PRELIMINARY QUANTITATIVE RISK ASSESSMENT OF EARTHQUAKE-INDUCED LANDSLIDES AT MAN-MADE SLOPES IN HONG KONG

GEO REPORT No. 98

H.N. Wong & K.K.S. Ho

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING DEPARTMENT

THE GOVERNMENT OF THE HONG KONG SPECIAL ADMINISTRATIVE REGION

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PREFACE

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R.K.S. Chan
Head, Geotechnical Engineering Office
February 2000

FOREWORD

In the current geotechnical practice in Hong Kong, provision is not made for earthquake loading in routine slope design. In order to examine the likely significance of failures of man-made slopes (viz. soil cut slopes and fill slopes) caused by earthquake loading, a preliminary assessment of the risk of earthquake-induced landslides in Hong Kong was carried out. Quantitative risk assessment (QRA) methodology was adopted in the preliminary analysis. The assessment systematically considered the seismicity in Hong Kong, effects of earthquake on slope stability and the likely consequence-to-life in the event of slope failures. The findings of the study are documented in this Report.

This pilot study, which was undertaken by Mr K.K.S. Ho and myself, forms part of the R&D Theme on QRA of Landslide Hazards in Hong Kong. It is intended to explore the feasibility of using QRA methodology to assist in the evaluation of the risk of earthquake-induced landslides.

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ABSTRACT

In this pilot study, standard quantitative risk assessment (QRA) techniques involving the use of fault trees and event trees have been used to evaluate the risk of failures of engineered man-made slopes due to earthquake loading. The risk of failure of pre-1977 slopes which had not been checked or upgraded to current standards is outside the scope of this Report.

This preliminary study consists of a global risk assessment and integrates the past work carried out on seismicity of Hong Kong, including the assessment of critical acceleration, dynamic response characteristics of soils, earthquake-induced displacements of slopes, likelihood of different degrees of soil saturation at the time of an earthquake, margin of safety due to partial saturation, etc. In the QRA, due account is taken of the different landslide hazards as well as the probability and consequence of the different types of slope failures.

The results of the preliminary QRA have been compared with the risk of rain-induced landslides of pre-1977 man-made slopes. For soil cut slopes and fill slopes that comply with current geotechnical standards, the results suggest that the risk of earthquake-induced landslide fatalities in respect of such slopes is only a relatively small proportion of that posed by rain-induced landslides at pre-1977 man-made slopes.

For slopes affecting important facilities, the quantification of the risk of earthquake-induced landslides may be made using a QRA framework similar to that presented in this Report. This may include a more rigorous allowance for the uncertainties in the key input parameters by means of tools such as reliability-based methods or Monte-Carlo simulations.

The successful application of the QRA technique in the preliminary assessment to evaluate seismic risk illustrates the feasibility and usefulness of this tool in such applications which will provide more insight to the problem than the conventional seismic hazard assessment methodology.

CONTENTS

1. INTRODUCTION

The vast majority of landslides at cut and fill slopes in Hong Kong are triggered by heavy rainfall (Brand, 1984). Slope failures which are directly attributable to recorded seismic activity have not been reported in Hong Kong. One possible reason for this is that Hong Kong has not experienced any strong earthquakes since urban development commenced within the last century or so.

In the current geotechnical practice in Hong Kong, no explicit provision is made for earthquake loading in routine slope design. In order to examine the likely significance of failures of man-made slopes (e.g. soil cut slopes and fill slopes) due to earthquake loading, a preliminary assessment of the risk of earthquake-induced landslides in Hong Kong was carried out. Standard quantitative risk assessment (QRA) methodology was adopted in the preliminary analysis. The assessment systematically considered the seismicity in Hong Kong, effects of earthquake on slope stability and the likely consequences with respect to loss of life in the event of slope failures. The findings of the study are documented in this Report.

This pilot study forms part of the R&D Theme on QRA of Landslide Hazards in Hong Kong. It is intended to explore the feasibility of using the QRA methodology to assist in the evaluation of the risk of earthquake-induced landslides at engineered slopes. The risk of earthquake-induced failure of pre-1977 slopes which had not been checked or upgraded to current standards is outside the scope of this Report.

A glossary of the standard QRA terminology adopted in this Report is given in Appendix A.

2. INPUT DATA FOR QUANTITATIVE RISK ASSESSMENT

2.1 General

In order to assess the risk of earthquake-induced slope failures in Hong Kong, the relevant technical data were examined and appropriate correlations were developed. Such data and correlations have been suitably generalized to facilitate a practicable QRA. Despite the necessary simplifying assumptions that have to be made, the assessment is nevertheless considered to be sufficiently representative for the range of man-made slopes that prevail in Hong Kong. For a global QRA such as that considered in this Report, it is not possible, and in fact not necessary, to derive very accurate such as that data and precise correlations for the individual slopes.

The relevant technical data and correlations derived and used in the risk assessment are described in the following Sections.

2.2 Seismicity of Hong Kong

The seismicity of Hong Kong has previously been studied by various authors, e.g. Lau (1972), Lam (1980), and Lam & Fong (1982). A comprehensive review of the available macroseismic and instrumental data on earthquakes around Hong Kong is documented in

GCO (1991).

Based on the data given in GCO (1991), a standard seismic hazard analysis can be performed to determine the return periods (T) of different bedrock peak ground accelerations (PGA). Results of analyses carried out by Pun & Ambraseys (1992), as shown in Figure 1, are adopted for the present QRA.

Although the seismic hazard analysis may be further refined with due consideration taken of alternative distribution of seismic sources and attenuation characteristics (e.g. Scott et al, 1994; Lee et al, 1998), the T-PGA curve given in Figure 1 is considered sufficiently representative for the present preliminary study.

2.3 Critical Acceleration

Pseudo-static analysis is the most commonly-used technique for routine seismic slope stability assessments. In this approach, the effects of ground vibration are modelled as an equivalent pseudo-static force (F) acting on the slope, which is defined as follows:

$$F = W.K \qquad (1)$$

where W = weight of the soil mass above a potential slip surface
K = net acceleration (unit in g) of the soil mass which is related to the PGA (see Section 2.3)

The critical acceleration (K_c) of a slope is the net acceleration under which the soil mass would be brought to a state of limit equilibrium, i.e. the Factor of Safety of the slope becomes unity. The value of K_c depends on the following factors:

(a) the static Factor of Safety (F_s) of the slope, i.e. the Factor of Safety before the application of the force F, and

(b) the characteristics of the slope-forming materials.

The relationship between K_c and F_s for typical cut and fill slopes in Hong Kong as shown in Figure 2 is adopted in this study. The derivation of the curves shown in Figure 2 is explained in Appendix B.

2.4 Dynamic Response Characteristics

The net acceleration (K) of a sloping soil mass can be expressed in terms of PGA by means of the following equation:

$$K = K_a.PGA \qquad (2)$$

where K_a = average seismic coefficient spectrum which reflects the dynamic response characteristics of the soil mass

Dynamic response analyses have been carried out by Ambraseys & Sarma (1967) using real earthquake records based on the one-dimensional shear beam solution (Figure 3). For practical purposes, the appropriate K_a values to be adopted in pseudo-static analyses may be determined using the curves shown in Figure 3.

For typical man-made slopes in Hong Kong ranging from 5 m to 50 m in height and comprising medium dense to dense soils with a typical shear wave velocity of 200 m/s, the corresponding fundamental periods of the slopes are in the range of 0.1 second to 0.7 second.

It can be seen from Figure 3 that the representative value of K_a may be taken as 1.0 for overall slope failures (i.e. corresponding to an α value, as defined in Figure 3, of 0.8 to 1.0) and 1.6 for local slope failures (i.e. an α value of 0.2 to 0.6). This implies that in assessing overall slope failures, no net magnification effect of the soil mass needs to be considered. However, in assessing local slope failures, some 60% magnification should be considered. Possible amplification of the ground motions due to site response effects are not considered in the present assessment, since the vast majority of the man-made slopes in developed areas in Hong Kong are not situated in areas underlain by soft soils.

2.5 Seismic-Induced Slope Displacements

When the acceleration imposed on the soil mass exceeds its critical acceleration, the net disturbing force on the soil mass will be larger than the net resisting force and slope displacements will result. Standard closed-form solutions have been derived (e.g. Newmark, 1965; Ambraseys, 1972 and Sarma, 1975) for assessing dynamic slope displacements using a sliding block model that assumes no variation in material strengths during and after slope displacements. Ambraseys & Menu (1988) used the sliding block assumptions to assess statistically the displacements of slopes with different ratios of K_c/K based on 48 near-field earthquake records with a magnitude of between 6.6 and 7.3. The correlations recommended by Ambraseys & Menu (op cit) are shown in Figure 4.

As the nature of strong motion in Hong Kong is not certain, reference is made to the correlations shown in Figure 4 for estimating the order of slope displacements for the purposes of hazard identification.

It should be noted that Ambraseys & Menu's solutions are useful for the cases where the difference between the peak soil strength and the post-peak strength is negligible. If the post-peak drop in the soil shear strength is not negligible, the post-peak behaviour of the soil should be taken into account.

2.6 Likelihood of Different Degrees of Soil Saturation at the Time of Earthquake

Except in the case of heavy rain, the majority of the slopes in Hong Kong will remain in a state of low degree of soil saturation and hence possess an additional margin of stability because of unsaturated shear strength. An approximate assessment has been made using the available rainfall data to examine the likelihood of slopes in Hong Kong that may be in a state of low, moderate or high degree of soil saturation in the event of an earthquake (Appendix C). The likelihood of low, moderate and high degree of soil saturation is taken to be 95%, 4.5%

and 0.5% respectively in the present assessment (Appendix C).

2.7 Additional Margin of Stability for Unsaturated Slopes

An assessment has been made of the additional margin of stability exhibited by typical slopes in Hong Kong when they are in an unsaturated state (Appendix D). A conservative value of soil suction and typical unsaturated soil parameters (Gan & Fredland, 1992) have been adopted in the limit equilibrium analyses. The results suggest that the typical additional margin of stability may be taken to be an increase in F_s of 0.3 and 0.15 for a low and moderate degree of saturation respectively.

3. HAZARD IDENTIFICATION

Differing earthquake motions will affect slopes to a different degree with the corresponding consequence of different types of slope failure. The range of earthquake-induced landslide hazards considered in the present QRA are categorised into four failure modes. As illustrated in Figure 5, the four modes of instabilities are:

(a) overall slope failure (denoted as OF),

(b) overall slope deformation with local slope failure (denoted as OD),

(c) local slope failure (denoted as LF), and

(d) local slope deformation (denoted as LD).

The failure trigger criteria can be expressed in terms of the ratio of PGA to K_c for each of the failure modes. These failure triggers have been derived by reference to the dynamic response characteristics and the likely range of earthquake-induced slope displacements (Appendix E). The respective criteria are summarised in Table 1. Two types of slopes are distinguished, viz:

(a) Slopes with ductile stress-strain characteristics - in this case, the slope-forming materials do not exhibit a significant reduction in shear strength with strain. Typical examples are dense (e.g. recompacted) fill slopes and unsaturated loose fill slopes.

(b) Slopes with brittle (or strain-softening) stress-strain characteristics - in this case, the slope-forming materials attain a peak shear strength at a relatively small strain level and the post-peak shear strength reduces with further shear strain. A typical example is a cut slope in saprolites. Saturated loose fill may be considered a special case in that large displacements or even uncontrolled failure (i.e. liquefaction) may result from earthquake loading if the slope-forming material is very brittle. However, saturated loose fill slopes do not comply with current geotechnical standards and are not considered in this Report.

4. FREQUENCY ASSESSMENT

The frequencies of occurrence of the four failure modes have been assessed based on the following considerations:

(a) Typical soil cut slopes in saprolite and compacted fill slopes with design F_s values of 1.1, 1.2 and 1.4 respectively.

(b) The failure trigger criteria shown in Table 1 are adopted.

(c) Soil cut slopes are considered to be brittle and dense fill slopes are considered ductile.

(d) The annual frequencies of occurrence of the respective PGA values (which have been calculated from the T-PGA curve given in Figure 1) are summarised in Table 2.

(e) A simplified fault tree for the occurrence of the different modes of failure is shown in Figure 6. Two independent events are considered, viz. the occurrence of a certain PGA and the state of saturation of the slope at the time of an earthquake.

(f) The likelihood of slopes being in a state of high, moderate or low degree of saturation is considered (Appendix C).

(g) The PGA values considered in the simplified fault tree for frequency assessments are based on the failure trigger criteria given in Table 1, with due account taken of the prevailing margin of stability of the slopes. For slopes under a high degree of saturation, the margin of stability is taken to be the design F_s for the slope minus unity; for slopes under a low degree of saturation, an additional margin of stability corresponding to an extra F_s value of 0.3 is allowed for, whereas a margin of 0.15 is assumed for a moderate degree of saturation (Appendix D). The frequencies of occurrence of different failure modes under differing degrees of soil saturation are presented in the form of a fault tree (see Table 3).

(h) The calculated frequencies of occurrence of the different modes of seismic instability are shown in Table 4.

5. CONSEQUENCE ASSESSMENT

An assessment of the likely consequence of the four failure modes has been made (Appendix F). The likely landslide fatality figures have been derived based on the historical data on landslide casualties, with allowance made for the nature and proximity of the affected facilities and the nature of the different failure modes (Wong & Ho, 1997).

The assessed consequences given the occurrence of the four modes of seismic instability as expressed in terms of fatality per year per slope are shown in Table 5.

6. RISK CALCULATION

The risk of earthquake-induced landslides in Hong Kong may be calculated from the frequency and consequence data (Tables 3 and 4). The risk, expressed in terms of average number of fatality per slope per year, has been assessed using the event tree formulation for engineered cut slopes and fill slopes. The results of the assessment are summarised in Table 5.

7. DISCUSSION

To put the results of the preliminary QRA in context, the findings have been compared with the risk of rain-induced landslides of pre-1977 man-made slopes (Table 8). For soil cut slopes and fill slopes that comply with current geotechnical standards, it can be seen from Table 8 that generally the risk of earthquake-induced landslide fatalities is relatively small compared with the risk arising from rain-induced landslides at pre-1977 man-made slopes which may not comply with current geotechnical standards. Also, it should be noted that the assessment is on the conservative side in that the minimum theoretical Factors of Safety have been assumed for the upgraded slopes. Pappin & Bowden (1997) also reported that for situations where the static factor of safety is adequate, the probability of seismic-induced slope failure is low enough to accord with international good practice.

Overall, it may be concluded that man-made slopes that have been engineered to current geotechnical standards do not pose a significant risk to life under earthquake loading.

As the risk of earthquake-induced landslides at such slopes is much lower than the risk of rain-induced landslides at pre-1978 substandard man-made slopes, this supports the current approach that efforts are directed to retrofitting those potentially substandard pre-1978 man-made slopes.

For old loose fill slopes that have been upgraded to current geotechnical standards by surface recompaction, psuedo-static stability analyses carried out by Wong (1991) suggest that overall seismic instability may occur through the deeper loose fill layer, depending on the magnitude of excess pore water pressures induced during earthquake excitation. However, given the presence of a less permeable layer of recompacted fill, the chances of the underlying loose fill body becoming nearly saturated at the time of an earthquake are likely to be very small.

For slopes affecting important facilities, the quantification of the risk of earthquake-induced landslides may be made using a QRA framework similar to that presented in this Report. This may include a more rigorous treatment of the uncertainties of the key input parameters by means of tools such as reliability methods or Monte-Carlo type simulations. In the preliminary assessment, best-estimate values have been adopted for the input parameters. Since the assessed risk of earthquake-induced landslides at man-made slopes that comply with current geotechnical standards is much lower than rain-induced landslide risk posed by pre-1978 man-made slopes, rigorous allowance for uncertainties has not been considered in the present global risk assessment.

8. CONCLUSIONS

A formal QRA methodology involving the use of fault trees and event trees has been adopted in a preliminary assessment of the risk of earthquake-induced landslides at man-made slopes in Hong Kong. It is concluded that the risk of earthquake-induced landslides at slopes designed or upgraded to current geotechnical standards is much smaller than the risk of rain-induced landslides for pre-1978 man-made slopes that have not been upgraded to current standards. The current geotechnical standards appear to be adequate in maintaining the overall risk of earthquake-induced failures on new slopes at a relatively low level, and efforts should continue to be directed to upgrading substandard man-made slopes.

The successful application of QRA techniques in the present preliminary assessment to evaluate earthquake risk demonstrates the usefulness of the systematic approach which involves hazard identification, frequency and consequence assessments and risk summation.

The following areas of further R&D work are useful for further improving the risk quantification to facilitate the formulation of optimal landslide risk management strategies:

(a) For the quantification of the risk of failure of pre-1977 man-made slopes due to earthquake, QRA techniques can be used with broad assumptions on the distribution of the probable actual Factors of Safety for the slope population. The landslide consequence model developed by Wong et al (1997) can also be further refined for application to the seismic risk assessment of pre-1977 slopes.

(b) Examination of the relationship between theoretical Factor of Safety and actual probability of failure to improve the accuracy of the QRA assessments.

(c) Quantification of the risk of rain-induced failure of post-1977 man-made slopes to further assess the relative risk between rain-induced and earthquake-induced landslides on such slopes.

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LIST OF TABLES

Table 1 - Trigger Criterial for Different Modes of Instability

Failure Mode	$\frac{\text{Peak Ground Acceleration}}{\text{Critical Acceleration}} = \frac{PGA}{K_c}$	
	'Brittle' Slopes	'Ductile' Slopes
Overall Slope Failure (OF)	>1.4	Not applicable
Overall Slope Deformation with Local Slope Failure (OD)	1.0 to 1.4	>2
Local Slope Failure (LF)	0.88 to 1.0	1.25 to 2
Local Slope Deformation (LD)	0.63 to 0.88	0.88 to 1.25
Note: The definitions for 'brittle' and 'ductile' slopes are given in Section 3 of the report.		

Table 2 - Frequency of Occurrence of PGA Levels Triggering Different Failure Modes (Sheet 1 of 4)

(a) Soil Cut Slopes (Brittle)				
Margin of Static Factor of Safety (K_c value shown in bracket)	Failure Mode	Range of PGA for Different Failure Modes[2] (g)	Return Period[3], T (years)	Annual Frequency of Occurrence[4], f
5% (K_c = 0.03 g)[1]	OF	> 0.042	> 40	2.5×10^{-2}
	OD	0.03 - 0.042	13 - 40	5.192×10^{-2}
	LF	0.026 - 0.03	8 - 13	4.808×10^{-2}
	LD	0.019 - 0.026	308	2.083×10^{-1}
10% (K_c = 0.06 g)	OF	> 0.084	> 500	2.000×10^{-3}
	OD	0.06 - 0.084	150 - 500	4.667×10^{-3}
	LF	0.053 - 0.06	100 - 150	3.333×10^{-3}
	LD	0.038 - 0.053	30 - 100	2.333×10^{-2}
20% (K_c = 0.12 g)	OF	> 0.168	> 5 500	1.818×10^{-4}
	OD	0.12 - 0.168	1 600 - 5 500	4.432×10^{-4}
	LF	0.106 - 0.12	1 150 - 1 600	2.446×10^{-4}
	LD	0.076 - 0.106	350 - 1 150	1.988×10^{-3}
25% (K_c = 0.14 g)	OF	> 0.196	> 9 000	1.111×10^{-4}
	OD	0.14 - 0.196	3 200 - 9 000	2.014×10^{-4}
	LF	0.123 - 0.14	2 000 - 3 200	1.875×10^{-4}
	LD	0.088 - 0.123	500 - 2 000	1.5×10^{-4}
30% (K_c = 0.17 g)	OF	> 0.238	> 25 500	4×10^{-5}
	OD	0.17 - 0.238	6 000 - 25 500	1.267×10^{-4}
	LF	0.15 - 0.17	3 850 - 6 000	9.307×10^{-5}
	LD	0.107 - 0.15	1 200 - 3 850	5.736×10^{-4}
35% (K_c = 0.195 g)	OF	> 0.273	> 57 000	1.754×10^{-5}
	OD	0.195 - 0.273	10 000 - 57 000	8.246×10^{-5}
	LF	0.172 - 0.195	6 100 - 10 000	6.393×10^{-5}
	LD	0.123 - 0.172	1 900 - 6 100	3.624×10^{-4}
40% (K_c = 0.22 g)	OF	> 0.308 g	> 120 000	8.333×10^{-6}
	OD	0.22 - 0.308	18 000 - 120 000	4.722×10^{-5}
	LF	0.194 - 0.22	10 000 - 18 000	4.444×10^{-5}
	LD	0.139 - 0.194	3 000 - 10 000	2.333×10^{-4}
50% (K_c = 0.27 g)	OF	> 0.378	> 500 000	2×10^{-6}
	OD	0.27 - 0.378	55 000 - 500 000	1.618×10^{-5}
	LF	0.238 - 0.27	25 000 - 55 000	2.182×10^{-5}
	LD	0.170 - 0.238	6 000 - 25 000	1.267×10^{-4}

Table 2 - Frequency of Occurrence of PGA Levels Triggering Different Failure Modes (Sheet 2 of 4)

(a) Soil Cut Slopes (Brittle)				
Margin of Static Factor of Safety (K_c value shown in bracket)	Failure Mode	Range of PGA for Different Failure Modes[2] (g)	Return Period[3], T (years)	Annual Frequency of Occurence[4], f
55% (K_c = 0.3 g)	OF	> 0.42	> 1 000 000	1×10^{-6}
	OD	0.3 - 0.42	100 000 - 1 000 000	9×10^{-6}
	LF	0.264 - 0.3	45 000 - 100 000	1.222×10^{-5}
	LD	0.189 - 0.264	9 000 - 45 000	8.889×10^{-5}
60% (K_c = 0.32 g)	OF	> 0.448	> 3 000 000	3.333×10^{-7}
	OD	0.32 - 0.448	170 000 - 3 000 000	5.549×10^{-6}
	LF	0.281 - 0.32	70 000 - 170 000	8.403×10^{-6}
	LD	0.201 - 0.281	11 000 - 70 000	7.662×10^{-5}
70% (K_c = 0.37 g)	OF	> 0.518	> 10 000 000	1×10^{-7}
	OD	0.37 - 0.518	5 000 000 - 10 000 000	1×10^{-7}
	LF	0.326 - 0.37	190 000 - 5 000 000	5.063×10^{-6}
	LD	0.233 - 0.326	23 000 - 190 000	3.822×10^{-5}

Table 2 - Frequency of Occurrence of PGA Levels Triggering Different Failure Modes (Sheet 3 of 4)

(b) Dense Fill Slopes (Ductile)				
Margin of Static Factor of Safety (K_c value shown in bracket)	Failure Mode	Range of PGA for Different Failure Modes[2] (g)	Return Period[3], T (years)	Annual Frequency of Occurrence[4], f
5% (K_c = 0.025 g)	OF	-	-	-
	OD	> 0.05	> 80	1.25×10^{-2}
	LF	0.031 - 0.05	13 - 80	6.442×10^{-2}
	LD	0.022 - 0.031	7 - 13	6.593×10^{-2}
10% (K_c = 0.045 g)	OF	-	-	-
	OD	> 0.09	> 600	1.667×10^{-3}
	LF	0.056 - 0.09	120 - 600	6.667×10^{-3}
	LD	0.040 - 0.056	35 - 120	2.024×10^{-2}
20% (K_c = 0.085 g)	OF	-	-	-
	OD	> 0.17	> 5 500	1.818×10^{-4}
	LF	0.106 - 0.17	1 150 - 5 500	6.877×10^{-4}
	LD	0.075 - 0.106	350 - 1 150	1.988×10^{-3}
30% (K_c = 0.115 g)	OF	-	-	-
	OD	> 0.23	>24 000	4.167×10^{-5}
	LF	0.144 - 0.23	3 700 - 24 000	2.286×10^{-4}
	LD	0.101 - 0.144	1 100 - 3 700	6.388×10^{-4}
35% (K_c = 0.13 g)	OF	-	-	-
	OD	> 0.26	> 40 000	2.5×10^{-5}
	LF	0.163 - 0.26	5 000 - 40 000	1.75×10^{-4}
	LD	0.114 - 0.163	1 500 - 5 000	4.667×10^{-4}
40% (K_c = 0.15 g)	OF	-	-	-
	OD	> 0.30	> 100 000	1×10^{-5}
	LF	0.188 - 0.30	8 500 - 100 000	1.076×10^{-4}
	LD	0.132 - 0.188	2 500 - 8 500	2.824×10^{-4}
50% (K_c = 0.175 g)	OF	-	-	-
	OD	> 0.35	> 300 000	3.333×10^{-6}
	LF	0.219 - 0.35	18 000 - 300 000	5.222×10^{-5}
	LD	0.154 - 0.219	4 500 - 18 000	1.667×10^{-4}
55% (K_c = 0.185 g)	OF	-	-	-
	OD	> 0.37	> 500 000	2×10^{-6}
	LF	0.231 - 0.37	25 000 - 500 000	3.8×10^{-5}
	LD	0.163 - 0.231	5 000 - 25 000	1.6×10^{-4}

Table 2 - Frequency of Occurrence of PGA Levels Triggering Different Failure Modes (Sheet 4 of 4)

(b) Dense Fill Slopes (Ductile)				
Margin of Static Factor of Safety (K_c value shown in bracket)	Failure Mode	Range of PGA for Different Failure Modes[2] (g)	Return Period[3], T (years)	Annual Frequency of Occurrence[4], f
60% (K_c = 0.20 g)	OF	-	-	-
	OD	> 0.40	> 1 000 000	1×10^{-6}
	LF	0.25 - 0.40	35 000 - 1 000 000	2.757×10^{-5}
	LD	0.176 - 0.25	6 500 - 35 000	1.253×10^{-4}
70% (K_c = 0.225 g)	OF	-	-	-
	OD	> 0.45	> 3 000 000	3.333×10^{-7}
	LF	0.281 - 0.45	70 000 - 3 000 000	1.395×10^{-5}
	LD	0.198 - 0.281	10 000 - 70 000	8.571×10^{-5}

Legend: OF denotes overall failure
OD denotes overall deformation with local slope failure
LF denotes local failure
LD denotes local slope deformation
[1] values derived from Figure 2 for a given F_s
[2] values derived from Table 1
[3] values derived from Figure 1
[4] $f = (1/T_1 - 1/T_2)$

Table 3 - Fault Tree Analysis (Sheet 1 of 2)

Slope Type	Degree of Soil Saturation	Range of PGA K_c	Frequency of Occurrence of Range of PGA Triggering Failure			Failure Modes	Frequency of Occurrence of Failure Modes		
			$F_s = 1.1$	$F_s = 1.2$	$F_s = 1.4$		$F_s = 1.1$	$F_s = 1.2$	$F_s = 1.4$
Soil Cut Slope	High degree of soil saturation (0.5%)	> 1.4	2.000×10^{-3}	1.818×10^{-4}	8.333×10^{-6}	OF	1×10^{-5}	9.09×10^{-7}	4.167×10^{-8}
		1.0 to 1.4	4.667×10^{-3}	4.432×10^{-4}	4.722×10^{-5}	OD	2.334×10^{-5}	2.216×10^{-6}	2.361×10^{-7}
		0.88 to 1.0	3.333×10^{-3}	2.446×10^{-4}	4.444×10^{-5}	LF	1.667×10^{-5}	1.223×10^{-6}	2.222×10^{-7}
		0.63 to 0.88	2.333×10^{-2}	1.988×10^{-3}	2.333×10^{-4}	LD	1.167×10^{-4}	9.94×10^{-6}	1.167×10^{-6}
	Moderate degree of soil saturation (4.5%)	> 1.4	1.111×10^{-4}	1.754×10^{-5}	1.000×10^{-6}	OF	5×10^{-6}	7.893×10^{-7}	4.5×10^{-8}
		1.0 to 1.4	2.014×10^{-4}	8.246×10^{-5}	9.000×10^{-6}	OD	9.063×10^{-6}	3.711×10^{-6}	4.05×10^{-7}
		0.88 to 1.0	1.875×10^{-4}	6.393×10^{-5}	1.222×10^{-5}	LF	8.438×10^{-6}	2.877×10^{-6}	5.5×10^{-7}
		0.63 to 0.88	1.500×10^{-3}	3.624×10^{-4}	8.889×10^{-5}	LD	6.75×10^{-5}	1.631×10^{-5}	4×10^{-6}
	Low degree of soil saturation (95%)	>1.4	8.333×10^{-6}	2.000×10^{-6}	1.000×10^{-7}	OF	7.916×10^{-6}	1.9×10^{-6}	9.5×10^{-8}
		1.0 to 1.4	4.722×10^{-5}	1.618×10^{-5}	1.000×10^{-7}	OD	4.486×10^{-5}	1.537×10^{-5}	9.5×10^{-8}
		0.88 to 1.0	4.444×10^{-5}	2.182×10^{-5}	5.063×10^{-6}	LF	4.222×10^{-5}	2.073×10^{-5}	4.810×10^{-6}
		0.63 to 0.88	2.333×10^{-4}	1.267×10^{-4}	3.822×10^{-5}	LD	2.216×10^{-4}	1.204×10^{-4}	3.631×10^{-5}

(a) Fault Tree Analysis for Slopes with Brittle Material

Table 3 - Fault Tree Analysis (Sheet 2 of 2)

Slope Type	Degree of Soil Saturation	Range of PGA K_c	Frequency of Occurrence of Range of PGA Triggering Failure			Failure Modes	Frequency of Occurrence of Failure Modes		
			$F_s = 1.1$	$F_s = 1.2$	$F_s = 1.4$		$F_s = 1.1$	$F_s = 1.2$	$F_s = 1.4$
Sense fill Slope	High degree of soil saturation (0.5%)	N/A	-	-	-	OF	-	-	-
		> 2	1.667×10^{-3}	1.818×10^{-4}	1.000×10^{-5}	OD	8.335×10^{-6}	9.09×10^{-7}	5.0×10^{-8}
		1.25 to 2	6.667×10^{-3}	6.877×10^{-4}	1.076×10^{-4}	LF	3.334×10^{-5}	3.439×10^{-6}	5.38×10^{-7}
		0.88 to 1.25	2.024×10^{-2}	1.988×10^{-3}	2.824×10^{-4}	LD	1.012×10^{-4}	9.94×10^{-6}	1.412×10^{-6}
	Moderate degree of soil saturation (4.5%)	N/A	-	-	-	OF	-	-	-
		> 2	1.00×10^{-4}	2.50×10^{-5}	2.00×10^{-6}	OD	4.5×10^{-6}	1.125×10^{-6}	9×10^{-8}
		1.25 to 2	4.00×10^{-4}	1.75×10^{-4}	3.80×10^{-5}	LF	1.8×10^{-5}	7.875×10^{-6}	1.71×10^{-6}
		0.88 to 1.25	1.50×10^{-3}	4.667×10^{-4}	1.60×10^{-4}	LD	6.75×10^{-5}	2.1×10^{-5}	7.2×10^{-6}
	Low degree of soil saturation (95%)	N/A	-	-	-	OF	-	-	-
		> 2	1.000×10^{-5}	3.333×10^{-6}	3.333×10^{-7}	OD	9.500×10^{-6}	3.166×10^{-6}	3.166×10^{-7}
		1.25 to 2	1.076×10^{-4}	5.222×10^{-5}	1.395×10^{-5}	LF	1.022×10^{-4}	4.961×10^{-5}	1.325×10^{-5}
		0.88 to 1.25	2.824×10^{-4}	1.667×10^{-4}	8.571×10^{-5}	LD	2.683×10^{-4}	1.584×10^{-4}	8.142×10^{-5}

(b) Fault Tree Analysis for Slopes with Ductile Material

Table 4 - Frequency of Occurrence of Different Failure Modes

(a) Cut Slopes			
Failure Modes	Static Factor of Safety		
	1.1	1.2	1.4
OF	2.792×10^{-5}	3.598×10^{-6}	1.817×10^{-7}
OD	7.726×10^{-5}	2.13×10^{-5}	7.361×10^{-7}
LF	6.733×10^{-5}	2.483×10^{-5}	5.582×10^{-6}
LD	4.058×10^{-4}	1.467×10^{-4}	4.148×10^{-5}

(b) Fill Slopes			
Mode of Instability	Static Factor of Safety		
	1.1	1.2	1.4
OD	2.234×10^{-5}	5.2×10^{-6}	4.566×10^{-7}
LF	1.535×10^{-4}	6.092×10^{-5}	1.55×10^{-5}
LD	4.37×10^{-4}	1.893×10^{-4}	9×10^{-5}

Table 5 - Consequence of Different Modes of Seismic-induced Failures

Mode of Instability	Consequence of Failure of Slopes Affecting Different Facilities (Fatality per year per slope)	
	Buildings (Consequence-to-life Category 1)	Roads (Consequence-to-life Category 2)
OF	$\frac{1}{6}=1.67\text{x}10^{-1}$	$\frac{1}{8}=1.25\text{x}10^{-1}$
OD	$\frac{1}{30}=3.33\text{x}10^{-2}$	$\frac{1}{40}=2.50\text{x}10^{-2}$
LF	$\frac{1}{60}=1.67\text{x}10^{-2}$	$\frac{1}{80}=1.25\text{x}10^{-2}$
LD	$\frac{1}{300}=3.33\text{x}10^{-3}$	$\frac{1}{400}=2.50\text{x}10^{-3}$

(a) - Soil Cut Slopes

Mode of Instability	Consequence of Failure of Slopes Affecting Different Facilities (Fatality per year per slope)	
	Buildings (Consequence-to-life Category 1)	Roads (Consequence-to-life Category 2)
OF	$\frac{1}{2}=5.00\text{x}10^{-1}$	$\frac{1}{15}=6.667\text{x}10^{-2}$
OD	$\frac{1}{10}=1.00\text{x}10^{-1}$	$\frac{1}{75}=1.3337\text{x}10^{-2}$
LF	$\frac{1}{20}=5.00\text{x}10^{-2}$	$\frac{1}{150}=6.667\text{x}10^{-3}$
LD	$\frac{1}{100}=1.00\text{x}10^{-2}$	$\frac{1}{750}=1.333\text{x}10^{-3}$

(b) - Fill Slopes

Table 6 - Event Tree Analysis (Sheet 1 of 2)

Type	Event	Frequency	Consequence	Risk
	OF	2.792×10^{-5}	1.25×10^{-1}	3.49×10^{-6} (48%)
$F_s = 1.1$	OD	7.726×10^{-5}	2.50×10^{-2}	1.932×10^{-6} (27%)
Road	LF	6.773×10^{-5}	1.25×10^{-2}	8.416×10^{-7} (11%)
	LD	4.058×10^{-4}	2.5×10^{-3}	1.015×10^{-6} (14%)
				$\Sigma = 7.279 \times 10^{-6}$ (100%)

Type	Event	Frequency	Consequence	Risk
	OF	3.598×10^{-6}	1.25×10^{-1}	4.5×10^{-7} (27%)
$F_s = 1.2$	OD	2.13×10^{-5}	2.50×10^{-2}	5.325×10^{-7} (32%)
Road	LF	2.483×10^{-5}	1.25×10^{-2}	3.104×10^{-7} (19%)
	LD	1.467×10^{-4}	2.50×10^{-3}	3.67×10^{-7} (22%)
				$\Sigma = 1.66 \times 10^{-6}$ (100%)

Type	Event	Frequency	Consequence	Risk
	OF	3.598×10^{-6}	1.67×10^{-1}	6.009×10^{-7} (27%)
$F_s = 1.2$	OD	2.13×10^{-5}	3.33×10^{-2}	7.093×10^{-7} (32%)
Building	LF	2.483×10^{-5}	1.67×10^{-2}	4.147×10^{-7} (19%)
	LD	1.467×10^{-4}	3.33×10^{-3}	4.885×10^{-7} (22%)
				$\Sigma = 2.21 \times 10^{-6}$ (100%)

Type	Event	Frequency	Consequence	Risk
	OF	1.817×10^{-7}	1.67×10^{-1}	3.034×10^{-8} (10%)
$F_s = 1.4$	OD	7.361×10^{-7}	3.33×10^{-2}	2.451×10^{-8} (9%)
Building	LF	5.582×10^{-6}	1.67×10^{-2}	9.322×10^{-8} (33%)
	LD	4.148×10^{-5}	3.33×10^{-3}	1.381×10^{-7} (48%)
				$\Sigma = 2.862 \times 10^{-7}$ (100%)

(a) - Soil Cut Slopes

Table 6 - Event Tree Analysis (Sheet 2 of 2)

Type	Event	Frequency	Consequence	Risk
	OF	-	6.667×10^{-2}	-
$F_s = 1.1$	OD	2.234×10^{-5}	1.333×10^{-2}	2.978×10^{-7} (16%)
Road	LF	1.535×10^{-4}	6.667×10^{-3}	1.023×10^{-6} (54%)
	LD	4.37×10^{-4}	1.333×10^{-3}	5.825×10^{-7} (30%)
				$\Sigma = 1.903 \times 10^{-6}$ (100%)

Type	Event	Frequency	Consequence	Risk
	OF	-	6.667×10^{-2}	-
$F_s = 1.2$	OD	5.2×10^{-6}	1.333×10^{-2}	6.932×10^{-8} (10%)
Road	LF	6.092×10^{-5}	6.667×10^{-3}	4.062×10^{-7} (56%)
	LD	1.893×10^{-4}	1.333×10^{-3}	2.523×10^{-7} (34%)
				$\Sigma = 7.278 \times 10^{-7}$ (100%)

Type	Event	Frequency	Consequence	Risk
	OF	-	5.000×10^{-1}	-
$F_s = 1.2$	OD	5.2×10^{-6}	1.000×10^{-1}	5.2×10^{-7} (10%)
Building	LF	6.092×10^{-5}	5.000×10^{-2}	3.046×10^{-6} (56%)
	LD	1.893×10^{-4}	1.000×10^{-2}	1.893×10^{-6} (34%)
				$\Sigma = 5.459 \times 10^{-6}$ (100%)

Type	Event	Frequency	Consequence	Risk
	OF	-	5.000×10^{-1}	-
$F_s = 1.4$	OD	4.566×10^{-7}	1.000×10^{-1}	4.566×10^{-8} (3%)
Building	LF	1.55×10^{-5}	5.000×10^{-2}	7.75×10^{-7} (45%)
	LD	9×10^{-5}	1.000×10^{-2}	9×10^{-7} (52%)
				$\Sigma = 1.721 \times 10^{-6}$ (100%)

(b) - Fill Slopes

Table 7 - Calculated Risk of Earthquake-induced Landslides

(a) Soil Cut Slopes				
Type of Cut Slopes	New Slopes		Upgraded pre-GCO Slopes	
Facility Affected	Buildings	Roads	Buildings	Roads
Consequence-to-life Category	1	2	1	2
Design Static Factor of Safety	1.4	1.2		1.1
Average Annual Risk of Earthquake-induced Landslides (Fatality per year per slope)	2.862×10^{-7}	1.66×10^{-6}	2.21×10^{-6}	7.279×10^{-6}

(b) Compacted Fill Slopes				
Type of Fill Slopes	New Slopes		Upgraded pre-GCO Slopes	
Facility Affected	Buildings	Roads	Buildings	Roads
Consequence-to-life Category	1	2	1	2
Design Static Factor of Safety	1.4	1.2		1.1
Average Annual Risk of Earthquake-induced Landslides (Fatality per year per slope)	1.721×10^{-6}	7.278×10^{-7}	5.459×10^{-6}	1.903×10^{-6}

Table 8 - Comparison of Risks of Landslides Caused by Rainfall and Earthquake Respectively (Sheet 1 of 2)

(a) Risk of Rain-induced Landslides at Pre-1977 Soil Cut Slopes

	Buildings	Roads
Global Annual Risk (Fatality per year per slope)	1/100 x 1/30 = 3.33×10^{-4}	1/100 x 1/40 = 2.5×10^{-4}
Note: See Wong & Ho (1998) for details of the risk assessment.		

(b) Risk of Rain-induced Landslides at Pre-1977 Fill Slopes

	Buildings	Roads
Global Annual Risk (Fatality per year per slope)	1/525 x 1/10 = 1.90×10^{-4}	1/525 x 1/70 = 2.72×10^{-5}
Note: See Wong & Ho (1998) for details of the risk assessment.		

(c) Risk of Earthquake-induced Landslides at New or Upgraded Soil Cut Slopes

Static FOS	Buildings	Roads
1.4	2.862×10^{-7} (≈ 0.08%)	N/A
1.2	2.21×10^{-6} (≈ 0.7%)	1.66×10^{-6} (≈ 0.7%)
1.1	N/A	7.279×10^{-6} (≈ 2.9%)
Note: The figure shown in bracket is the ratio of the risk of earthquake-induced failure for post-GCO soil cut slopes (complying with current standards) to the risk of rain-induced failure for pre-GCO soil cut slopes.		

Table 8 - Comparison of Risks of Landslides Caused by Rainfall and Earthquake Respectively (Sheet 2 of 2)

(d) Risk of Earthquake-induced Landslides at Recompacted Fill Slopes

Static FOS	Buildings	Roads
1.4	1.721×10^{-6} (≈ 0.9%)	N/A
1.2	5.459×10^{-6} (≈ 2.9%)	7.278×10^{-7} (≈ 2.7%)
1.1	N/A	1.903×10^{-6} (≈ 7%)

Note: The figure shown in bracket is the ratio of the risk of earthquake-induced failure for post-GCO fill slopes (i.e complying with current standards) to the risk of rain-induced failure for pre-GCO fill slopes.

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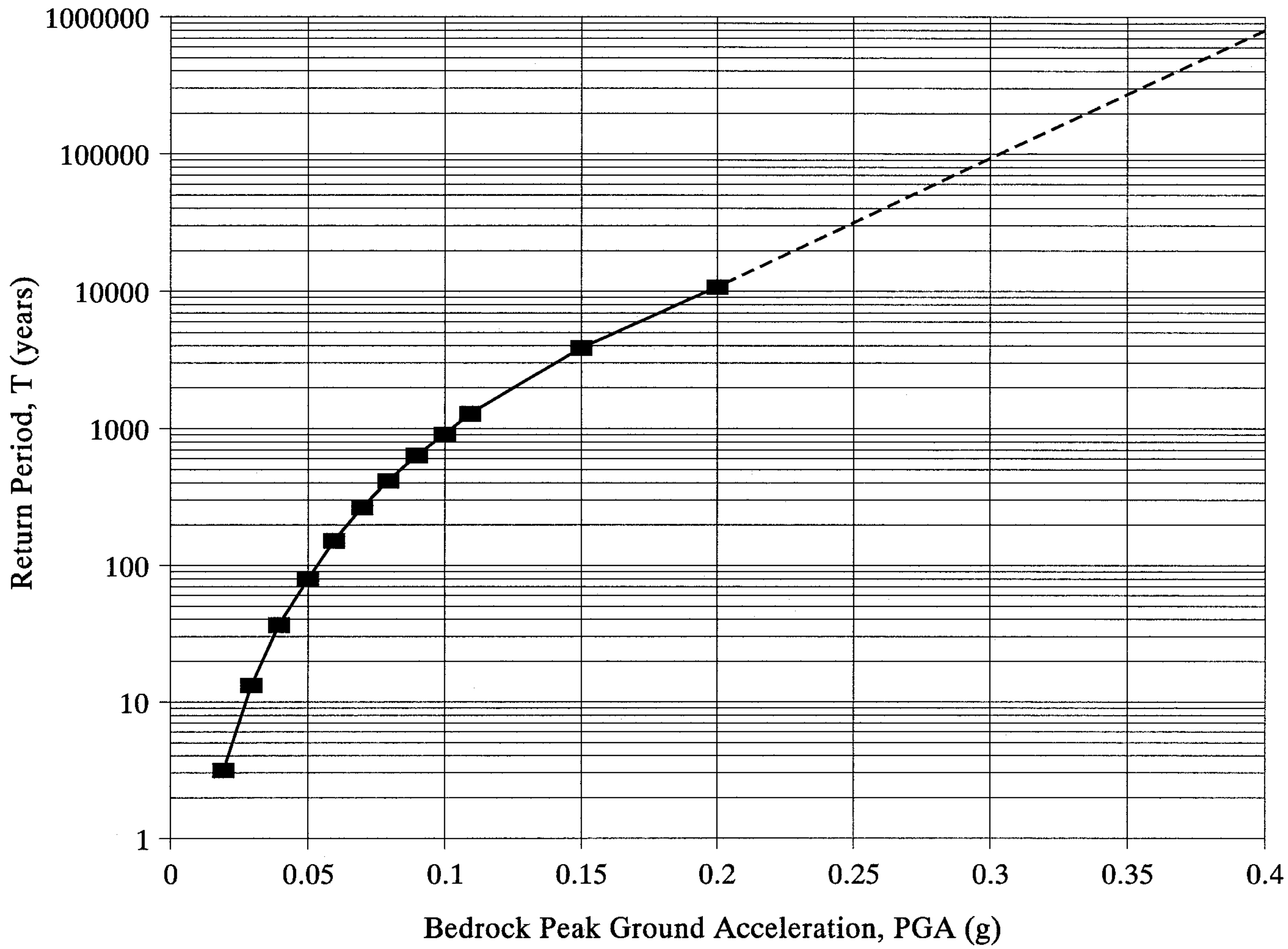
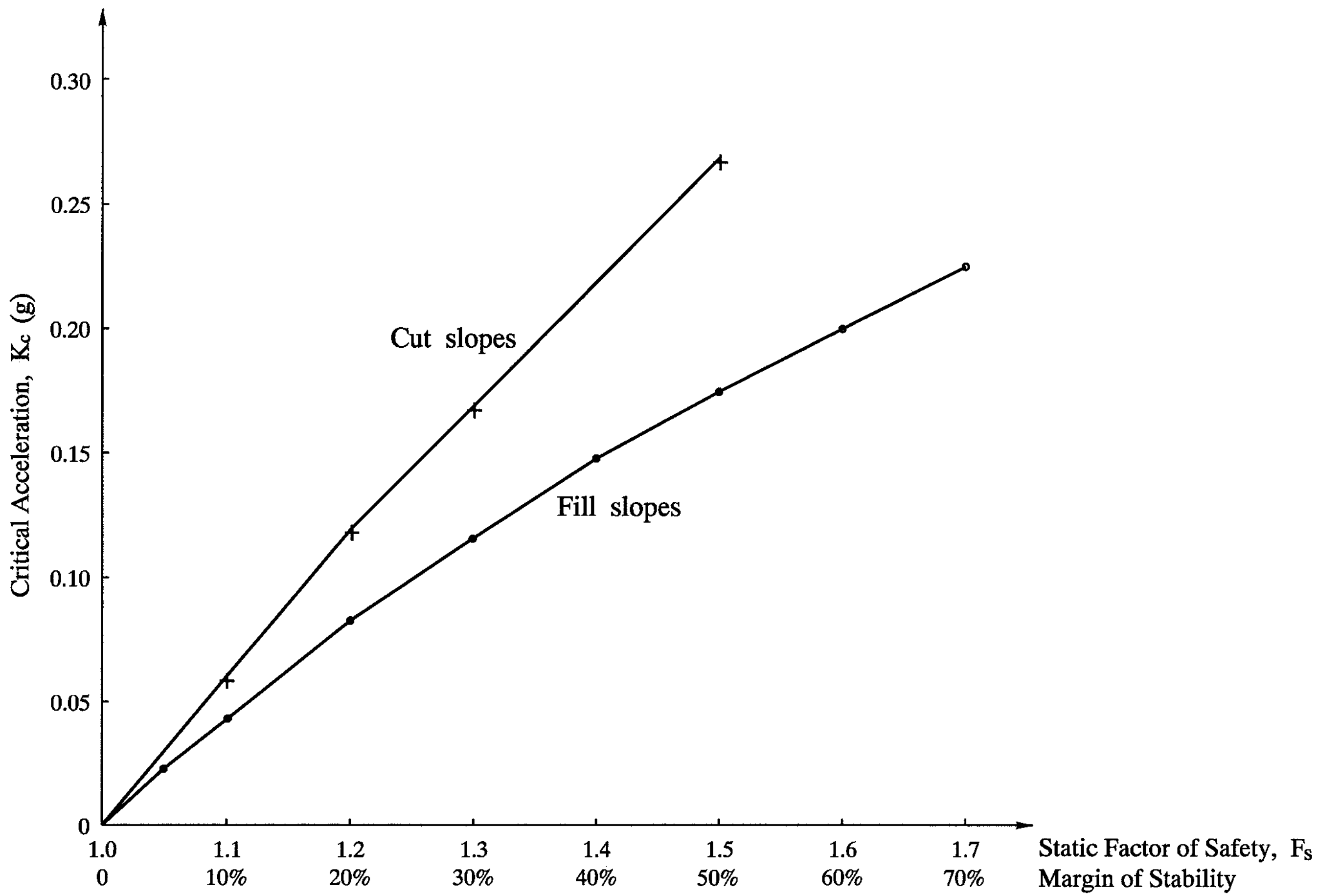


Figure 1 - Peak Ground Acceleration at Bedrock for Different Return Period (after Pun & Ambraseys, 1992)

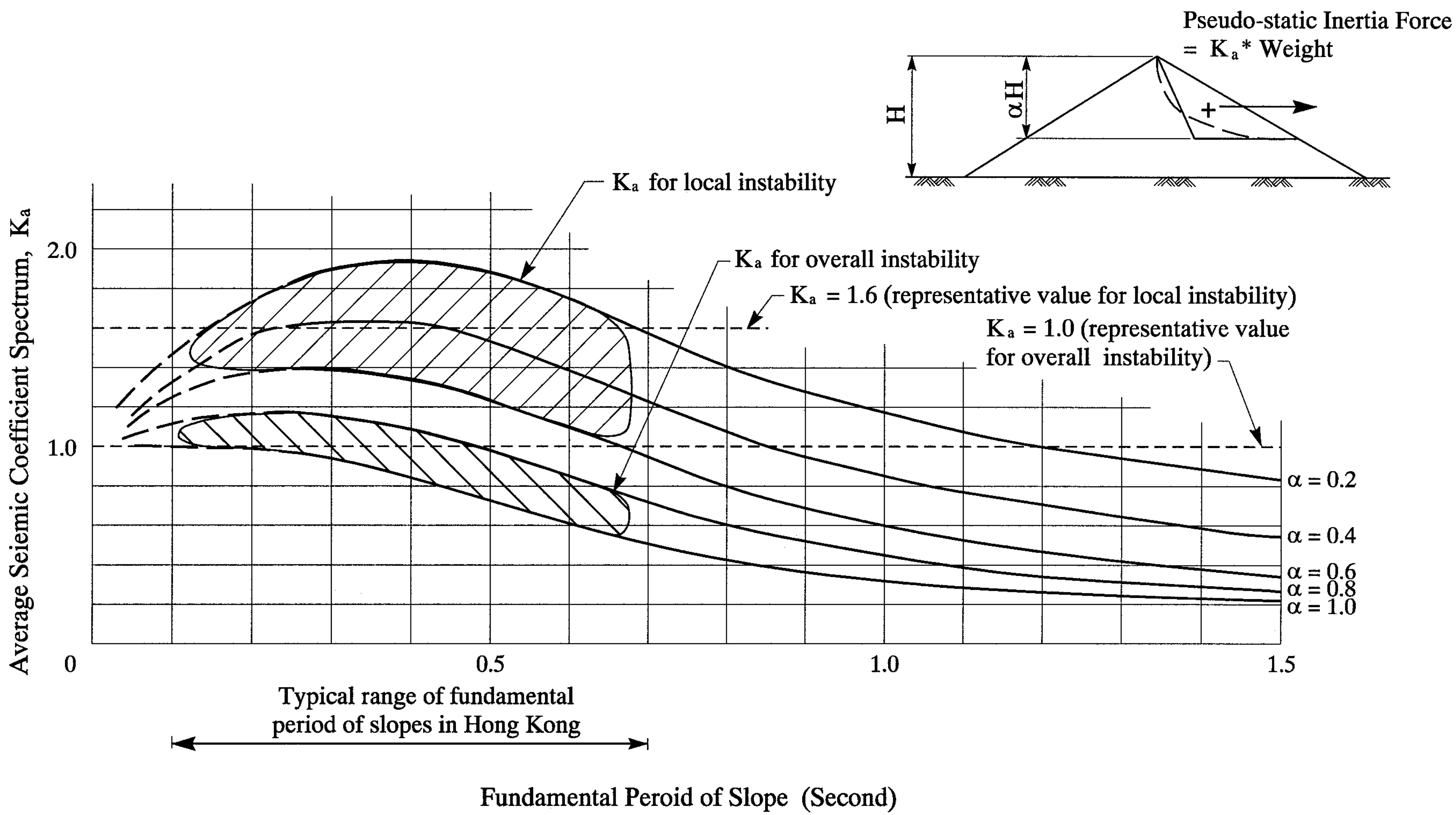


Notes :

(1) No excess pore water pressure generated form seismic vibration is considered.

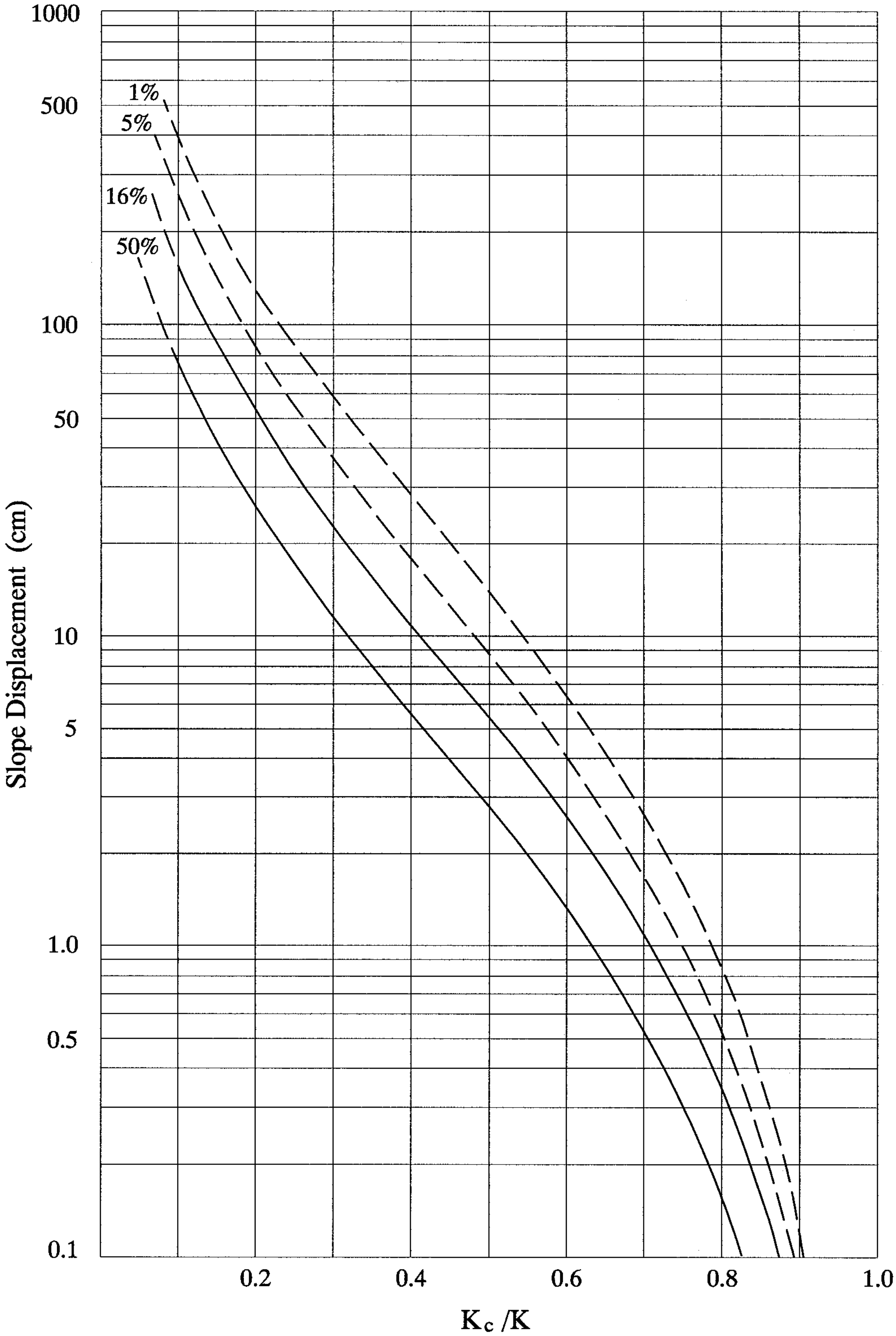
(2) Typical soil parameters for cut and fill slopes in Hong Kong are considered :
- for fill slopes : $c' = 0$, $\phi' = 30°$ to $40°$
- for cut slopes : $c' = 5$ kN/m^2 to 10 kN/m^2 , $\phi' = 35°$ to $45°$

Figure 2 - Critical Accelerations for Slopes with Different Static Factors of Safety



Note : Damping = 20 %

Figure 3 - Average Seismic Coefficient Spectrum (after Ambraseys & Sarma, 1967)



Notes : (1) K_c = Critical acceleration (g)
K = Net acceleration at the soil mass (g)
(2) Precentage shown is the probability of occurrence

Figure 4 - Slope Displacements for Different K_c/K Ratios
(after Ambraseys & Manu, 1988)

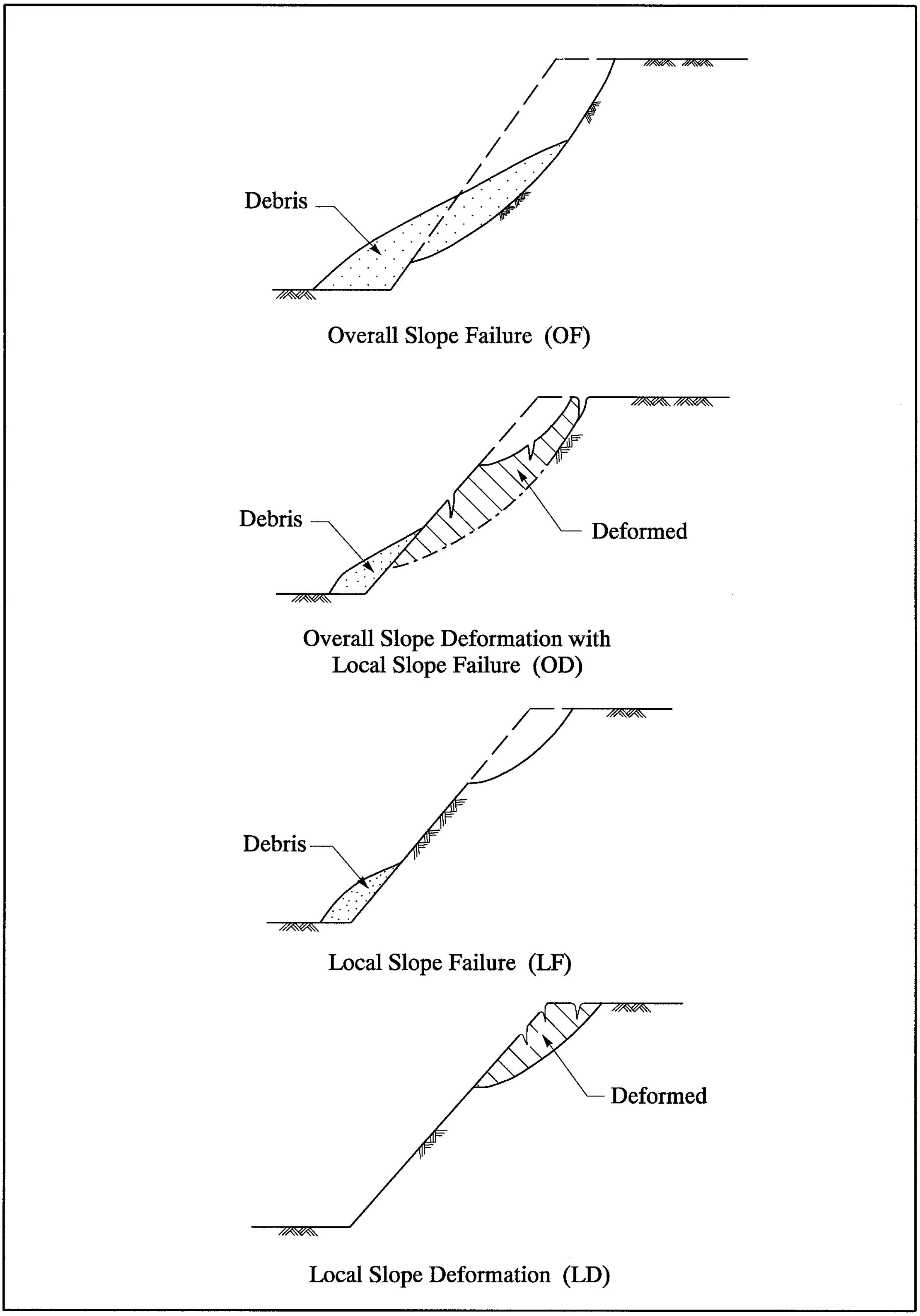
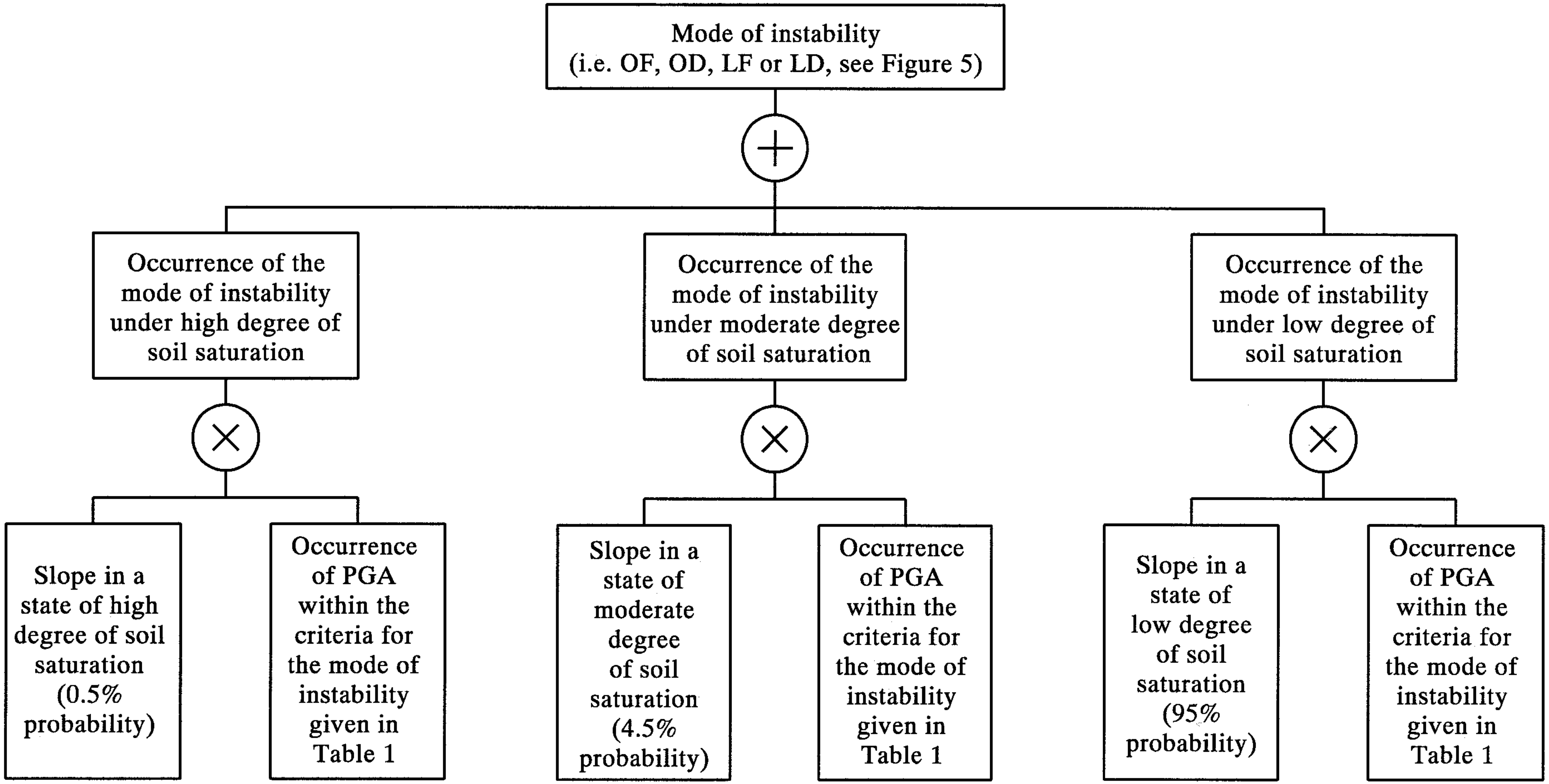


Figure 5 - Modes of Seismic-induced Slope Instability



Legend :

⊕ 'or' gate   ⊗ 'and' gate

Notes :
(1) See Appendix C for calculation of the frequency of occurrence of different degree of soil saturation.
(2) See Table 2 for calculation of the frequency of occurrence of different PGA events.
(3) See Tables 1 and 2 for calculation of the frequency of occurrence of different modes of instability.

Figure 6 - Fault Tree for Different Modes of Instability

APPENDIX A

A GLOSSARY OF QRA TERMINOLOGY ADOPTED

CONTENTS

A.1 QRA TERMINOLOGY ADOPTED IN THIS REPORT

A number of terms related to quantitative risk assessment have been adopted in this Report. The definitions of these terms are given below:

Landslide Hazard -	A condition with the potential for causing an undesirable consequence. The description of landslide hazard should include the location, volume and velocity of the potential landslide.
Hazard Identification -	The recognition that a landslide hazard exists and the assessment of its characteristics.
Frequency Analysis -	The evaluation of the number of occurrence of the hazards within a given time period (frequency denotes the likelihood of hazard occurrence).
Consequence Analysis -	The evaluation of the outcomes arising from the occurrence of hazards (consequence denotes the severity of damage in the event of hazard occurrence).
Risk -	A measure of the likelihood and severity of an adverse effect resulting from the occurrence of a hazard within a specified time period. For practical purposes, risk may be taken as the product of frequency and consequence).
Quantitative Risk Assessment -	The quantitative evaluation of risk through integrating the results of frequency and consequence analyses and exercising value judgment regarding the outcome of the evaluation.
Risk Management -	The complete process of risk assessment and risk control.
Individual Risk -	The risk of fatality or injury to any identifiable individual at a given locality within the zone affected by the hazard.
Societal Risk -	The risk of multiple fatalities or injuries that the population located within the zone affected by the landslide hazard is collectively exposed to.
Tolerable Risk -	The risk level that society is prepared to live with in order to secure certain net benefits in the confidence that it is being properly managed. In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk

even though they recognize it is unacceptably high.

Acceptable Risk - The risk level that society is willing to bear and where expenditure to achieve further risk reduction is not considered justifiable.

Vulnerability - A measure of the degree of loss to a given element or within the area affected by the hazard, having regard to the characteristics of the hazard and the type and proximity of the affected elements. It is expressed on a scale of 0 (no loss) to 1 (total loss).

Probability - A measure of the likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes.

APPENDIX B

CALCULATION OF CRITICAL ACCELERATION

CONTENTS

B.1 GENERAL

The critical acceleration (K_c) of typical soil cut slopes and dense fill slopes with different static Factors of Safety (F_s) in Hong Kong is calculated in order to derive the generalised correlation between K_c and F_s. The following assumptions are made in the assessment:

(a) Fill slopes are considered to be composed of a frictional soil (i.e. effective cohesion, c' = 0) with an effective angle of shearing resistance (ϕ') varying from 30° to 40°, and a total of 18 cases with F_s ranging from 1.05 to 1.5 were analysed.

(b) Cut slopes are considered to be composed of a c'-ϕ' soil, and a total of 34 cases (c' between 5 kN/m^2 and 10 kN/m^2 and ϕ' between 35° and 45°) with F_s ranging from about 1.1 to 1.6 were analysed.

(c) Limit equilibrium slope stability analyses were carried out in order to calculate the values of F_s of the slopes and pseudo-static method of seismic slope stability analyses was used to calculate the corresponding K_c values.

(d) Fill slopes that are not prone to collapse were analysed using the infinite slope assumptions and hence the available closed-form solutions (e.g. Wong & Pang, 1992) were adopted to calculate K_c and F_s. Typical soil parameters of 35° to 40° were assumed.

(e) Cut slopes were analysed using the method of slices following the recommendations by Sarma (1973) using the computer program "EQS" (Sarma, 1974) to calculate K_c and F_s.

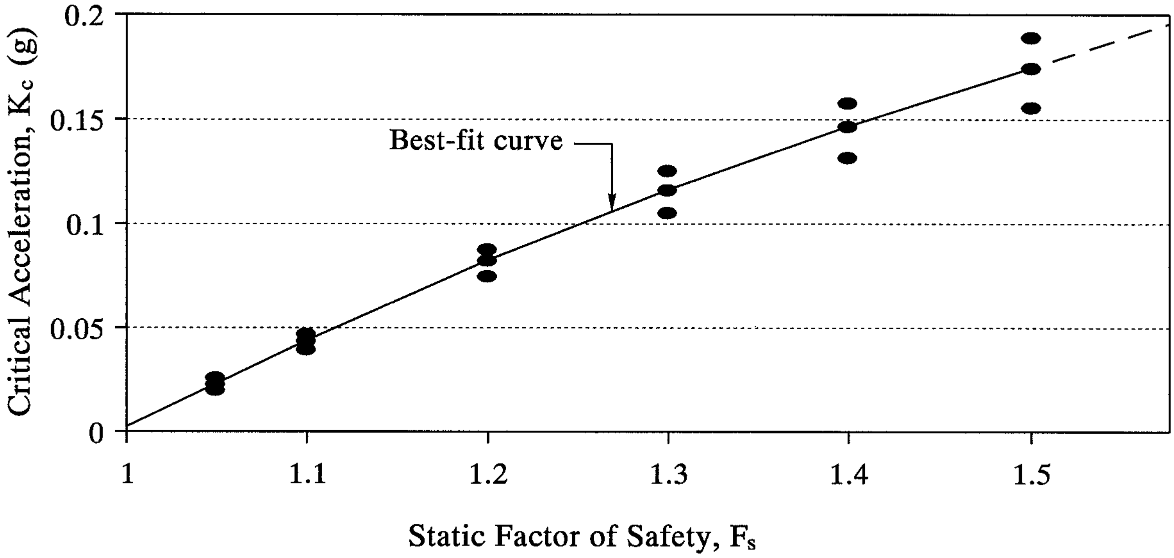
(f) It is assumed that no excess pore water pressure will be generated due to earthquake loading - this assumption applies to slopes with a low degree of saturation (i.e. where Skempton (1954)'s pore pressure coefficient B approaches zero), and for soils with a small dynamic pore pressure parameter, A_n (Sarma & Jennings, 1980).

(g) Sensitivity analyses were carried out to examine the correlations between K_c and F_s for the case where excess pore water pressure would be generated during earthquake loading for loose soils that are under a high degree of saturation (i.e. assuming B = 1 and A_n = 0.5).

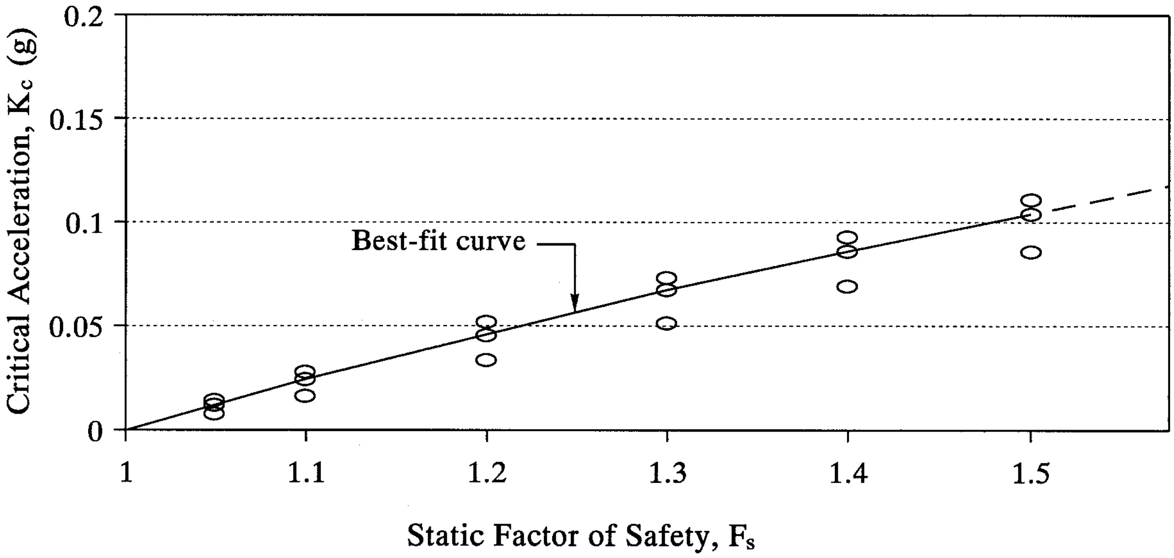
B2 RESULTS OF ANALYSIS

The results of the analyses for fill slopes are shown in Figure B1 whereas those for cut slopes are shown in Figure B2. The lines given in the Figures provide a reasonable fit to the results. These lines are shown in Figure 2 of this Report and have been used for the QRA.

LIST OF FIGURES

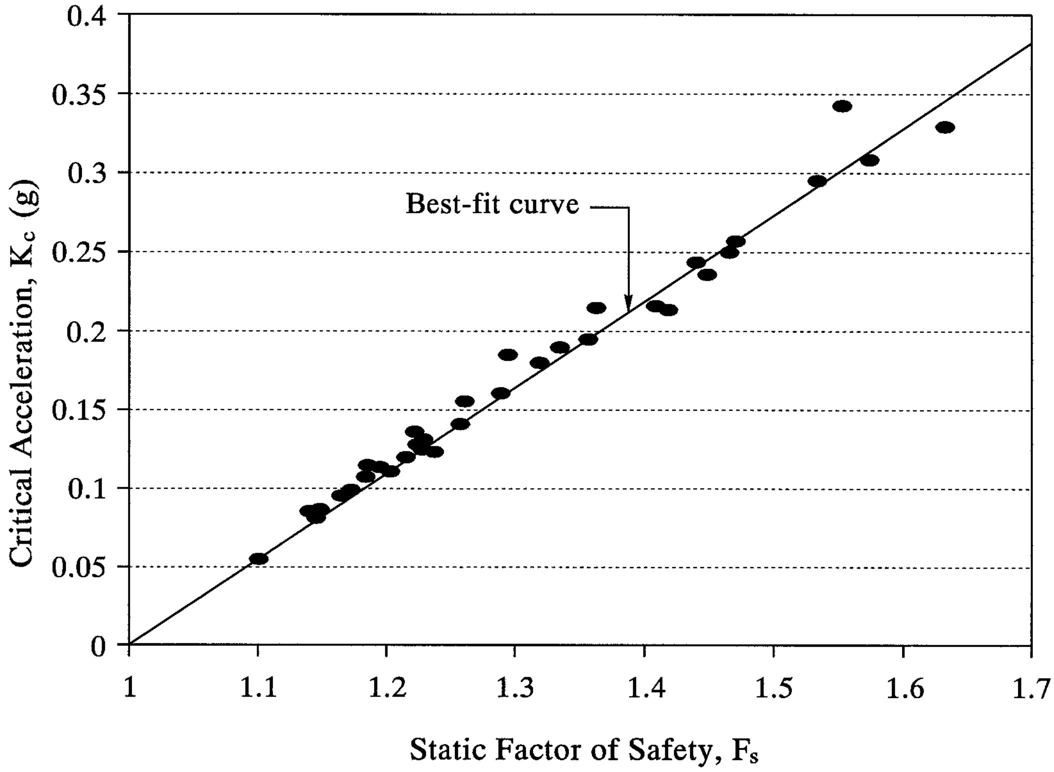


(a) Cases with No Excess Pore Water Pressure Generation (e.g. B=0)

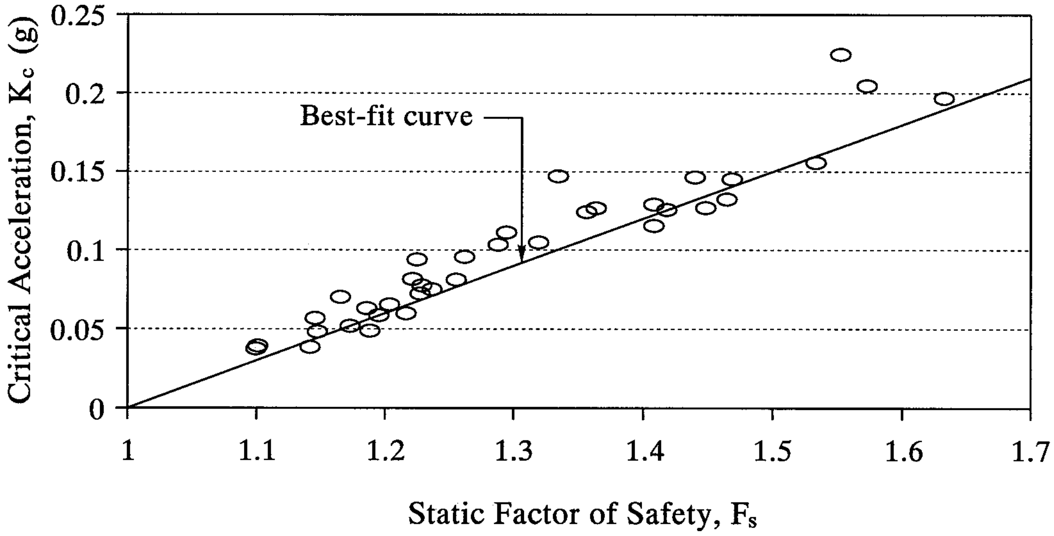


(a) Cases with Excess Pore Water Pressure Generation (e.g. B=1.0, A=0.5)

Figure B1 - Critical Accelerations of Typical Fill Slopes



(a) Cases with No Excess Pore Water Pressure Generation (e.g. B=0)



(a) Cases with Excess Pore Water Pressure Generation (e.g. B=1.0, A=0.5)

Figure B2 - Critical Accelerations of Typical Cut Slopes

APPENDIX C

ASSESSMENT OF LIKELIHOOD OF DIFFERENT DEGREES OF SOIL SATURATION

CONTENTS

C.1 GENERAL

During periods of dry weather, slopes are in a state of low degree of soil saturation and soil suction is developed. However, in the event of heavy rain, prolonged infiltration of water into the soil will bring about a high degree of saturation, destroying the soil suction in the process. In this Appendix, an assessment is made of the likelihood of slopes in Hong Kong being in differing states soil saturation (viz. low, moderate and high degree of saturation) at the time of an earthquake which occurs randomly in time and only lasts for a very short period. The following factors are considered in the assessment:

(a) the threshold rainfall that is required to bring a typical slope to a moderate or higher degree of soil saturation respectively, and

(b) the frequency of occurrence of rainfall exceeding the threshold value based on the available rainfall records in Hong Kong.

C.2 THRESHOLD RAINFALL

Data on changes in soil suction in slopes in Hong Kong as a result of heavy rain are scarce. Some early work on field measurements of soil suction of slopes in Hong Kong was reported by Anderson (1984) and a review of work on suction measurements is recently presented by Shen (1997). However, a generalised correlation between rainfall and soil suction cannot be confidently developed from the limited field data available.

In order to estimate the likely order of the threshold rainfall for the present assessment, a simplified deterministic approach has been adopted as described below.

From the wetting band theory (Lumb, 1962), the amount of infiltration (I_f) required to wet a soil layer of depth h from the slope surface is given by the following equation:

$$I_f = k.t = h.n. (S_f - S_o) \qquad \text{(C1)}$$

where k = permeability of the soil
t = duration of rainfall
n = porosity of the soil
S_f = final degree of soil saturation
S_o = initial degree of soil saturation

To bring a typical soil slope with, say, n = 35% to a moderate or higher degree of saturation, I_f would be of the order of about 50 mm for h = 1 m and $S_f - S_o = 0.15$ based on the above equation.

The required amount of rainfall is also expected to be of this order, taking into account that part of the rainfall that may not infiltrate into the slope because of surface runoff could be offset, to a certain extent, by subsurface recharge from the upslope areas which would contribute to increasing the degree of soil saturation. This order of rainfall is equivalent to a moderately heavy rain with an intensity of about 10^{-6} m/s (which is also the typical k value of saprolite and fill in Hong Kong) which lasts for about half to one day. Hence, the threshold rainfall is taken to be 50 mm/day for the present purposes.

As for the threshold for achieving a high degree of soil saturation, reference is made to the statistical findings of historical rainfall data reported by Lam & Leung (1994). For the present assessment, a daily rainfall of 380 mm (which corresponds approximately to a return period of 10 years) is taken to be the threshold for the condition of a high degree of soil saturation.

C.3 ANALYSIS OF RAINFALL RECORDS

The digitised records of daily rainfall taken from 48 automatic raingauges over the period 1 January 1990 to 31 December 1996 were analysed. For each of the raingauges, the number of days for which the rainfall exceeded the threshold value of 50 mm/day and 380 mm/day was counted. The results are summarised in Figures C1 and C2.

C.4 PROBABILITY OF EXCEEDANCE OF THRESHOLD RAINFALL

With reference to Figure C1, the probability that slopes will be in a state of moderate or higher degree of saturation may be taken to be approximately 5% at any given transient time interval when slopes are subjected to earthquake loading.

Based on Figure C2, it is conservatively assumed that the probability that slopes will be in a state of high degree of saturation corresponds to 0.5%. Hence, the probability that slopes will be in a state of moderate degree of saturation is 4.5%.

The probability that slopes are in a state of low degree of saturation at the time of an earthquake is therefore taken as 95%.

C.5 REFERENCE

Lam, C.C. & Leung, Y.K. (1994). Extreme Rainfall Statistics and Design Rainstorm Profiles at Selected Locations in Hong Kong. Royal Observatory Technical Note No. 86, 89p.

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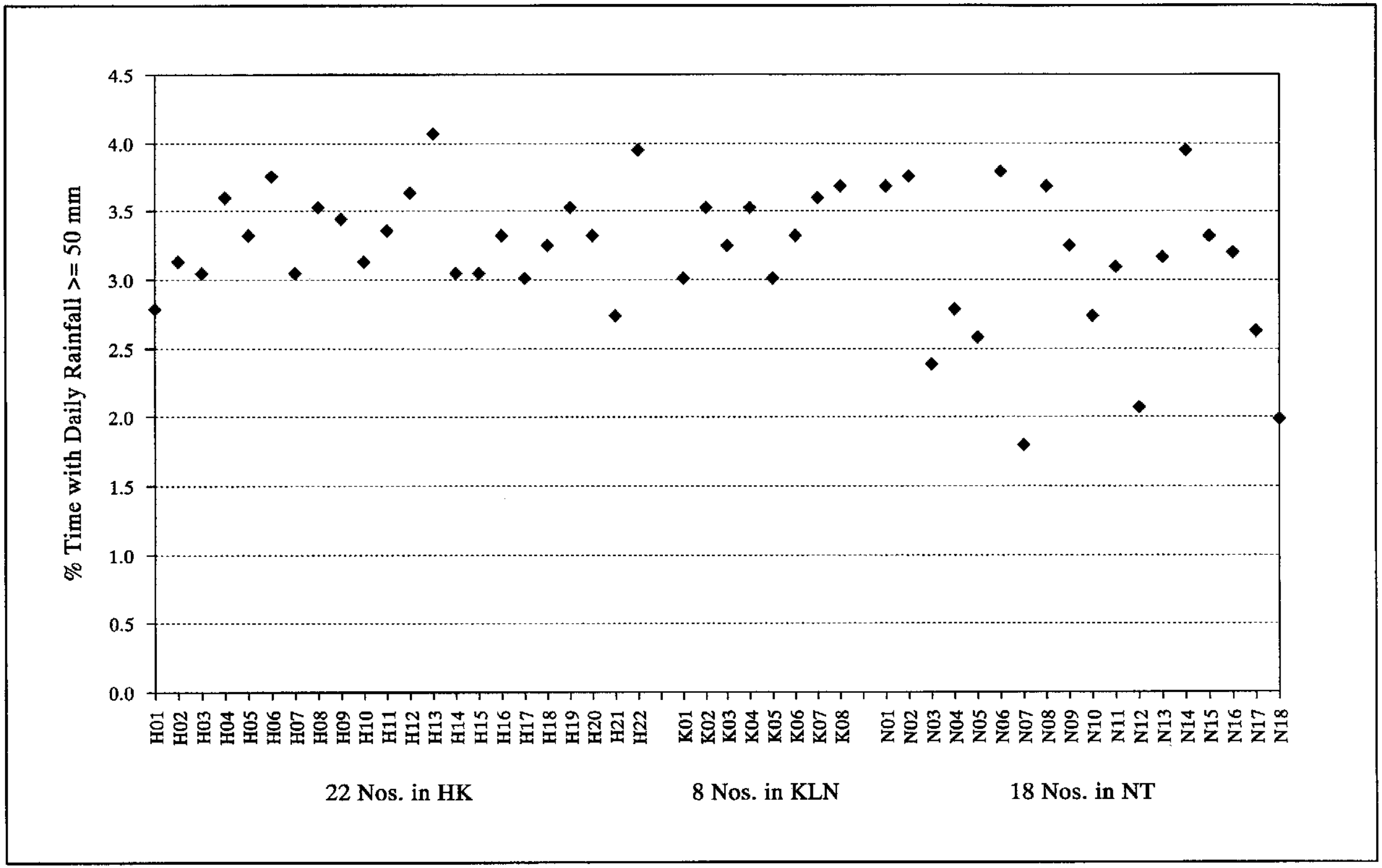

Figure C1 - Percentage of Raingauge-days with Daily Rainfall Exceeding 50 mm

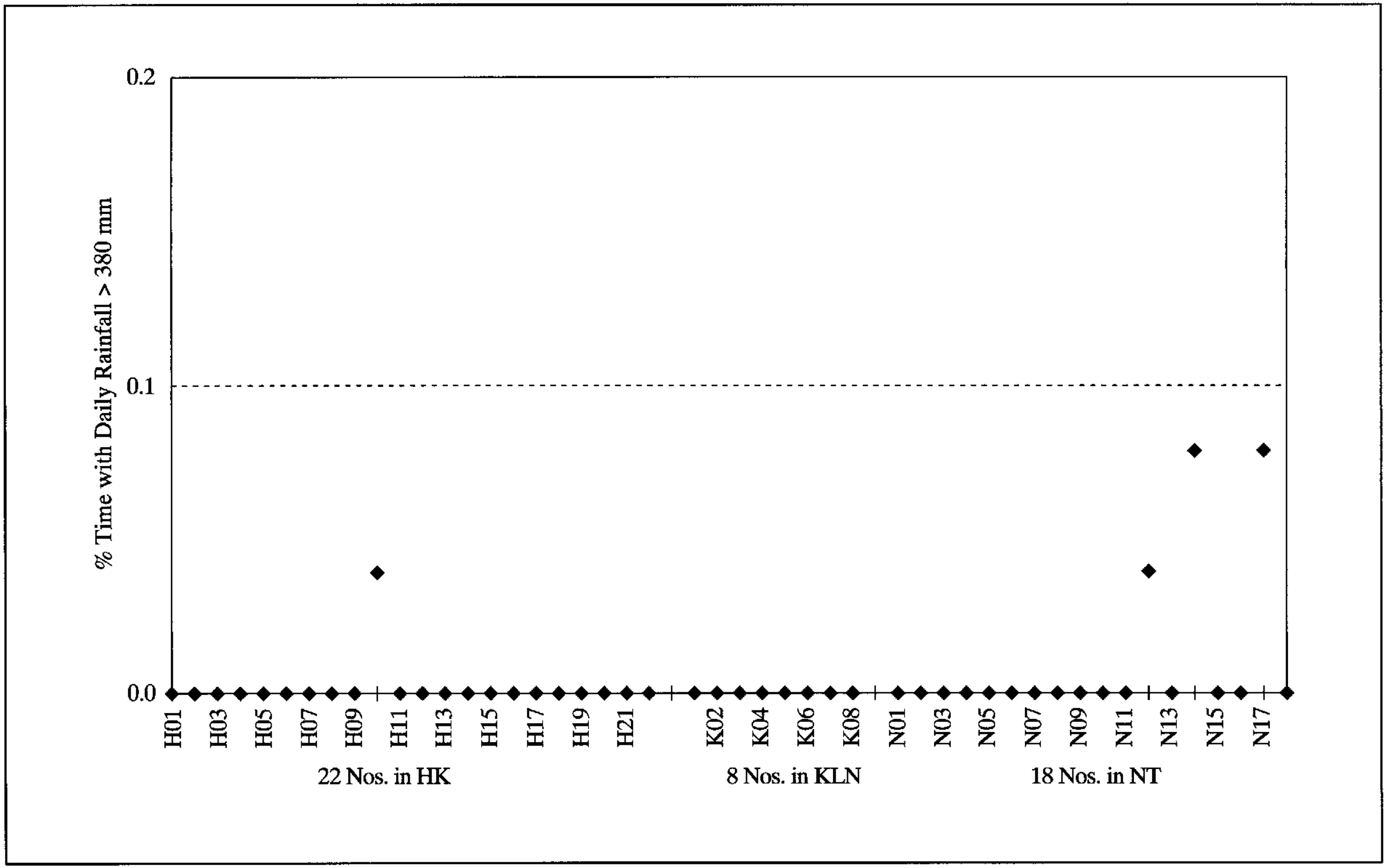

Figure C2 - Percentage of Raingauge-days with Daily Rainfall Exceeding 380 mm (1990 - 1996)

APPENDIX D

ASSESSMENT OF ADDITIONAL MARGIN OF STABILITY FOR SLOPES UNDER A LOW DEGREE OF SATURATION

CONTENTS

D.1 GENERAL

The design Factor of Safety of a slope relates to a ten-year return period rainfall condition (GCO, 1984) and current design practice usually assumes that the soil would be in a fully saturated state under the design rainfall. When the slope is under a low degree of saturation, the presence of soil suction would enhance the shear strength and the actual Factor of Safety of the slope will be higher than the design value.

An attempt has been made to quantity the additional margin of stability (i.e. the difference between the actual Factor of Safety and the design Factor of Safety) for typical slopes in Hong Kong. The following assumptions were made in the assessment:

(a) The difference in the Factor of Safety of the slope between the ten-year rainfall condition and the fully saturated state is neglected - this is generally conservative and is acceptable since the main groundwater table for the majority of the slopes in Hong Kong is fairly low.

(b) Slopes under a low degree of saturation are taken to have an average soil suction of 15 kN/m^2 based on field monitoring results reported by Anderson (1984) and those recorded in recent forensic landslide studies which indicate that suction of this order is present within the top few metres of the soil below the ground surface.

(c) The unsaturated shear strength of a partially saturated soil is given by the following (Fredlund & Rahardjo, 1993):

$$\tau_f = c' + (\sigma_f - u_a)_f \tan \phi' + (u_a - u_w)_f \tan \phi^b$$

where
- τ_f = shear stress on the failure plane at failure
- c' = apparent cohesion in terms of effective stress
- $(\sigma_f - u_a)_f$ = net confining pressure on the failure plane at failure
- σ_f = total normal stress on the failure plane at failure
- u_a = pore air pressure
- ϕ' = angle of shearing resistance associated with the net normal stress state variable $(\sigma_f - u_a)_f$
- $(u_a - u_w)_f$ = matric suction at failure
- u_w = pore water pressure
- ϕ^b = notional angle of shearing resistance that denotes the rate of change in shear strength relative to changes in matric suction

(d) The value of ϕ^b is taken to be 35° based on the findings of Gan & Fredland (1992 & 1994) that ϕ_b is approximately equal to the ϕ' value of the soil for low matric suctions within about 20 kN/m^2.

D.2 RESULTS OF ASSESSMENT

Limit equilibrium slope stability analyses were carried out with the use of the computer program "EQS" (Sarma, 1974). The Factor of Safety corresponding to saturated shear strengths and the additional margin of stability due to the component of unsaturated soil strengths were analysed.

The results of the analyses are summarised in Figure D1. It should be noted that the data points of large additional margin of stability shown in Figure D1 correspond to shallow slip surfaces which are more vulnerable to loss of soil suction during heavy rains. For this reason, a more conservative value of 0.3 is taken for the additional margin of stability which is adopted in the QRA.

For a moderate degree of soil saturation, the additional margin of safety may be taken to be 0.15.

LIST OF FIGURES

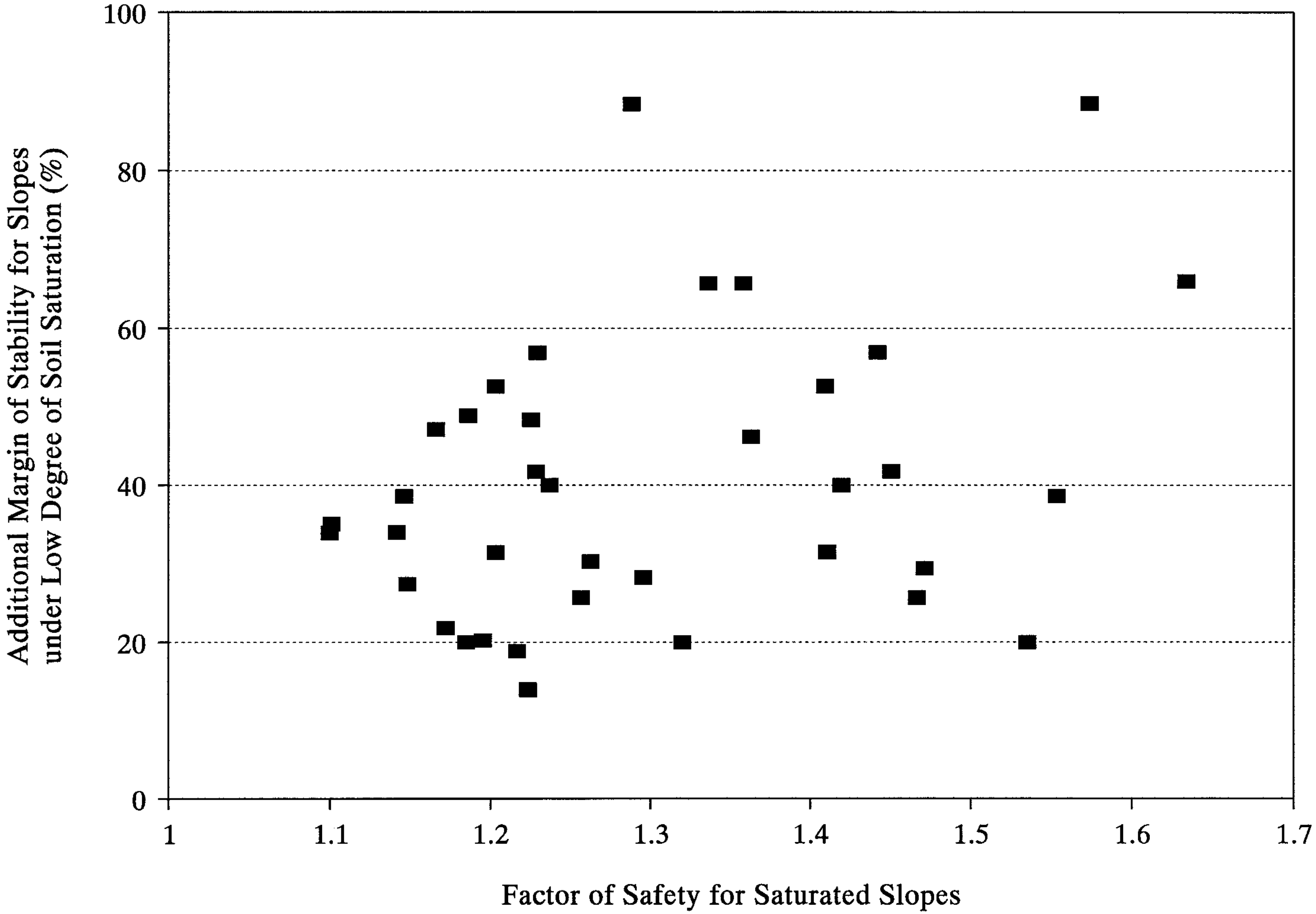

Figure D1 - Additional Margin of Stability Due to Soil Being in an Unsaturated State

APPENDIX E

DERIVATION OF THE TRIGGER CRITERIA FOR DIFFERENT FAILURE MODES

CONTENTS

E.1 DERIVATION OF THE TRIGGER CRITERIA FOR DIFFERENT FAILURE MODES

The failure trigger criteria, expressed in terms of the ratio of PGA to K_c, for the occurrence of the different modes of earthquake-induced instabilities are shown in Table 1. These have been derived based on the following considerations:

(a) From the dynamic response analyses (Section 2.4), the representative average values of seismic coefficient spectrum for the different failure modes are as follows:

K_a = 1.0 for overall slope failure or deformation

K_a = 1.6 for local slope failure or deformation

(b) Seismic displacement analyses (Section 2.5) indicate the following for slopes with brittle characteristics:

- Slope failure (both overall and local failures) occurs when $K_c/K < 0.7$. This corresponds to a slope displacement of the order of 10 mm to 30 mm (i.e. approaching 0.5% strain for a relatively small slope of 2 m to 6 m in height) before significant reduction in soil strength takes place.

- significant slope deformation (overall and local) occurs when $K_c/K < 1.0$.

(c) Seismic displacement analyses (Section 2.5) indicate the following for ductile slopes:

- Overall slope failure is unlikely.

- Overall slope deformation and local slope failure will become a problem when $K_c/K < 0.5$ (the corresponding slope displacement is of the order of 100 mm).

- Local slope deformation becomes a problem when $K_c/K < 0.7$ (when $K_c/K \geq 0.7$, the slope displacement is generally small, being less than about 10 mm).

(d) The respective ratio of PGA to K_c for the different failure modes was calculated from the above using the equation $K = K_a$ *PGA.

In this Report, brittle slopes refer to those slopes which exhibit a significant post-peak reduction in soil shear strength. Typical examples include cut slopes in saprolite for which the c' component may reduce substantially upon failure (e.g destruction and breaking of bonds).

Ductile slopes refer to those slopes which do not exhibit a significant post-peak reduction in soil shear strength. Typical examples include dense fill slopes and unsaturated fill slopes.

APPENDIX F

ASSESSMENT OF CONSEQUENCE-TO-LIFE FOR DIFFERENT FAILURE MODES

CONTENTS

F.1 CONSEQUENCE-TO-LIFE IN THE EVENT OF SLOPE FAILURES

Reference was made to the available historical landslide data as well as the generalised landslide consequence model and the global QRA as described by Wong et al (1997) to try to establish the likely consequence of failure. The likely landslide fatality figures took into account the nature and distribution of the affected facilities as well as the nature of the different failure modes.

For soil cut slopes, the assessed global consequence to life (in terms of fatality per year) for buildings and roads is 1 in 30 and 1 in 40 respectively. The available landslide data suggest that about 10% of the landslides are major failures and 90% are minor failures. By fitting the global landslide consequence data, the consequence to life of a major landslide (i.e. overall slope failure, OF) is 1 in 6 and 1 in 8 for buildings and roads respectively, and the corresponding consequence to life of a minor landslide (i.e. local slope failure, LF) is 1 in 60 and 1 in 80 for buildings and roads respectively. For instability modes OD and LD, the consequence to life will be less and the corresponding consequences-to-life are reduced notionally by a factor of five compared to instability modes OF and LF respectively. The consequence-to-life assumed for the various instability modes is summarised in Table 5.

For fill slopes, the assessed global consequence to life to buildings and roads is 1 in 10 and 1 in 70 respectively. Similar assumptions as for soil cut slopes are made regarding the consequence-to-life for the different failure modes for fill slopes (see Table 5).