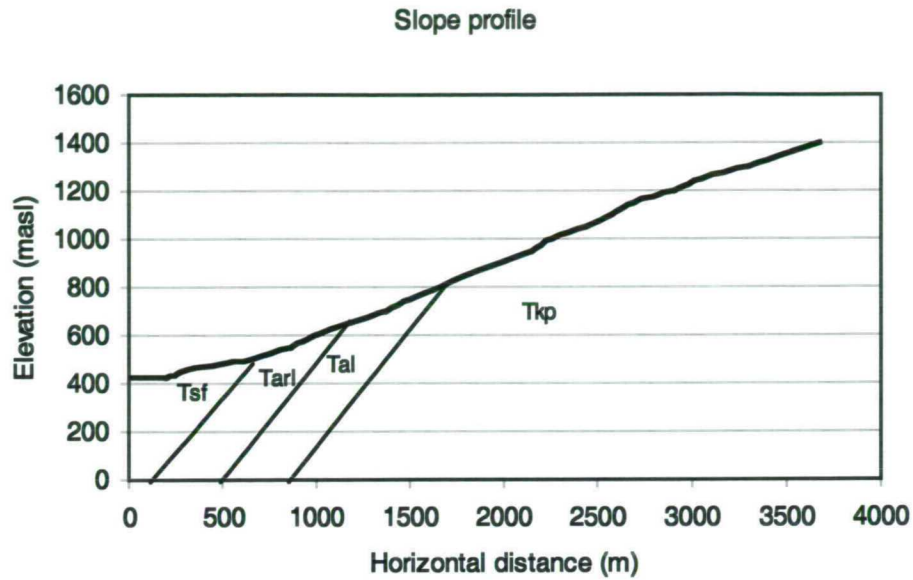


Figure 8.6 Slope cross section along the A-A' line in Figure 8.5.

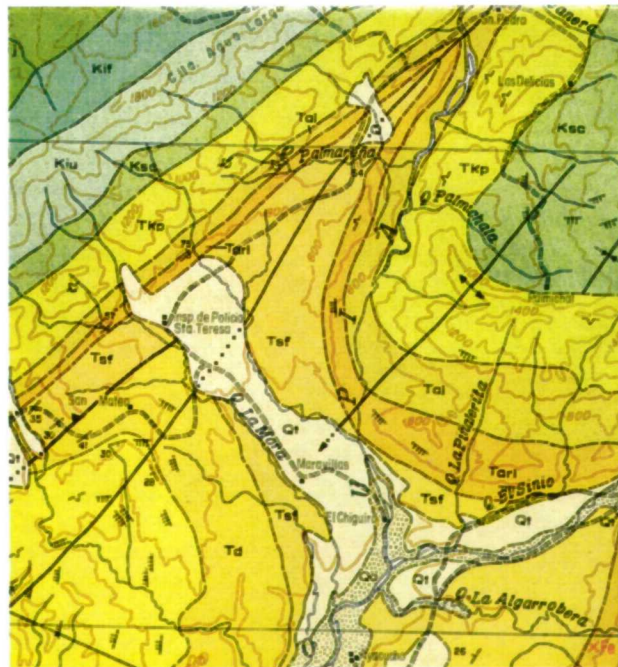


Figure 8.7 Geological map around the slope area.

The region is under the influence of the tectonic activity of the Borde Llanero Fault System, which defines the tectonic boundary along the piedmont from south to north of the country. The north segment of the Guaicaramo Fault belonging to this system was the cause of the 1995 earthquake. The Guaicaramo Fault is associated with thrust movement with a NE strike direction, the fault plane dip towards the west, and has a small right-lateral slip component.

The Borde Llanero Fault System has been the cause of frequent seismic activity for a long time. Some of the most important events associated with this system are the 1927 Gigante, 1834 Valle del Sibundoy, 1967 Colombia (Huila), and 1923 Paratebuena earthquakes (INGEOMINAS, 1995). Many other small seismic events are frequently reported within this area. The area is located within the High Seismic Hazard Zone established by the Colombian Seismic Code (INGEOMINAS, 1995). The 1995 Tauramena earthquake, which was included in the present database, was reported to be a $M_s=6.7$ shallow earthquake. The slope under study is located 15 km southwest of the earthquake epicentre, and lie within the fault rupture projection on the surface. Epicentral distance is required for applying the PGA attenuation relationship for Colombian earthquakes (Ojeda and Martínez, 1995), although this expression was developed with only four strong earthquakes and many small events and tends to under estimate PGA, particularly for the near-field zone, it is the only one currently available for the region so far.

The area has a tropical climate with an average total annual rainfall of 2255.7 mm, and a distribution that shows two maxima in April-May and October-November, and a dry period from November to March. This distribution can be seen in Figure 8.8. Maximum and minimum annual total rainfall for the period 1983-2000 were 3169 mm and 1068 mm, respectively.

8.4.2 Regional hazard assessment

Determination of regional parameters to assess earthquake-induced landslides hazard for this case is based on the results presented in Chapter 7. According to the general description of the slope under study, presented in the previous section, the lithology of the slope material can be identified as susceptible to earthquake-induced landslides as can be seen from Tables 3.2 and 3.3. Due to the high fracturing and the absence of a pre-existing slip surface, this slope may fail as rock fall or

disrupted rock slide. Because of the relatively low slope angle this slope is unlikely to become a rock avalanche. Thus, for the present application this slope can be analysed based on the graphs obtained for disrupted slides.

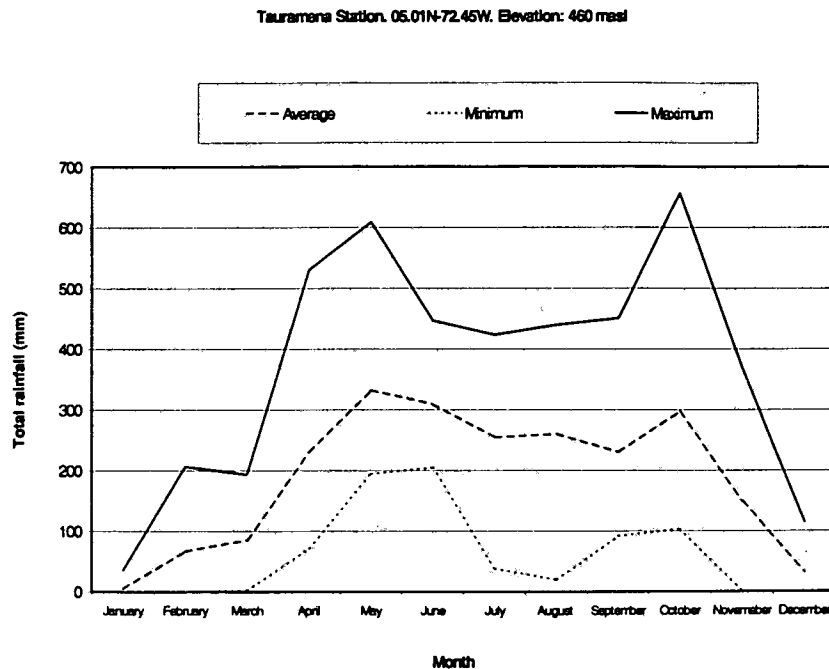


Figure 8.8 Annual rainfall distribution for the Tauramena Station.

Based on the earthquake magnitude, the potential area affected by landslides ranges between 1000 km² and 3500 km², depending on the rainfall conditions. As it was stated in Chapter 7, the maximum distance to which disrupted slides can be triggered is a function of the precedent long- and short-term rainfalls. Based on the earthquake magnitude, the maximum distance at which disrupted slides can be found is between 20 and 50 km. When extreme dry or wet precedent conditions are expected, these distances may be shorter or larger than these values. For very wet conditions, maximum epicentral distance could increase up to about 100 km. As is observed in Figure 8.8 maximum historical monthly rainfalls during rainy months can be larger than 500 mm, which may increase both the area and distance to which slides would be expected. In that case an area between 3500 km² and 10000 km² may be expected. These observations show that the studied slope lies within the affected area for the hypothetical earthquake considered, even after dry climatic conditions. Consequently a more detailed analysis is required to assess its hazard level.

8.4.3 Landslide hazard assessment

Since the triggering and susceptibility functions defined in the previous section depend on the climatic factors, and it was shown that these factors may vary over a wide range, the landslide hazard assessment must be carried out for different precedent climatic conditions. Considering the two extreme cases, a very dry season and a very wet season, the parameters required for this application were determined and compiled in Table 8.8.

Table 8.8 Rainfall parameters and susceptibility functions values for the studied slope

Rainfall Parameter	Value
Maximum annual rainfall	3169.0 mm
Minimum annual rainfall	1068.0 mm
Average annual rainfall	2255.7 mm
Maximum annual excess	913.3 mm
Maximum annual deficit	-1187.7 mm
Maximum monthly rainfall	657.0 mm (October)
Minimum monthly rainfall	0.0 mm (January)
Average monthly rainfall (October)	297.3 mm
Average monthly rainfall (January)	5.4 mm
Maximum monthly excess	359.7 mm
Maximum monthly deficit	-5.4 mm

Based on these values, the maximum and minimum susceptibility function expected for the slope can be calculated, as presented in Table 8.9. These values were obtained by using slope height and angle of 1000 m and 16°, respectively.

Table 8.9 Geometric and susceptibility functions for maximum and minimum rainfall conditions

	Geometric parameter		
	G(H)	G(β)	G(H $\cdot\beta$)
Geometric function (G)	0.62	0.15	0.398
Maximum Susceptibility function (S_{max})	0.768	0.533	0.657
Minimum susceptibility function (S_{min})	0.169	0.144	0.268

Based on the estimates of the maximum and minimum distances to landsliding that can be expected for the hypothetical earthquake, and using the appropriate attenuation relationship for PGA, the triggering functions presented in Table 8.10 can be calculated for this slope. These values were

obtained by using a PGA for the site of 0.072 g and a maximum PGA of 0.125 g, which were also calculated by using the available attenuation equation.

Table 8.10 Triggering function for different precedent rainfall conditions.

Maximum epicentral distance	PGA (g)	Triggering function (T)
20 km	0.063	0.145
50 km	0.034	0.417
100 km	0.019	0.500

With the calculated susceptibility and triggering functions, the hazard level for the slope can be estimated by plotting these functions in the hazard space proposed in the previous section and presented in Figures 8.9 to 8.11.

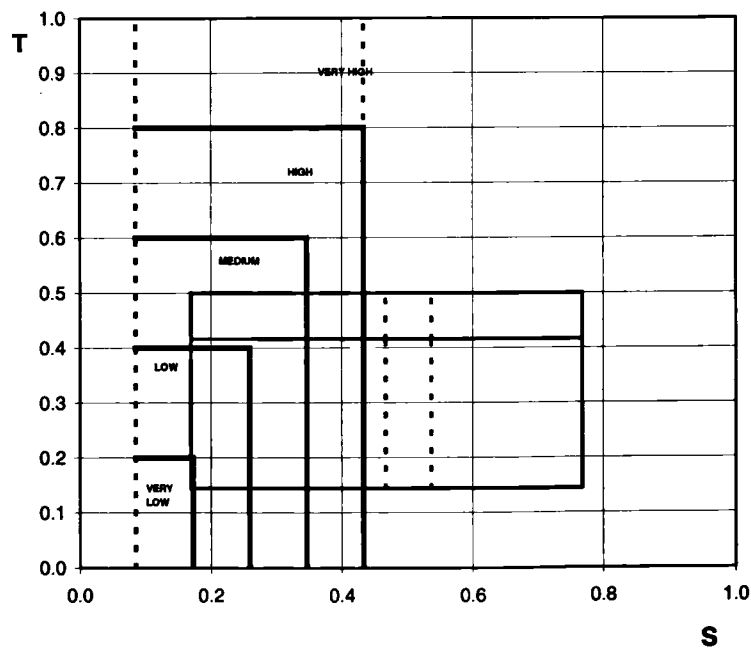


Figure 8.9 Landslide hazard for the studied slope based on slope height . Red lines define the range of variability of the susceptibility function (vertical lines) and triggering function (horizontal lines), for all possible climatic conditions in the region. Horizontal red lines define the range of variability of the triggering function. Vertical red dashed lines represent susceptibility for average annual rainfall for the wettest month (right line) and the driest month (left line).

In Figure 8.9 the landslide hazard for the studied slope is represented by the interval of the triggering and susceptibility functions for different climatic conditions. Vertical solid lines represent the susceptibility function for the wettest (right limit) and driest (left) conditions. The dashed vertical lines represent the susceptibility function for the average annual rainfall for the wettest month (right) and driest month (left). Horizontal red lines represent the triggering function for different climatic conditions. The upper line represents the triggering function when very wet conditions are expected, which was calculated by assuming a maximum epicentral distance to disrupted slides of 100 km. The two lower lines represent the triggering function for the likely expected conditions; the upper of these lines represents wetter climatic conditions than average whereas the lower line represents the limit for dry conditions.

The first observation that can be made is that the landslide hazard is strongly dependent of the climatic conditions, and, as expected, the susceptibility function is more affected by this parameter than the triggering function. Depending on the climatic conditions, the susceptibility may vary from low to very high. For the average annual rainfall this function lies within the very high zone. The triggering function, on the other hand, for normal climatic conditions classifies, within the very low and low levels, although if extreme climatic conditions are expected, the triggering increases to the medium level.

Figure 8.9 corresponds to the case when the susceptibility function is calculated using the slope height as the geometric parameter. The present case corresponds to a very long and high slope with a total height of 1000 m, which is the responsible of the high susceptibility values found. Height used in defining the model was the total height affected by landslides, which in most of the cases was smaller than the total height of the slope affected. Total height was reported for very few cases making almost impossible to use this factor instead of the slide height. This means that considering the total height of the slope, in particular for the present case, increases too much the susceptibility function. If the total height is considered to fail for the present slope the landslide hazard vary from low to very high depending on the climatic conditions.

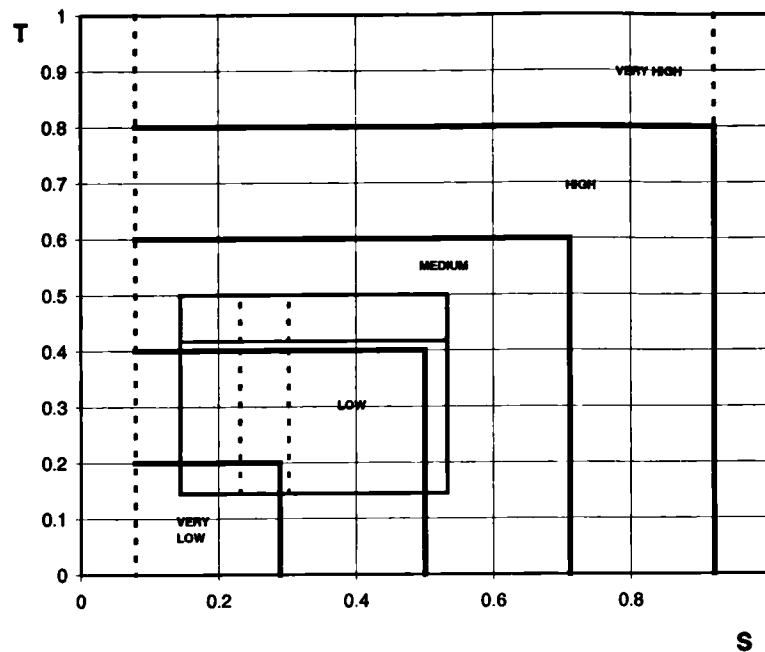


Figure 8.10 Landslide hazard for studied slope based on slope angle. Red lines define the range of variability of the susceptibility function (vertical lines) and triggering function (horizontal lines), for all possible climatic conditions in the region. Horizontal red lines define the range of variability of the triggering function. Vertical red dashed lines represent susceptibility for average annual rainfall for the wettest month (right line) and the driest month (left line).

When slope angle is used instead of slope height for defining the susceptibility function, this parameter becomes more uniform and the historical cases lie within a wider range as can be observed from Figure 8.10. An advantage of using slope angle instead of slope height is that the range between which the parameter varies may be considered as a closed range between 0 and 90 degrees. Possible combinations of triggering and susceptibility functions when slope angle is used as the geometric parameter lie within the space presented in Figure 8.10. Vertical and horizontal lines represent the same conditions as explained for the Figure 8.9. In this case the susceptibility functions range between very low to medium, with the average annual condition within the very low zone. The triggering function is represented by the same intervals as for previous graph. For exceptionally wet conditions the triggering becomes higher lying within the medium zone. When using the slope range as a geometric factor, the landslide hazard for the slope is between very low and low increasing to medium when very wet conditions are present.

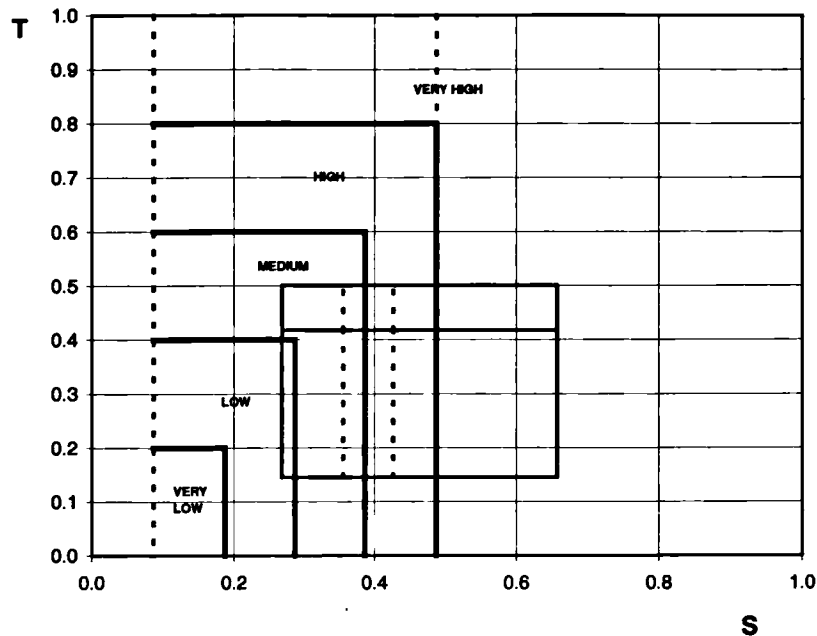


Figure 8.11 Landslide hazard for studied slope based on the product of slope angle and slope height. Red lines define the range of variability of the susceptibility function (vertical lines) and triggering function (horizontal lines), for all possible climatic conditions in the region. Horizontal red lines define the range of variability of the triggering function. Vertical red dashed lines represent susceptibility for average annual rainfall for the wettest month (right line) and the driest month (left line).

A combination of slope height and slope angle was also used to determine the hazard boundaries from historical cases as explained in previous section. This case was also evaluated by using this combination, which was taken as the product of the slope height and the slope angle. As observed in Figure 8.11, the effect of slope height extreme values reduces the susceptibility range where the historical cases lie and increases the susceptibility band for the Santa Teresa present slope. The same limitation analysed for the graph based on slope height may be consider for this parameter, that is, the slope height used for defining the susceptibility function was the height of failed slopes rather than the total slope height, which is the parameter taken for defining the slope susceptibility in the present example. The susceptibility function when, using the product of slope height and slope angle, for the present slope varies from low to very high, with the rainfall varying from medium to high. Landslide hazard for the slope from this graph varies from low to very high depending on the climatic conditions.

8.4.4 Summary

The simple application of the proposed model shows both its advantages and limitations. As can be observed from Figures 8.9 to 8.11 the model allows the variability of the slope susceptibility and minimum seismic triggering, required to cause landslides, to be considered simultaneously.

Although results for the Santa Teresa present example show a wide variability in the triggering function, susceptibility function and landslide hazard, it must be taken into account that the band shown was found by using the most extreme conditions, which possibly will never occur for the zone. This is a combination of extremely wet or dry rainfall seasons, with an extremely wet or dry monthly period. As can be seen from actual historical data, a wet or dry long-term condition does not necessarily leads to a wet or dry short-term condition.

Comparing the different graphs it is apparent that the total height, for the present slope, is not an appropriate parameter for hazard assessment due the implicit assumption that the total height will fail, whereas, historical cases have shown that only a part of the total slope fails. In order to avoid this, a combination of parameters such as the total height and the position of the slope failure from the divide would be desirable. However, the distance to the divide was a parameter that only for few cases was reported and no conclusive evidence could be obtained for considering this possibility.

Based on the present results, total height and the combination of total height with slope angle must be used with care, especially when very high slopes are being analysed. For the Santa Teresa example, the slope profile was obtained from a 1:25,000 topographic map, in which the maximum altitudinal difference that can be estimated was 25 m. If a larger scale map were used, a more detailed section along the slope could be found and the analysis for individual sections could be carried out. This observation may suggests that the model is applicable with higher accuracy when geometric features are obtained from 1:10,000 or larger scale maps.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

This thesis is concerned with the study of earthquake-induced landslides, in particular the identification of those factors that contribute to slope instability and their interrelation. This is a huge subject area to study, which has been for a long time of research interest. The present thesis must be thought of as a series of contributions to different elements of earthquake-induced landslides analysis, rather than an attempt to provide a definitive solution.

Large economical and environmental losses have been produced by seismically-induced landslides produced during or after historical earthquakes. An inventory of these historical cases can be used for assessing the hazard, vulnerability and risk of infrastructure in communities which present similar regional conditions. Qualitative and quantitative analyses of factors commonly involved in past cases is the base for the hazard assessment of slopes potentially affected by seismic loads in the future. Based on these analyses, the four main characteristics of earthquake-induced landslide hazard can be defined: type or style, location, magnitude or intensity and temporal occurrence. These characteristics were identified, analysed and discussed for a large number of historical cases, in order to apply them in the new proposed model.

The starting point of the present databases was the review and study of available studies and inventories of earthquake-induced landslides. Although this issue has been studied for a long time and historical cases are reported from remote times, few studies and inventories have been presented. A brief summary of these documents was included. For this reason, the present databases are considered as an outstanding contribution for future studies of earthquake and landslide relationships. Most of the documents and data sources used for the databases compilation were studied and summarised in an annex document available as an internal report of the

Earthquake Engineering and Engineering Seismology Section at Imperial College. Detailed descriptions of geologic effects of historical earthquakes, and the spatial distribution of earthquake-induced landslides world-wide is also included.

Effects identified as being produced by earthquakes on slopes are the change in stress state, the change in material properties, and variation of groundwater level and slope inclination; other effects have been produced by secondary factors such as tsunamis and seiches, which also act directly on slopes. In order to consider the particular conditions of each site it is imperative that strong-motion characteristics and material behaviour for each site be estimated.

An attempt to include other parameters different to those usually employed in landslide analysis was made in this thesis. It was confirmed that Arias Intensity is a parameter that could correlate well with landsliding, but a detailed analysis of this factor requires the development of more specific attenuation relationships in order to consider local conditions. Many proposals that combine Arias Intensity with Newmark displacements have been used for stability and hazard analyses, but as stated previously, these relationships do not consider the influence of climatic conditions on the critical values used for determining those relationships. This leads to the suggestion that these kinds of relationships can be improved by considering climatic conditions of historical cases, and consequently make them temporally variable.

Peak ground acceleration and velocity have the advantage that more attenuation expressions have been reported for them in different areas, which makes them easier to use. Relationships developed in this thesis based on PGA and AI proved to be more consistent and uniform than those that use PGV, but velocity was available only for limited zones and this may restrict its applicability. With the development of new attenuation relationships for other parameters different from acceleration and velocity, a comparison between different factors and landslides can be improved by using the landslides database compiled for this project. PGV and PGD have the additional inconvenience that they are dependent on the filtering process of instrumental records and actual values may differ considerably from the reported values.

PGA has been commonly used in pseudo-static analysis and critical acceleration defined as a predictive parameter, but it has been shown from back analyses of actual cases included in this project that this parameter may be influenced by simplifications during its definition. Additionally, a local PGA higher than the critical value does not necessarily lead the slope to fail. On the other hand, many slopes, particularly in brittle materials, have shown a tensile failure rather than shear failure, making the pseudo-static approaches inappropriate for analysis.

From the historical data compiled, it was shown that landsliding depends not only on the seismic shaking but also on the slope susceptibility, which in turn is strongly influenced by climatic conditions, thus use of strong-motion parameters alone is limited. Earthquake magnitude and climatic conditions were combined in order to improve relationships proposed to evaluate areas affected by landslides and maximum distance to particular types of landslides. It was shown that small earthquakes are capable of producing landslides in very susceptible slopes: a minimum magnitude of 2.9 was reported in this thesis. However, for magnitudes lower than 4.0 only single slides have been reported, hence, a minimum magnitude of 4.0 seems to be the effective lower limit when magnitude is used as predictive parameter. High slope susceptibility causes landslides to occur under macroseismic intensities lower than the level where these geologic effects are considered by different intensity scales, which makes this parameter inappropriate for predictive purposes.

Although Kriging seems to be a promising approach for the prediction of strong-motion parameters in specific localities, its use has not been extensive and more analysis is required to warrant its applicability. Available attenuation equations were used for the prediction of strong-motion parameters, although in some regions no equations are available and for many cases it was impossible to include strong motion in the models. In other cases, the available equations require some parameters that were not easy to find for historical cases, which suggests that parameters compiled in this study must be continuously complemented in order to re-evaluate the present findings and, if necessary, re-evaluate the proposed relations. Variations in attenuation equation were explored in an attempt to improve the predictions, but the extra work required is not compensated by an improved prediction, consequently, the use of published attenuation equations

seems to be the best tool available for this purpose, especially if source, path and site effects on strong motion are included in these equations.

Many intrinsic and extrinsic factors contribute to the slope susceptibility, which were identified and analysed from historical cases compiled in this study. Slope material and lithological sequence have been shown to define not only the slope susceptibility but also the failure mechanism. Furthermore, it was shown that the same deposit requires different shaking intensity to be affected by different failure mechanisms as a function of precedent climatic conditions. Based on lithology and geometric slope characteristics, different hazard scenarios can be identified within the area under study, which shows how the hazard is not only spatially variable but also temporally variable for a specific slope. This variability was considered in developing the model proposed. Most of the cases compiled in the present database have been shown to be controlled by structural geological features. Some were controlled by the presence of planes of weakness, others were controlled by the aspect ratio, which defined the relation between discontinuities and slope direction, and many other were controlled by the groundwater regime. Although a model which considers the influences of these factors would be desirable, lack of data makes this a very difficult task at this time.

Critical scenarios defined from the geological factors can be complemented by the geometric features of affected slopes. Slope height and angle are the most commonly reported parameters and they were used in developing the proposed model. Different failure mechanisms within a specific lithological unit are dependent on these factors. Common and minimum values reported for historical cases were compiled, and they can be used simultaneously with the lithology description for preliminary hazard assessments. These values were also used to evaluate and update previous relationships. Other geometric features were identified to be important, such as slope shape, although for most of the cases these factors were not found.

Rainfall conditions have been extensively analysed for rainfall-induced landslides, but only one of the available models for earthquake-induced landslides hazard assessment considers this factor. From analysis of regional areal extension of landsliding and individual landslides it was found that a combination of long-term and short-term rainfall conditions plays an important role in landslide

during seismic events. A new regional hazard assessment model was proposed that includes these short-and long-term rainfall conditions.

This first model classifies as a Grade-1 model and is used to estimate the potential area affected by landslides and the maximum distance from the epicentre and from the fault projection to which different types of landslides can be expected during a given earthquake. Although the area affected by landslides estimated by using the present model is not a very useful parameter, due to possible eccentricity of this area with respect to seismic source, this parameter can be use as a preliminary assessment of the potential damage due to landsliding. Maximum distance to different types of landslides was incorporated into the second model proposed in order to establish the minimum or critical level of seismic intensity required to trigger each type of landslide under different climatic conditions.

The applicability of different methods for earthquake-induced landslides hazard assessment was evaluated based on the results and analysis of the present database, and a new model was proposed, which looks to improve previous models by considering site strong-motion parameters, which with appropriate attenuation relationships are capable of taking into account source, path and site effects on strong motion.

A major difference between previous models and the new one is the inclusion of climatic factors into the analysis not only as a factor that alters the slope susceptibility but also influences minimum level of shaking required for a landslide to be induced. Considering climatic factors also gives the opportunity of identifying and assessing hazard of different failure mechanisms. Consequently, the new model provides the opportunity to assess not only the hazard intensity but also the style, and the spatial and temporal variability of each type of hazard. In order to consider the uncertainty in those variables included in the analysis, stochastic analyses for determining these parameters can be carried out.

Hazard intensity, in this second model, is defined as a qualitative level ranging from negligible to very large hazard. This method is based on the estimation of a triggering and susceptibility

functions. Plotting these two functions in a hazard space gives the ranges of triggering and susceptibility which determine different hazard levels. The triggering function was defined in term of site strong-motion parameters, whereas the susceptibility function was defined in terms of geometric factors such as the slope height and slope angle, and in terms of both long- and short- term total and excess rainfalls. The triggering and susceptibility functions were defined as relative values respect to the maximum possible range of values of each parameter included because that allows each function to range within a closed band of dimensionless or parameterised values between 0 and 1.

The present data and results can be used to improve previous and present models in order to include other factors that were identified to influence the landsliding process. Particularly, use of data to develop a model based on multivariate statistical analysis is strongly advisable in order to assess quantitatively landslide hazard. This kind of models require the compilation of negative evidence of slopes that have not failed during historical earthquakes.

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ANNEX A

KRIGING FOR SPATIAL VARIABLE PREDICTION

This Annex issues with theoretical basis for Kriging technique use for spatial variable prediction. This technique has shown to be very useful in seismic hazard analysis studies and earthquake-induced landslides zonation practice. However, its application for these purposes have been so far very limited and special studies that explore its advantages and limitations is required. Theoretical concept expressed in this section are completely based on Davis (1986).

The word Kriging is synonymous with optimal prediction. It is a method of interpolation that predicts unknown values from data observed at known localities. Ordinary Kriging is linear because its estimates are weighted linear combinations of the available data; it is unbiased since it tries to have the mean residual or error, m_R , equal to zero; it is the best estimate because it aims at minimising the variance of errors, σ^2_R .

In practical situations we never know the true answers or the actual errors before we attempt our estimation. This means that we never know m_R and therefore cannot guarantee that it is exactly zero. Nor do we know σ^2_R , therefore, we cannot minimise it. Kriging builds a model of the available data and works with the average error and the error variance for the model. Ordinary Kriging uses a probability model in which the bias and the error variance can both be calculated and then choose weights for the nearby samples that ensure that the average error for our model, m_R , is exactly zero and that our modelled error variance, σ^2_R , is minimised.

A random function model is used since this type of model enables to express the error, its mean value, and its variance. In this kind of model, the available data are viewed as the result of some

random process. At every point where there is not a sample, the estimative of the unknown true value is done by using a weighted linear combination of the available samples:

$$\hat{v} = \sum_{i=1}^n w_i v_i \quad (\text{A.1})$$

the set of weights, w_i , is allowed to change as unknown values at different locations are estimated.

If the error or residual, r , of any particular estimated value is defined to be the difference between the estimated value and the true value at that location:

$$\text{Error of } i\text{-th estimate} = r_i = \hat{v}_i - v_i \quad (\text{A.2})$$

then the average error of a set of k estimates is:

$$\text{Average error} = m_R = \frac{1}{k} \sum_{i=1}^k r_i = \frac{1}{k} \sum_{i=1}^k (\hat{v}_i - v_i) \quad (\text{A.3})$$

however, this equation involves quantities that are unknown, namely the true values $v_1, v_2, \dots, v_k$.

The probabilistic solution to this problem consists of conceptualising the unknown values as the outcome of a random process and solving the problem for the conceptual model. For any point t which it is attempted to estimate the unknown value, the model is a stationary random function that consists of several random variables, one for the value at each sample location, $V(x_1), V(x_2), \dots, V(x_n)$, and one for the unknown value at the point where the value is being tried to estimate, $V(x_0)$. Each of these random variables has the same probability law; at all locations, the expected value of the random variable is $E\{V\}$. The expected value of a random variable is its mean or average outcome, that is the weighted average of the n possible outcomes, with each outcome being weighted by its probability of occurrence:

$$E\{V\} = \bar{m} = \sum_{i=1}^n p_i V_i \quad (\text{A.4})$$

Any pair of random variables has a joint distribution that depends only on the separation between the two points and not on their locations. The covariance between pairs of random variables separated by a particular distance, h , is $\tilde{C}_v(h)$.

Every value in this model is seen as the outcome of a random variable; the samples are outcomes of random variables, as is the unknown true value. The estimate is also a random variable since it is a weighted linear combination of the random variables at the available sample locations:

$$\tilde{V}(x_0) = \sum_{i=1}^n w_i V(x_i) \quad (\text{A.5})$$

similarly, the estimation error, defined as the difference between the estimate and the random variable modelling the true value, is also a random variable:

$$R(x_0) = \tilde{V}(x_0) - V(x_0) \quad (\text{A.6})$$

By substituting the previous equation which expressed $\tilde{V}(x_0)$ in terms of other random variables, $R(x_0)$ can be expressed solely in terms of the original $n+1$ random variables in the random function model:

$$R(x_0) = \sum_{i=1}^n [w_i V(x_i) - V(x_0)] \quad (\text{A.7})$$

The error that is made when the unknown value at x_0 is estimated is an outcome of the random variable $R(x_0)$.

The error at any particular location has an expected value of zero, which can be guaranteed by doing:

$$\sum_{i=1}^n w_i = 1 \quad (\text{A.8})$$

The error variance, σ^2_R , of a set of k estimates can be written as:

$$\sigma^2_R = \frac{1}{k} \sum_{i=1}^k (r_i - m_R)^2 = \frac{1}{k} \sum_{i=1}^k \left[\vartheta_i - \nu_i - \frac{1}{k} \sum_{i=1}^k (\vartheta_i - \nu_i) \right]^2 \quad (\text{A.9})$$

$\nu_1, \nu_2, \dots, \nu_k$ are the true values and $\vartheta_1, \vartheta_2, \dots, \vartheta_k$ are the corresponding estimates. If a mean error of zero is assumed:

$$\sigma^2_R = \frac{1}{k} \sum_{i=1}^k (r_i - 0)^2 = \frac{1}{k} \sum_{i=1}^k [\vartheta_i - \nu_i]^2 \quad (\text{A.10})$$

again in this case, the true values are unknown then use of random function models are used. There exist n+1 random variables, n of which model the behaviour of the phenomenon at the nearby sample locations and one of which models its behaviour at the location whose value is trying to be estimated.

The variance of a weighted linear combination is given by:

$$\text{Var} \left\{ \sum_{i=1}^n w_i V_i \right\} = \sum_{i=1}^n \sum_{j=1}^n w_i w_j \text{Cov} \{ V_i V_j \} \quad (\text{A.11})$$

This expression can be transformed to:

$$\sigma^2_R = \sigma^2 + \sum_{i=1}^n \sum_{j=1}^n w_i w_j - 2 \sum_{i=1}^n w_i \tilde{C}_{i0} \quad (\text{A.12})$$

Once the random function model parameters are chosen, especially the variance σ^2 and all the covariances $\tilde{C}_{ij}$, Equation A.12 gives an expression for the error variance as a function of n variables, namely the weighting values $w_1, w_2, \dots, w_n$.

The minimisation of a function of n variables is usually accomplished by setting the n partial first derivatives to zero. This produces a system of n equations and n unknowns that can be solved. However, solution must be restricted to possible solutions for which the set of weights sum to 1. Such problem of constrained optimisation can be solved by the technique of Lagrange parameters. Here, a new unknown variable is introduced into the equation for σ^2_R . This new variable is called μ , the Lagrange parameter, which introduced into Equation A.12 leads to:

$$\sigma^2_R = \sigma^2 + \sum_{i=1}^n \sum_{j=1}^n w_i w_j - 2 \sum_{i=1}^n w_i \tilde{C}_{i0} + 2\mu \left(\sum_{i=1}^n w_i - 1 \right) \quad (\text{A.13})$$

under this condition, there are $n+1$ equations and $n+1$ unknowns. Furthermore, setting the partial first derivative to zero with respect to μ will produce the unbiased condition:

$$\frac{\partial(\sigma^2_R)}{\partial\mu} = \frac{\partial \left(2\mu \sum_{i=1}^n w_i - 1 \right)}{\partial\mu} = 2 \sum_{i=1}^n w_i - 2 \quad (\text{A.14})$$

setting this equation to zero produces the unbiased condition:

$$\sum_{i=1}^n w_i = 1 \quad (\text{A.15})$$

since the unbiased condition is already included in the $n+1$ equations that result from the differentiation of σ^2_R , the solution of these $n+1$ equations will produce the set of weights that minimises σ^2_R under the constrain that the weights sum to 1. This solution will also provide a value for μ_0 .

The differentiation of Equation A with respect to w_1 is given by:

$$\frac{\partial(\sigma^2_R)}{\partial w_1} = 2 \sum_{j=1}^n w_j \tilde{C}_{1j} - 2\tilde{C}_{10} + 2\mu \quad (\text{A.16})$$

setting this to zero produces:

$$2 \sum_{j=1}^n w_j \tilde{C}_{1j} - 2\tilde{C}_{10} + 2\mu = 0$$

$$\sum_{j=1}^n w_j \tilde{C}_{1j} + \mu = \tilde{C}_{10} \quad (\text{A.17})$$

similar equations are obtained by differentiating Equation A.13 with respect to other weights.

The set of weights that minimise the error variance under the constraint that they sum to 1 therefore satisfies the following $n+1$ equations:

$$\sum_{j=1}^n w_j \tilde{C}_{ij} + \mu \quad \forall i = 1, 2, \dots, n \quad (\text{A.18})$$

$$\sum_{i=1}^n w_i = 1$$

this system of equations, referred as the Ordinary Kriging System, can be written as:

$$C \cdot w = D$$

(A.19)

$$\begin{bmatrix} C_{11} & \dots & C_{1n} & 1 \\ \vdots & & \vdots & \vdots \\ C_{m1} & \dots & C_{mn} & 1 \\ 1 & \dots & 1 & 0 \end{bmatrix} \cdot \begin{bmatrix} w_1 \\ \vdots \\ w_n \\ \mu \end{bmatrix} = \begin{bmatrix} C_{10} \\ \vdots \\ C_{n0} \\ 1 \end{bmatrix}$$

solving this equation is obtained:

$$w = C^{-1} \cdot D \quad (A.20)$$

To minimise the modelled error variance, it is necessary to choose the $(n+1)^2$ covariances that will describe the spatial continuity of the random function model. In practice this is done by choosing a function $C(h)$, and calculating all the requires covariances from this function. Once the $(n+1)^2$ covariances have been chosen, the C and D matrices can be built. The set of weights that will produce unbiased estimates with the minimum error variance for the random function model is given by Equation A.20.

As it is assumed that the random variables in the random function model all have the same mean and variances, these two assumptions allows to develop expressions of Kriging in terms of the model variogram, γ_{ij} , which is given by:

$$\gamma_{ij} = \frac{1}{2} E\{[V_i - V_j]^2\} = \sigma^2 - C_{ij} \quad (A.21)$$

In terms of the variogram, the ordinary kriging system can be written as:

$$\sum_{j=1}^n w_j \gamma_{ij} - \mu = \gamma_{i0} \quad \forall i = 1, 2, \dots, n \quad (\text{A.22})$$

$$\sum_{i=1}^n w_i = 1$$

with the modelled error variance given by:

$$\sigma^2_R = \sum_{i=1}^N w_i \gamma_{i0} + \mu \quad (\text{A.23})$$

The basic variograms models can be divided into two types; those that reach a plateau and those that do not. Variograms of the first type are often referred to as transition models. The plateau they reach is called the sill and the distance at which they reach this plateau is called range, these values are shown schematically in Figure A.1. some of the transition models reach their sill asymptotically. For each model, the range is arbitrarily defined to be that distance at which 95% of the sill is reached.

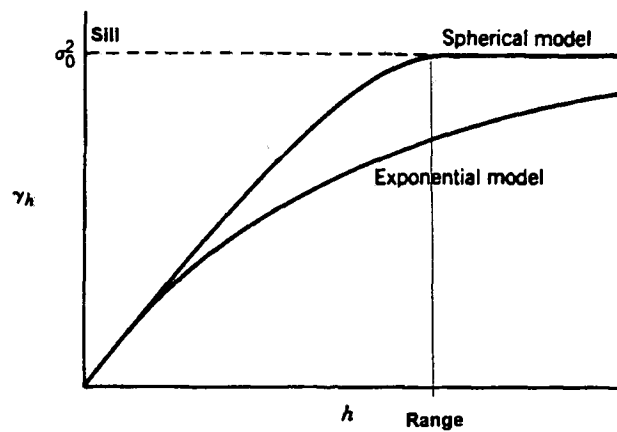


Figure A.1 Spherical and exponential variograms. Both models show equal sill but different range.

Variogram models of the second type do not reach a plateau, but continue increasing as the magnitude of h increases. Such models are often necessary when there is a trend of drift in the data values.

The variogram value for $h=0$ is strictly zero, however, sample variograms present a discontinuity at the origin. This discontinuity is modelled by a constant, C_0 , which is included in the variogram model.

The most common basic models used in Kriging technique are the spherical, exponential, Gaussian and lineal models. The standardised equation for the spherical model is given by:

$$\gamma(h) = \begin{cases} C_0 + C_1 \left(1.5 \frac{h}{a} - 0.5 \left(\frac{h}{a} \right)^3 \right) & \text{if } h \leq a \\ C_0 + C_1 & \text{if } |h| > a \end{cases} \quad (\text{A.24})$$

where a is the range. This model is shown in Figure A.1. It has a linear behaviour at small separation distances near the origin but flattens out at large distances, and reaches the sill at a .

The exponential model is expressed as:

$$\gamma(h) = \begin{cases} 0 & \text{if } |h| = 0 \\ C_0 + C_1 \left(1 - \exp\left(-\frac{3h}{a}\right) \right) & \text{if } |h| > 0 \end{cases} \quad (\text{A.25})$$

this model reaches its sill asymptotically, with the practical range a . Like the spherical model, the exponential model is linear at very short distances near the origin, however, it rises more steeply and then flattens out more gradually, as can be shown in Figure A.1.

Spherical and exponential model with different sill and range values were used for seismic parameters prediction of individual landslides. As shown in Chapter 2, this technique may be very useful for this purpose, however, definition of appropriate sill and range values requires additional

studies. It was also shown that if model is applied to residuals from a base trend, predictions tend to improved, however, selection of sill and range values, in this case, also requires additional analysis.

ANNEX B

ATTENUATION RELATIONSHIPS USED FOR
STRONG MOTION PARAMETERS PREDICTION

For specific landslide locations prediction of seismic parameters was required. This prediction was based on mainly attenuation relationships for particular regions where landslides were reported. When possible attenuation equations obtained by regression of actual data from particular earthquakes were used for calibrate predictions from available attenuation expressions. Particularly for PGV, energy density and duration. Due to lack of data, parameters used for developing the proposed model were PGA, PGV and AI that are parameters for which attenuations equation for different regions were obtained. This annex issues with the reference of expressions used for obtaining these parameters. No detailed description of each expression was made due to space limitations but references are given as support of these expressions.

B.1 Attenuation relationships for Peak Ground Acceleration

Expressions used for prediction of PGA are summarised in Table B.1. References consulted for equation application are also shown.

Table B.1 Attenuation relationships for PGA predictions.

Country/Region	Attenuation equation	Reference
Western USA	Boore <i>et al.</i> , 1993	Douglas, 2001
Europe and Middle East	Ambraseys & Bommer, 1991	Douglas, 2001
New Zealand	Zhao <i>et al.</i> , 1997	Douglas, 2001
Argentina	Youngs <i>et al.</i> , 1997	Douglas, 2001
Chile	Youngs <i>et al.</i> , 1997	Douglas, 2001
Perú	Youngs <i>et al.</i> , 1997	Douglas, 2001
Colombia	Ojeda & Martínez, 1999	Ojeda & Martínez, 1999
Japan	Fukushima & Tanaka, 1990	Fukushima <i>et al.</i> , 1998
Costa Rica	Schmidt <i>et al.</i> , 1997	Schmidt <i>et al.</i> , 1997
El Salvador	Dahle <i>et al.</i> , 1995	Douglas, 2001
Guatemala	Dahle <i>et al.</i> , 1995	Douglas, 2001
Nicaragua	Dahle <i>et al.</i> , 1995	Douglas, 2001
China	Crouse & McGuire, 1996	Douglas, 2001
Nepal	Jain <i>et al.</i> , 2000	Douglas, 2001
Indian Himalayas	Jain <i>et al.</i> , 2000	Douglas, 2001
USA, Washington, Alaska	Crouse, 1991	Douglas, 2001
Eastern USA	Free, 1996	Douglas, 2001
Eastern Canada	Free, 1996	Douglas, 2001

B.2 Attenuation relationships for Peak Ground Velocity

Expression used for prediction of PGV are summarised in Table B.2. References consulted for equation application are also shown.

Table B.2 Attenuation relationship for PGV predictions.

Country	Attenuation equation	Reference
Italy	Sabetta & Pugliese, 1987	Sabetta and Pugliese, 1987
Japan	Midorikawa, 1993	Fukushima <i>et al.</i> , 1998
Western USA	Ambraseys, 1990	Ambraseys, 1990

B.3 Attenuation relationships for Arias Intensity

Predictions of Arias Intensity values were done by using the Wilson and Keefer (1985) equation. This expression has the inconvenience that for near-field locations the equation is not truncated, so the Scott (1999) equation was used for constraining Arias Intensity values for short distances.

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ANNEX C

LANDSLIDE HAZARD ASSESSMENT MODELS

Many methods for landslide hazard assessment models have been proposed. Some of them are summarised in the present Table C.1. Conventions of abbreviations used in this Annex are as follows:

Method

Number in parenthesis correspond to the following references:

(1) Varnes, 1984; (2) Humbert, 1977; (3) Carrara and Merenda, 1974; (4) Stevenson, 1977; (5) Carrara *et al.*, 1978; (6) Wislocki and Bentley, 1991; (7) Brand, 1988; (8) Mateos and Ferrer, 1994; (9) Bosi, 1978; (10) Barisone and Bottino, 1990; (11) Hungr, 1997; (12) Lee, 1997; (13) Mackay, 1997; (14) Mostyn and Fell, 1997; (15) Hansen, 1984; (16) Soldati, 1999; (17) Aleotti *et al.*, 2000a; (18) Aleotti *et al.*, 2000b; (19) Ali Jawaid, 2000; (20) Anbalagan *et al.*, 2000; (21) Ayalew, 2000; (22) Basu, 2000; (23) Dhakal *et al.*, 2000; (24) Froidi and Bonini, 2000; (25) Janova, 2000; (26) Kullar *et al.*, 2000; (27) Liu, 2000; (28) MahdaviFar, 2000; (29) Mazzoccola and Sciesa, 2000; (30) Michael-Leiba *et al.*, 2000; (31) Parkhurst and Flavell, 2000; (32) Pletty and Murphy, 2000; (33) Rybar *et al.*, 2000; (34) Sarkar *et al.*, 2000; (35) Thurston and Degg, 2000; (36) Uromeihy, 2000; (37) Wu *et al.*, 2000; (38) Yi *et al.*, 2000; (39) Zhou *et al.*, 2000; (40) Zezere *et al.*, 2000; (41) Ramírez and González, 1989; (42) Caro *et al.*, 1989; (43) Martínez and García, 1989; (44) Caro and García, 1989; (45) Montero and Cortés, 1989; (46) Shuk, 1996; (47) Crozier, 1984.

Type of analysis

HA: Heuristic Approach

LI: landslides Inventory

D: Deterministic

SA: Statistical Analysis Technique

GA: Geomorphic Analysis

LDiA: Landslides Distribution Analysis

LD_eA: Landslides Density Analysis

QMC: Qualitative Map Combination

MSA: multivariate Statistical Analysis

BSA: Bivariate Statistical Analysis

Hazard Assessment value

H: High

VH: Very High

M: Medium

L: Low

VL: Very Low

MC: most Common

C: Common

LC: Least Common

S: Stable

SS: Semi-Stable

GS: Generally Stable

MS: Moderately Stable

MU: Moderately Unstable

U: Unstable

N: None

RS: Relatively Stable

HU: Highly Unstable

US: Uncertain Stability

AL: Active Landsliding

DA: dormant Landsliding

SLP: Some Landsliding Potential

NL: No Landsliding

DIZ: Direct Impact Zone

IIZ: Indirect Impact Zone

FZ: Flow Zone

RF: Rock Fall Zone

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Method	Type of analysis	Technique Characteristics	Scale	Hazard assessment value
IAS, 1988 (37)	SA	MSA		Qualitative (S, SU, U, HU, depending on Information value)
Yi et al., 2000/Zhou & Yan, 2000. (38)(39)	HA	GA		Qualitative (description based on past even
Zezere et al., 2000 (40)	SA	MSA	1:10.000	Quantitative
Ramírez & González, 1989 (41)	HA	QMC	1:25.000	Qualitative (I to VI)
Caro et al., 1989 (42)	D		1:5.000	Quantitative/Qualitative
Martínez and García, 1989 (43)	HA	GA	1:10.000	Qualitative (H, M, L)
Caro & García, 1989 (44)	HA	GA	1:10.000	Qualitative (VH, H, M, L, VL)
Montero & Cortés, 1989 (45)	HA	GA	1:10.000.000	Qualitative (VH, H, M, L, VL)
MTN, 1968 (46)	SA	BSA	1:5.000/1:10.000	Quantitative
Crozler, 1984 (47)	HA	GA		Qualitative (I to VI)

Method	Type of analysis	Technique Characteristics	Scale	Hazard assessment value
Khullar, 2000 (26)	HA	QMC		
		Hazard divided into three levels by superimposing maps of contributing factors which are classified in three levels themselves.	1:50.000	Qualitative (H, M, L)
Liu, 2000 (27)	HA	QMC		Qualitative (five levels defined)
		Hazard level defined by using Matter-elements technique		
MahdaviFar, 2000 (28)	HA	QMC		Qualitative (VL, L, M, H, VH depending on values of fuzzy function)
		Hazard levels defined by using Fuzzy logic technique		
RHAP, 1999 (29)	HA	GA		Qualitative (H, M, L)
		Slope susceptibility defined by number of contributing factors identified in each area		
RES, 1993 (29)	HA	GA		Quantitative
		Stability indexes for rock slopes defined by combination of weighted factors. Weighting values defined by analysing the problem under the Rock Engineering System approach.		
Matterock, 1998 (29)	HA	GA		Qualitative (H, M, L)
		Hazard level defined by volume of potential rock slide		
Michael-Leiba et al., 2000 (30)	LI	LDIA		Quantitative
		Hazard defined by annual probability obtained from recurrence functions for different terrain units		
Allot & Lomax, 1997 (31)	HA	GA		Qualitative (VH, H, M, L, VL, depending on scoring)
		Slope susceptibility defined by adding scores given to contributing factors for cut and embankments		
Petley & Murphy, 2000 (32)	HA	GA	1:25.000	Qualitative (0 to 4)
		Hazard magnitude defined by potential volume of different mechanism and frequency is associated to recurrence of triggering factors capable of inducing movements. Hazard levels defined by combining magnitude and frequency		
Rybar et al., 2000 (33)	HA	GA	1:50.000	Qualitative (S, P, U)
		Slope susceptibility defined by combining contributing factors		
Sarkar et al., 2000 (34)	HA	GA	1:25.000	Qualitative (L, M, H)
		Hazard intensity defined by expected rate of movement, which is defined by geomorphological description		
Thurston & Degg, 1996 (35)	LI	LDeA	1:50.000	Qualitative (VL, L, M, H, VH)
		Hazard levels defined by adding weighted contributing factors. Weighting factors defined by an Index obtained by density of each factor within landsliding areas.		
Uromailhy, 2000 (36)	HA	QMC	1:50.000	Qualitative (VL, L, M, H, VH, depending on total score)
		Hazard levels defined by adding scores of contributing factors for different terrain units		
Wu, et al., 2000 (37)	HA	QMC		Qualitative (VS, S, FS, US, VUS, depending on dimensionless grey class standard)
		Hazard level defined by using Grey cluster technique		

Method	Type of analysis		Technique Characteristics	Scale	Hazard assessment value
Froelich et al., 1978 (15)	HA	GA	Relative slope stability defined by combination of contributing factors		Qualitative (L, ML, MH, H)
Jones et al., 1961 (15)	SA	MSA	A bivariate discriminant function was used to differentiate stable from unstable conditions		Quantitative (based on discriminant function values)
Neuland, 1976 (15)	SA	MSA	A discriminant function between stable and unstable slopes obtained by multivariate analysis. Potential failures defined based on distance from discriminant function value to centroid value for stable and unstable cases		Qualitative
NEWTECH (16)	HA	GA	Slope susceptibility defined from activity of identified landslides and combination of contributing factors.		Qualitative (Inactive, Dormant, Active)
Aleotti et al., 2000 (17)	LI	LDeA	Susceptibility index defined for each Land Unit by defining a observed landsliding index (density of landslides within each Unit) and a potential landsliding index (susceptibility due to lithology)		Qualitative (L, ML, MH, H)
Aleotti et al., 2000 (18)	HA	GA	Hazard defined by superimposing a critical altimetric level and a critical runoff belt		Qualitative (DIZ, IZ, FZ)
Ali Jawaid, 2000 (19)	HA	QMC	Weighting factors defined for contributing factors and combined by Fuzzy logic math. Final score obtained by adding each factor score defined different hazard levels		Quantitative/Qualitative
Anbalagan, 1992 (20)	HA	QMC	Hazard zones defined by summation of weighting factors for contributing parameters for division (facets) within study area	1:25,000/1:50,000	Qualitative (L, M, H)
Ayalew, 2000 (21)	HA	GA	Relative slope stability defined by combination of contributing factors		Qualitative (S, SS, U, RF)
Basu, 2000 (22)	LI	LDiA	Slope susceptibility defined from check list of contributing factors obtained from study of past events, and topographic maps		Qualitative (I to V)
Dhakal et al., 2000 (23)	SA	MSA	Hazard defined by combining three hazard maps. One multivariate and two univariate analysis.	1:25,000/1:50,000	Quantitative
Froldi & Bonini, 2000 (24)	HA	GA	Probability of failure within homogeneous terrain units is defined as average value of weighting factors assigned for identified contributing factors from past events		Qualitative (VL, L, M, H, depending on probability of failure)
Janova, 2000 (25)	HA	GA	Slope susceptibility defined by combining contributing factors	1:50,000	Qualitative (U, RS, S, with subdivisions)

Method	Type of analysis	Technique Characteristics	Scale	Hazard assessment value
Thurber & Golber, 1993 D (11)		Hazard intensity distribution obtained by stochastic analysis of past events. Intensity defined by a slide characteristic as volume, and temporal and spatial probability for each site defined from historical data and physical modelling, respectively.	1:15.000	Quantitative (1 to 6, based on mechanism morphometric characteristics and probabilities).
Siddle et al., 1991 (12)	SA	MSA	1:10.000	Quantitative/Qualitative (AL, DL, SLP, NL)
Lee & Moore, 1991 (12)	LI	LDIA	1:2.500	Qualitative
Brawner & Wyllie, 1975 (13)	HA	QMC		Qualitative (A to E, based on potential instability)
Moon et al., 1992 (14)	HA	GA		Quantitative
Fell et al., 1996 (14)	HA	GA		Quantitative
Fell et al., 1996 (14)	HA	GA		Quantitative
Fell et al., 1996 (14)	SA	MSA		Quantitative
Conway et al., 1980 (15)	LI	LDIA	1:50.000	Qualitative
Brunsdan et al., 1975	HA	GA		Qualitative
Kienholz, 1978 (15)	HA	GA	1:10.000	Qualitative (0 to 3, levels defined by description of potential damage)
Ives & Bovis, 1978 (15)	HA	GA		Qualitative (1 to 5)
Parkes & Day, 1975 (15)	SA	MSA		Quantitative
Drennon & Schleining, 1975 (15)	LI	LDeA	1:24.000	Qualitative (six levels defined)

Method	Type of analysis	Technique Characteristics	Scale	Hazard assessment value
Carrara et al., 1978 (1)(5)	SA	MSA Hazard defined as summation of weighted factors and slope categories defined from them	1:10.000	Quantitative/Qualitative (VL, L, H, VH)
Wislocki & Bentley, 1991 (6)	HA	GA Hazard ranks defined by expert opinion of conditional factors	Point	Qualitative
Brand, 1988 (7)	HA	GA Hazard defined by combining contributing factors	1:20.000/1:2.500	Qualitative (I to IV)
Koilara & Watkins (7)	HA	QMC Hazard defined by scores obtained from combining contributing factors		Quantitative
Mateos & Ferrer, 1994 (8)	LI	LDIA Hazard for different slide types is defined by combining critical values of contributing and triggering factors	1:10.000	Qualitative (L, M, H)
Valentini, 1967 (9)	SA	BSA Defines hazard by combining statistically lithology and slope angle		Qualitative (HU, US, S)
Lucini, 1969 (9)	HA	GA Defines hazard as a function of lithology and slope angle		
Vallario, 1973 (9)	LI	LDeA Hazard defined by a stability index obtained from density of landslides per unit area for different lithologies		
Panicucci, 1968 (9)	HA	QMC Hazard defined by numbers obtained by scoring contributing factors		Quantitative (20, S to 100, U)
Papani & Tellini, 1973 (9)	HA	QMC Hazard defined as a combination of favourable or unfavourable condition of contributing factors		Qualitative (I to IV)
Sfrondini, 1970 (9)	HA	GA Hazard defined by contributing factors combination		Qualitative
Pozzo & Sfrondini (9)	HA	GA Hazard defined by combination of lithology and slope angle, then variations made considering favourability of other contributing factors		Qualitative
Amadesi & Altri, 1977 (9)	HA	QMC Hazard defined by a number obtained by scoring lithology, structure and slope angle, then altered to consider effect of land-use, geomorphology and seismicity		Quantitative/Qualitative (VH, H, M, L, S)
Bosi, 1978 (9)	LI	LDIA Hazard defined by a stability index obtained from density of landslides per unit area for different lithologies classified according with slide type, erosion process and cause		Quantitative/Qualitative
Barisone & Bottino, 1990 (10)	HA	QMC Hazard defined by combining rock joint properties and possible slope failures, these factors are scored and multiply to produced a instability number		Quantitative (from 0 to 4)

Method	Type of Technique Characteristics			Scale	Hazard assessment value
	HA	GA	LI		
Radbruch-Hall et al., 1976 (1)	HA	GA	Frequency of slope failures in groups of geologic unit	1:7.500.000	Qualitative (H, M, L)
Wulshusen, 1979 (1)	LI	LDiA	Landslide distribution for different types	1:4.500.000	Qualitative (MC, C, LC, H, L, VL)
Radbruch & Crowther, 1973 (1)	LI	LDeA	Density of landslides per unit area for different lithologies	1:1.000.000	Qualitative (1 to 6)
Rodriguez Ortiz et al., 1978 (1)	LI	LDeA	Density of landslides per unit area for different lithologies	1:100.000	Qualitative
Nilsen et al., 1979 (1)	LI	LDiA	Landslide distribution for different slope angle units, susceptible bedrock and surface geologic unit	1:125.000	Qualitative (S, GS, MS, MU, U, N)
Blanc & Cleveland, 1968 (1)	HA	GA	Combines lithologic units by strength and critical slope angle for each one	1:19.200	Qualitative
Brabb et al., 1972 (1)	LI	LDeA	Hazard value based on percentage of area of different geologic units cover by landslides, then subdivided by slope angle		Qualitative (I to VI and L)
ZERMOS, 1972- (1)(2)	HA	GA	Hazard value based on presence or absence of unfavourable values or conditions of contributing factors	1:25.000/1:10.000	Qualitative (Three or four colours from H to subdivision of these also reported)
Carrara & Merenda, 1974 (1)(3)	LI	LDiA	No hazard value given but slope classes based on failed slopes inventory	1:25.000	
Malgot et al., 1973 (1)	LI	LDiA	Hazard defined by contributing factors combination for different landslide types	1:10.000	Qualitative (U, RS, S, with subdivisions)
Stevenson, 1977 (1)(4)	HA	QMC	Risk defined from a number obtained by product of scores given for each factor condition		Qualitative/Quantitative (S, U, based on scores)
Meneroud, 1978 (1)	HA	QMC	Risk defined by a number obtained by adding contributing factor scores		Quantitative
Veccia, 1978 (1)	HA	QMC	Terrain groups defined by a terrain index obtained by scoring contributing factors		Quantitative
Klugman & Chung, 1976 (1)	D		Slope categories defined by distribution of factor of safety	1:25.000/1:50.000	Quantitative
Huma & Radelescu, 1978 (1)	HA	QMC	Hazard defined by contributing factors combination for different landslide types	1:25.000	Qualitative (seven degrees of hazard define

ANNEX D

DATA COLLECTION SPREADSHEETS

In order to compile and process data about earthquakes inducing landslides and earthquake-induced landslides three spreadsheets were designed. An application example of these sheets are shown in this Annex. This example corresponds to data collected from several sources for the 1980 Irpinia earthquake (Italy).

First sheet compiled information about the general setting of the area where the earthquake occurred. The second sheet is the compilation of general information about landslides triggered by the earthquake, whereas the third sheet compiled information about a specific landslide. From these kind of sheet, data compiled in databases presented in the C.D annexed was obtained.

4. STRONG MOTION RECORDING

STATION	COORDINATES		DEFORET DESCRIPTION	d ₁ km	d ₂ km	INSTRUMENT ORIENTATION		PEAK GROUND ACCELERATION (g)			PEAK GROUND VELOCITY (m/s)			AREAS INTENSITY (mm)			DURATION (s)
	N/S	E/W				L	T	L	T	V	L	T	V	L	V	T	
ARSENIO	41.029	14.461		71.6	65.0	N-S	E-W	0.029	0.036	0.021	0.039	0.029	0.036	0.018	0.022	0.009	0
AULETTA	40.560	15.578		31.1	11.8	N-S	E-W	0.061	0.060	0.036	0.046	0.055	0.042	0.068	0.039	0.021	0
BALICCIANO	40.671	15.069		25.1	20.3	N-S	E-W	0.136	0.109	0.178	0.207	0.322	0.144	0.376	0.478	0.183	5.9
BALICCIANO	41.121	14.796		31.7	41.0	N-S	E-W	0.035	0.055	0.020	—	—	—	0.062	0.068	0.029	0
BALICCIANO	41.043	15.543		31.5	23.2	N-S	E-W	0.096	0.061	0.084	0.189	0.143	0.101	0.286	0.233	0.153	18.8
BOVINO	41.251	15.510		34.5	27.5	N-S	E-W	0.047	0.067	0.029	0.045	0.036	0.028	0.061	0.065	0.022	0
BOVINO	40.474	15.635		42.6	23.0	N-S	E-W	0.217	0.171	0.150	0.112	0.086	0.068	0.518	0.469	0.216	8.3
CALITRI	40.917	15.439		17.8	8.5	N-S	E-W	0.159	0.170	0.168	0.269	0.284	0.193	1.060	1.351	0.814	44.9
GARIGLIANO	41.259	15.027		15.1	10.3	N-S	E-W	0.033	0.033	0.022	0.082	0.063	0.037	0.033	0.025	0.010	0
LAURIA	40.048	15.836		41.9	82.0	N-S	E-W	0.015	0.016	0.009	—	—	—	0.004	0.005	0.001	0
MARINO	40.791	14.764		47.6	35.0	N-S	E-W	0.107	0.136	0.055	0.085	0.131	0.052	0.333	0.539	0.098	16.7
MONTEBELLUNA	40.929	15.669		32.9	24.1	N-S	E-W	0.109	0.098	0.073	0.141	0.002	0.072	0.652	0.498	0.262	41.6
S. SEVERO	41.684	15.346		100.5	88.0	N-S	E-W	0.024	0.021	0.013	0.023	0.021	0.016	—	—	—	—
STURNO	41.023	15.117		32.4	20.2	N-S	E-W	0.210	0.312	0.229	0.362	0.618	0.241	1.318	1.519	0.571	19.6
TORRE DEL GRECO	40.001	14.306		71.4	66.7	N-S	E-W	0.062	0.040	0.036	0.054	0.057	0.055	0.400	0.045	0.030	0
TRICARICO	40.621	16.157		71.9	41.5	N-S	E-W	0.045	0.035	0.024	0.063	0.084	0.041	0.051	0.052	0.020	0
VIGESTE	41.879	16.164		140.5	135.0	N-S	E-W	0.035	0.031	0.013	0.024	0.021	0.025	0.029	0.023	0.005	0

5. REFERENCES

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 (21). Colacchia et al., 1985
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 (23). Colacchia et al., 1985
 (24). Agnelli et al., 1985
 (25). Gargano et al., 1985
 (26). Del Prete et al., 1985
 (27). Gargano et al., 1985
 (28). Gargano et al., 1985
 (29). Gargano et al., 1985

EARTHQUAKE INDUCED LANDSLIDES

1. IDENTIFICATION							
DATE: 23-XI-1986	EARTHQUAKE NAME: <u>IRPINIA</u> STATE/COUNTRY: <u>CAMPANIA-ITALY</u>						
CODE: <u>77-182-181</u>							
2. GENERAL DATA							
NUMBER OF LANDSLIDES: <u>20,000</u> AREA AFFECTED BY LANDSLIDES: <u>1212.4</u> (km ²) COMMON TYPES OF LANDSLIDES 1. <u>Translational slides</u> 2. <u>Rotational slides</u> 3. <u>Complex slides</u> 4. <u>Rock falls</u> Maximum d ₁ (km) d ₂ (km) >70% 50-70% 30-50% 10-30% 1-10% <1%	INDUCED LANDSLIDES Landslide type Disrupted slides-soil Disrupted slides-rock Coherent slides-soil Coherent slides-rock Lateral spreads and flows Deep slides Surface slides Minimum Intensity Maximum d ₁ km Maximum d ₂ km REFERENCES (1), (2), (3), (4), (5), (6), (7), (8), (11), (12), (13), (14), (16), (17), (18), (19).						
3. LANDSLIDES							
CODE	LANDSLIDE	TYPES/MATERIAL	COORDINATES N/E +/-	FAULT PLANE PROJECT DISTANCE km	EPICENTRAL DISTANCE km	CLOSEST ACCELEROGRAPH DISTANCE km	INTENSITY MM
	VALVA	rock fall / Limestone	40.760 15.350	0.0	2.8		IX
	BALIA MOUNTAIN	rock fall / limestone					IX
	COLLEAIO	rock fall / limestone	40.720 15.290	2.5	7.5		IX
	BALVANO	rock fall / limestone of dolomite	40.680 15.540	4.2	26.3		VII
	S. GIUSEPPE LA SPIGA 3	slide / sandstone of clay	41.270 14.930	58.3	63.9		VI
	SAURO stream	slide / clay & sandstone of clay	40.360 16.010	59.2	73.9		VI
	ANDRETTA	slide / clay & conglomerate	40.740 15.070	5.0	18.1		VIII
	TRABIA DEL LONGARRO	slide / sandstone, sandy clay & dolomite	40.740 15.070	11.7	25.5		VIII
	CALITRI	slide / clay & sandstone of clay	40.870 15.440	8.3	15.3		VIII
	S. ANGELO DEL LONGARRO	slide / clay & conglomerate	40.930 15.180	7.5	20.7		X
	CARUSCLE	slide / sandstone (rock fall)	40.810 15.220	0.0	7.8		IX
	S. PELE	slide / clay & sand	40.820 15.540	5.8	16.2		VII
	SEVERCHIA	Flow / debris of sandstone of clay	40.760 15.210	4.2	10.3		VIII-IX
	CALITRI	Flow / clay & sandstone of clay	40.870 15.440	8.3	15.3		VIII
	PICERARO	Andesite	40.681 15.135	10.8	35.8		VII
	GIARROSA	Andesite	40.630 15.137	13.3	37.0		VII
	GUACCIA	Rotational slide		22.5	31.2		VII

LANDSLIDE REPORT

1. IDENTIFICATION AND LOCATION		2. GENERAL CHARACTERISTICS		3. GEOLOGICAL STRUCTURES		4. ROCK CONDITION		5. GROUND WATER CONDITION			
NAME: <u>ADULETTA</u> CODE: <u>111-MI-M88-7</u> DRAINAGE BASIN: <u>SAGBA GIVER</u> EARTHQUAKE: <u>1891A</u> DATE: <u>23-11-1980</u> ELEVATION CROWN: <u>790</u> msl RUPTURE SURFACE TOE: _____ msl FOOT: <u>350</u> msl		LITHOLOGY DESCRIPTION <u>Conglomerates: poorly cemented and almost totally bedded. Presence of large blocks that may reach 1m³ of size surrounded by rounded pebbles and clasts.</u> <u>Sandy and silty clay layers: clay slightly cemented and somewhat bedded.</u> <u>few conglomerate beds</u> <u>clays of conglomerates</u>		PRIMARY IGNEOUS Dykes Sills Ring Structures Lacoliths Plutons Stocks Batholiths Sheets SEDIMENTARY Normal bedding Cross bedding METAMORPHIC Foliation		SECONDARY FOLDS Anticline Syncline Monocline Valley bulges Salt domes FAULTS Normal Strike slip Thrust Oblique JOINTS Strike-slip joints Folded sediments		ROCK CONDITION Fresh Fractured Weathered Grade VI Grade V Grade IV Grade III Grade II Grade I		PIREATIC LEVEL Below failure surface Above failure surface and below middle slide height Above middle slide height Superficial SLOPE DRAINAGE Into landslide Out landslide Not estimable Spring points Ponds Parallel to slope	
STATE/COUNTRY: <u>AVELAND</u> COORDINATES: LATITUDE: <u>40.940</u> LONGITUDE: <u>8.379</u> LOCAL SEISMIC PARAMETERS PGA (g) _____ PGV (m/s) _____ Ia (m/s) _____ D_r (s) _____ Lateral () _____ Transversal () _____ Vertical () _____ EPICENTRAL DISTANCE: <u>10.1</u> km DISTANCE TO FAULT PLANE PROJECTION: <u>5.0</u> km		DIP NORMAL TO SLIDE FACE Horizontal Vertical Oblique PARALLEL TO SLIDE FACE Horizontal Vertical Oblique Dipping out Dipping in		MATERIAL PROPERTIES (clays) ³sc / pc. $\gamma_r = 20.0 / 21.8 \text{ kN/m}^3$ $w_L = 11.4 / 14.6 \%$ $w_U = 88.0 / 42.0 \%$ $I_p = 17.0 / 20.0 \%$ $CF = 30.0 / 45.0 \%$ $Sr = 60 / 90 \%$		MOVEMENT AND FAULT DIRECTION 					
BEDROCK <u>Variegated clays</u>		DISPLACED MASS: LENGTH (Ld) = _____ m WIDTH (Wd) = _____ m DEPTH (Dd) = _____ m HEIGHT (Hd) = _____ m VOLUME (Vd) = <u>30.00</u> m ³		MORPHOMETRIC CHARACTERISTICS RUPTURE SURFACE: LENGTH (Lr) = <u>1000</u> m WIDTH (Wr) = <u>400</u> m DEPTH (Dr) = <u>150</u> m HEIGHT (Hr) = <u>200</u> m AREA (Ar) = <u>0.08</u> m ² Pre-failure slope: <u>7.5</u> Post-failure slope: <u>1.0</u> Deposit slope: Shape factor (Dr/Lr): <u>0.004</u> Distance from the hillside crest to slide crown: <u>2.5</u> m							

C. TOPOGRAPHIC SETTING AND LAND USE			
PROFILE SHAPE		TOPOGRAPHIC PARAMETERS	
Simple Rectilinear (R) Concave (V) Convex (X) Compound Type: _____ Complex Type: $V \pm V/X \pm R$ Not known: _____	Inflection Number: <u>1</u> Inflection Index: 1. <u>0.517</u> m/m PLAN SHAPE Uniform Convergent Divergent Not known: _____	Maximum altitudinal difference: <u>200</u> m Relative Relief: <u>950</u> m/km ² Knick point angle: _____ degrees Other: _____ x <u>DN/area</u> for bedrock plan	LAND USE RURAL AREA Bare Crops (type): _____ Pasture Shrubs Forest Other: _____ URBAN AREA Artificial fill Road cut Other: _____
E. NOTES - National data reported landslide since the 1984 earthquake - Movement started 12 hours after earthquake - Started at the foot and propagated uphill. - Three days after earthquake area was fully developed.			
D. HYDROLOGICAL SETTING			
PARAMETERS			
Temperature: _____ °C Air moisture: _____ % Solar bright: _____ h ET: _____ mm Annual Rainfall: <u>640</u> mm Climatic Zone: _____	Rainfall Type Orographic Convective Cyclonic Typhoon Monsoon Climatic Zone: _____	RAINFALL PARAMETERS The October-November 1980 rainfall was much higher than the average and in particular in November 1980. 70% of total monthly rainfall was recorded in the first 15 days. Four days after the earthquake there was no rain. Accumulated during January, November, present, drought.	
3. FAILURE MECHANISM			
MATERIAL		MECHANISM SEQUENCE	
ROCK SOIL <u>plio. Pleistocene clays</u> <u>& conglomerates</u>	☒ Sliding (multi-light related) ☐ Fall ☐ Flow ☐ Creep ☐ Topple ☐ Lateral Spread	FAILURE SURFACE Weakness zone Soil/Rock interface Discontinuity plane Pre-existing slip surfaces	ACTIVITY CONDITION Distribution of Activity Retrogressing Advancing Widening Confined Enlarging Diminishing Moving
MOVEMENT VELOCITY		STABILIZING WORKS	
Extremely low Very low Low Moderate Rapid Very rapid Extremely rapid	< 16 mm/y 16 mm/y - 1.6 m/y 1.6 m/y - 13 m/m 13 m/m - 1.8 m/h 1.8 m/h - 3.0 m/min 3.0 m/min - 5.0 m/sec > 5.0 m/sec	State of Activity Active Reactivated Suspended Inactive: Dormant Abandoned Stabilized Relict Stable	Style of Activity Complex Composite Multiple Successive Single
4. DAMAGES			
PROPERTY, LIFE LINES DAMAGE AND SOCIAL EFFECTS			
Town, village, etc. Houses No _____ Country Houses No _____ Main Roads _____ Country Roads _____ Rail Roads _____ Others: _____		Aqueduct Pipe line Bridge - Viaduct Retaining walls or similar structures Dammed river, creek, etc. <u>Southern River (3 weeks)</u> Deaths: _____ Injuries: _____	
REFERENCES:			
(1), (2), (M)			

