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STUDY REPORT

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Calibration Studies for Landslide Quantitative Risk Assessment

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EXECUTIVE SUMMARY

Geotechnical practitioners in the past have relied heavily on qualitative judgement to assess whether the stability of a slope is adequate to allow development, complemented by the use of numerical techniques such as the factor of safety concept. Quantitative risk assessment (QRA) techniques that have been applied to areas such as the nuclear and hazardous waste industries and dam safety can also be used to quantify the uncertainty associated with the stability of slopes. These techniques use both numerical methods and subjective judgement to express numerically the risk attached to any particular slope. Although procedures are well established, the use of QRA to evaluate slope stability risk is very much an emerging concept overseas and still in its infancy in New Zealand.

One of the recommendations arising out of a previous research study for the Building Research Research Association of NZ (BRANZ) was that a series of case studies should be carried out to assess the usefulness of QRA techniques, "with emphasis on the probabilistic characterisation of landslide hazard and quantification of the risk". The principle objective of this publication is to present the use of QRA in some New Zealand slope stability case studies to highlight its use for evaluating risk acceptability, and to describe its advantages and limitations.

The basic framework and terminology for slope stability QRA are reviewed.

Four case studies were selected; one each in Tauranga and Queenstown and two in Nelson, The Tauranga study involved a rural residential subdivision where, following a period of prolonged rainfall a landslide was activated on an upslope property. The failure resulted in destruction of a detached garage and minor damage to a dwelling. Subsequently, civil proceedings against the territorial authority, the land developers and their engineering consultants were initiated on the basis that the land was of inadequate long-term stability. Due to these proceedings a large amount of information about ground conditions was gathered, which has proved to be suitable for use in QRA. Rainfall records were also evaluated. The QRA analysis showed that there was a high probability of a landslide and that the risk of death to an occupant of a dwelling in the subdivision was unacceptably high, and that also the risk of damage to buildings would be thought unacceptable.

The Queenstown example involved a substantial slope failure during excavation of building platforms for townhouses on a sloping site. The landslide affected much of the site and encroached onto two adjoining properties. Study of the schist rock showed it to contain defects (foliation shears and joints) that were predisposed to cause failure. There was a commercial risk as the site is no longer suitable for residential development and a potential health and safety risk to site workers. QRA showed that while the risk may have been acceptable in commercial terms, it would have highlighted the risk to the developer prior to landsliding, allowing preventative measures to have been carried out.

At a site in Nelson heavy rainfall triggered a landslide from a natural slope that narrowly missed a house. Subsequent site investigations showed the slope was unstable and the house was removed to prevent possible total loss. Several older landslips were evident on the slope above the site. QRA involving rainfall event return periods and review of existing ground conditions showed that the risk of death to an occupant of the building was unacceptably high, and that the risk of damage to the building would also have been considered too high.

Another site in Nelson was also affected by the same rainfall event, which caused a cut slope to fail and debris impacted on the rear of the house. Site investigations found that a clay layer within gravels, exposed by cutting of the slope, had acted as an aquitard, and formed a sliding surface. QRA showed that removing the toe of the natural slope considerably increased the risk of landsliding, and therefore the risk to the occupants and damage to the building.

Lastly the study looked into whether QRA could be a useful tool in assessing "imminent loss" claims. The Earthquake Commission Act (1993) allows the Commission to consider whether a property is at risk of "imminent loss" from landsliding, and to fund remedial works, rather than to wait until damage occurs. In order to assess such claims a geotechnical practitioner has to try to predict the timing of a future landslip event.

While it is not possible to predict the exact timing of a landslip event, it was found that QRA provided a useful framework to better understand the vulnerability of a site to landslip damage as well as the risks and consequences involved in failure. QRA can also be used to assess the expected costs of remedial works versus the cost of repairing damage at some future date.

ABSTRACT

Quantitative risk assessment (QRA) procedures are well established but their application to the evaluation slope stability has not been widely adopted in New Zealand. This publication reviews QRA procedures and applies QRA to four case studies which are considered typical slope failure events in the New Zealand context and are amenable to analysis. Each case study site and the QRA procedures adopted are described and illustrated. Probability distributions and stability analyses illustrate the procedures. In three cases QRA shows the risk of death to an occupant of a house is unacceptably high by international guidelines, and that while there are no accepted guidelines the risk of damage to a dwelling, would probably be considered by a house owner to be unacceptable. The other case study demonstrated that while commercially the risk may have been tolerated, its economic effects could have been minimised had the developer been appraised of the risk in advance. The study also considered the application of QRA in the assessment of "imminent loss" claims, one of the provisions of the Earthquake Commission Act (1993).

1. INTRODUCTION

1.1 General

Traditionally geotechnical practitioners have relied heavily on experience-based judgement to assess whether the stability of a slope is adequate to allow development, complemented by the use of numerical techniques such as the factor of safety concept to provide reassurance. New techniques that have arisen out of other areas such as the nuclear and hazardous waste industries and dam safety are now available to assist geotechnical practitioners in quantifying the uncertainty associated with the stability of slopes. These techniques use both numerical methods and/or subjective judgement to assist in quantifying the uncertainties, or the risk inherent in any system such as a slope, and are collectively referred to as Quantitative Risk Assessment (QRA).

QRA can be used in the context of slope stability to provide alternative procedures for formalising the process of engineering judgement. QRA consists of two components, namely the assessment of the probability of slope failure and identification of the consequences of failure.

The use of QRA methods is not intended to replace those established and accepted procedures. Rather, it offers the possibility of expressing numerically the risk attached to any particular slope, compared with terms that are currently expressed qualitatively using such phrases as "safe" and "unsafe". It can be used to compare the numerical value of the assessed risk against risk acceptance criteria, and therefore has the potential to be used within the framework of international standards such as the recently published generic AS/NZS 4360: 1999 "Risk Management".

Although procedures are well established, the use of QRA to evaluate slope stability risk is very much an emerging concept overseas and still in its infancy in New Zealand.

1.2 Objectives

In 1999, BRANZ published a review of QRA methods for determining slope stability risk at building sites (Riddolls & Grocott, 1999). This document attempted to bring together the "state of the art" of QRA landslide methods based on a review of international literature.

One of the recommendations arising out of the BRANZ study was that a series of case studies should be carried out to assess the usefulness of QRA techniques, "with emphasis on the probabilistic characterisation of landslide hazard and quantification of the risk". The principle objective of this publication is to present a number of slope stability case studies in terms of QRA methods, to highlight the applicability of its use for evaluating risk acceptability, and to describe its advantages and limitations.

1.3 Scope

The framework of QRA for evaluation of slope stability has been previously reviewed (Riddolls & Grocott, 1999), but is summarised here in Section 2 to allow continuity for the reader within the context of this publication. The framework presented in Section 2 includes a summary of common definitions, which are used as the basis for the QRA case studies presented in Sections 3 to 6, as well as a discussion of the QRA process.

Sections 3 to 6 summarise four New Zealand QRA slope stability case studies. There was considerable difficulty in identifying appropriate case studies for the QRA analyses presented here, which is an acknowledgment by the authors that not all landslides are amenable to this form of analysis. Factors that contributed to the exclusion of sites included overly complicated slope geological models that would not have allowed meaningful QRA analysis, as well as lack of adequate data for analysis. One potential case study had to be eliminated due to litigation associated with insurance claims. The four case studies that are summarised are therefore considered typical of the types of sites most suitable for QRA analysis.

Finally, the report in Section 7 is a review of the "imminent loss" provisions of the existing Earthquake Act 1993. The use of formal QRA analysis was thought to have potential usefulness for geotechnical practitioners as a means of clarifying issues associated with the legal definition of imminent loss, generally taken to mean that property loss or damage is likely within "four seasons" (i.e., one year).

2. FRAMEWORK FOR QUANTITATIVE RISK ASSESSMENT

2.1 Definitions

Risk assessment in general and landslide risk assessment in particular has resulted in a plethora of different definitions by a range of different authors, resulting in confusion, misinterpretation and misuse of terminology. "Risk" means different things to different people.

Definitions provided by a recent publication (Fell & Hartford, 1997) which have been promulgated for use with landslide QRA appear to be gaining wider acceptance, and are used in this document. The original sources are also referenced:

Acceptable risk: a risk for which, for the purposes of life or work, we are prepared to take pretty well as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks (Health and Safety Executive, 1992; ANCOLD, 1994).

Hazard: a condition with the potential for causing an undesirable consequence. Descriptions of landslide hazard, particularly for zoning purposes, should include the volume or area of the landslide, and the probability of its occurrence. There may also be value in describing the velocity, and the differential velocity of the landslide (Varnes, 1984; Fell, 1994; United Nations, 1991).

Elements at risk (E): meaning the population, buildings and engineering works, economic activities, public services utilities and infrastructure in the area potentially affected by landslides (Varnes, 1984; Fell, 1994; United Nations, 1991).

Hazard identification: the recognition that a hazard exists and the definition of its characteristics (Canadian Standards Association, 1991).

Individual risk: the risk to any identifiable (named) individual who lives within the zone impacted by the slope failure; or who follows a particular pattern of life that might subject him or her to consequences of slope failure (Fell & Hartford, 1997).

Probability (P): the likelihood of a specific outcome, measured by the ratio of specific 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 an outcome is certain (Standards Australia and Standards New Zealand, 1995).

Risk: a measure of the probability and severity of an adverse effect to health, property or the environment (Canadian Standards Association 1991).

Risk is often estimated by the mathematical expectation of the consequences of an adverse event occurring (i.e. the product of "probability x consequences"). However, a more general interpretation of risk involves probability and consequences in a non-product form. This presentation is sometimes useful in that a spectrum of consequences, with each magnitude having its own corresponding occurrence, is outlined. For landslides, both representations are useful, the latter being used initially with the intangible consequences identified with subsequent expected value calculations for those consequences where "risk costs" can be estimated and compared with quantitative decision criteria (Canadian Standards Association 1991).

Risk analysis: the use of available information to estimate the risk to individuals or populations, property or the environment, from hazards. Risk analyses generally contain the following steps: scope definition - hazard definition - risk estimation (Canadian Standards Association 1991).

Risk assessment: the process of risk analysis and risk evaluation (Canadian Standards Association 1991).

Risk control: the process of decision-making for managing risk, and the implementation, enforcement, and re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input (Canadian Standards Association 1991).

Risk estimation: the process used to produce a measure of the level of health, property, or environmental risk being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration (Canadian Standards Association 1991).

Risk evaluation: the stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing risks (Canadian Standards Association 1991).

Risk management: the complete process of risk analysis and risk evaluation (Fell & Hartford, 1997)

Safe: free from harm or risk (Standards Australia and Standards New Zealand, 1995).

Safe slope: one which is sufficiently stable as not to impose unacceptable risks to the public by its presence (adapted from US Bureau of Reclamation, 1989).

Societal risk: the risk 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 (Fell & Hartford, 1997).

Specific risk (R_s): probability x vulnerability for a given element (Varnes, 1984; Fell, 1994; United Nations, 1991):

$$R_s = P \times V$$

System: a bounded, physical entity that achieves in its environment a defined objective through interaction of its parts. This definition implies that:

(a) the system is identifiable
(b) the system is made up of interacting parts or subsystems
(c) all of the parts are identifiable, and
(d) the boundary of the system can be identified (Canadian Standards Association, 1991)

For a risk-based safety evaluation, the system will generally comprise two sub-systems, the potentially unstable slope and anything impacted by partial or complete failure of the slope. Some hazards are internal to the system (internal weaknesses); others, such as extreme rainfall and earthquakes, are external hazards which cross the boundary of the system.

Tolerable risk: a risk that we are 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 (Fell & Hartford, 1997).

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not being properly controlled.

Total risk (R_t): the expected number of lives lost, persons injured, damage to property and disruption of economic activity. It is the product of specific risk (Rs) and elements at risk (E) over all landslides and potential landslides in the study area:

$$R_t = \sum (E \times R_s)$$
$$= \sum (ExPxV) \text{ (Fell \& Hartford, 1997)}$$

Vulnerability: the degree of loss to a given element or set of elements within the study area affected by the landslide(s). It is expressed on a scale of 0 (no damage) to 1 (total loss) (Varnes, 1984; Fell, 1994).

For a 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 the person(s) affected by the landslide.

2.2 The Quantitative Risk Assessment Process

The QRA process is discussed in detail in a number of recent publications (Riddolls & Grocott, 1999; AGS, 2000), and is summarised briefly here to allow reader continuity. The QRA process includes risk analysis, risk assessment and risk management, as illustrated by the flow chart in Figure 1, which is based on work presented by Fell & Hartford (1997) and the IUGS Working Group on Landslides (1997).

2.2.1 Risk Analysis

Risk analysis involves identification of the landslide hazard and assessment of its frequency of occurrence, as well as consideration of the consequences of landsliding if persons and/or property are impacted by failure. Firstly a thorough assessment of the types, characteristics and frequency of landslides in a given study area is carried out in order to identify the hazard. Where the frequency of landslides cannot be determined directly from field evidence, or where engineered slopes are involved, analytical or numerical techniques are required to evaluate the probability of failure.

The frequency of landsliding can be expressed in one or more of the following ways:
- The annual frequency of occurrence within the designated study area
- The probability of failure, and
- The driving forces exceeding the resisting forces expressed in terms of probability or reliability expressed as an annual frequency. This treats one or more of the input parameters for conventional deterministic slope stability analysis (such as shear strength, bulk density, groundwater pressure, earthquake acceleration etc.) as probability distributions.

Secondly consequence analysis is carried out to establish the elements at risk (persons and/or property) which could be impacted by any failure and to determine their vulnerability in the event of failure. Consequence analysis is therefore an assessment of the conditional probability of the consequences occurring given failure to have occurred. The elements at risk are relatively easy to identify in terms of the population and the value of the property potentially exposed to the landslide hazard. Assessment of the probability of a consequence occurring to the elements at risk is much more difficult to quantify and traditionally has been based mainly on subjective judgement. Consequence analysis requires consideration of:

- where will the landslide occur and what will be the probability of the element at risk being impacted by the failure, termed the spatial probability, $P(S|F)$.

- what is the probability of the element at risk being present at the time of impact, termed the temporal impact, P (T|S). Normally this conditional probability would not apply to property due to it being fixed in space, other than to moving vehicles on transportation routes, where the proportion of time the vehicle is exposed to the hazard would need to be allowed for.
- what is the probability of loss of life, or what proportion of the property value will be damaged, given impact by the failure, and given as V (L|T) for the vulnerability of an individual and V (P|T) for the proportion of the property value lost.

The above conditional probabilities P (S|F), P (T|S), V(L|T) and V(P|T) are all expressed as values from 0 to 1. By assessing consequences in this manner, allowance can be made for the location of an element at risk in relation to the landslide and the length of time of exposure to the hazard. Wong et al., 1997 presented values for the conditional probability functions V (L|T) and V (P|T) for death from landslide impact based on historical data from Hong Kong (Table 1).

Table 1: Summary of Hong Kong vulnerability ranges and recommended values for death from landslide debris

(from Wong et al., 1997)

Vulnerability of Person in Open Space			
Case	Range in Data	Recommended Value	Comments
If struck by a rockfall	0.1 - 0.7	0.5	May be injured but unlikely to cause death
If buried by debris	0.8 - 1.0	1.0	Death by asphyxia
If not buried	0.1 - 0.5	0.1	High chance of survival
Vulnerability of Person in a Vehicle			
Case	Range in Data	Recommended Value	Comments
If the vehicle is buried/crushed	0.9 -1.0	1.0	Death is almost certain
If the vehicle is damaged only	0 - 0.3	0.3	High chance of survival
Vulnerability of Person in a Building			
Case	Range in Data	Recommended Value	Comments
If the building collapses	0.9 - 1.0	1.0	Death is almost certain
If the building is inundated with debris and the person buried	0.8 - 1.0	1.0	Death is highly likely
If the building is inundated with debris and the person not buried	0 - 0.5	0.2	High chance of survival
If the debris strikes the building only	0 - 0.1	0.05	Virtually no danger

Consequence analysis (P(C|F)) is the product of the above conditional probabilities in the form of:

(1) P(C|F) = P(S|F) x P(T|S) x V(L|T), for people

and:

(2) P(C|F) = P(S|F) x V(P|T) x value, for fixed property

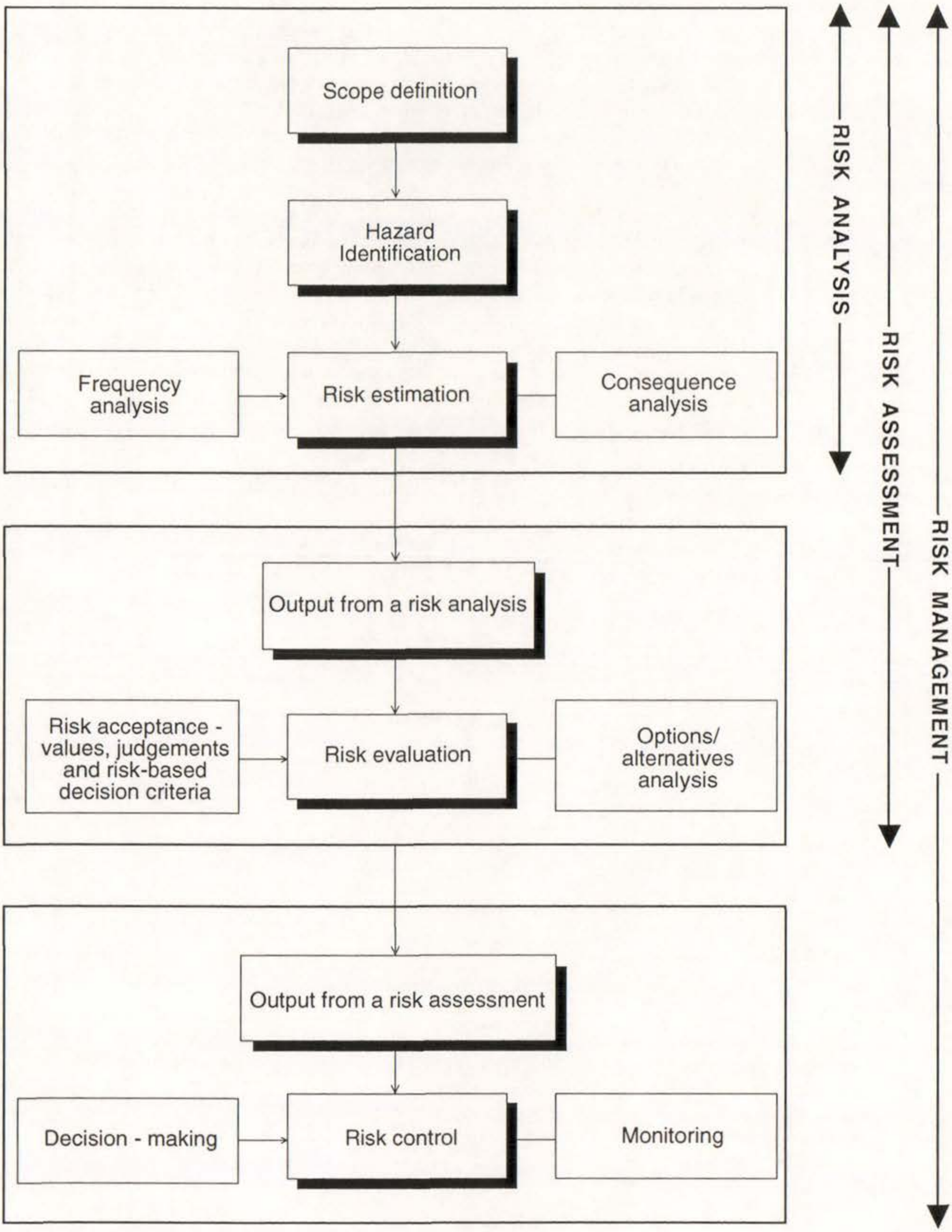


Figure 1: Quantitative Risk Assessment (based on Fell & Hartford 1997; IUGS, 1997)

In assessing the consequences, allowance should be taken of the population density, location of any facility on the slope, degree of protection offered to persons by the type of facility they are housed in, and the degree of warning available.

Consequence analysis requires the scale (i.e. volume), travel distance and velocity of failure to be taken account of. The travel distance and velocity of debris depend critically on the scale and mechanisms of failure as well as the mobility of debris. The extent of the accumulation zone of the failure material at the slope toe and the velocity of the failure material are collectively an indication of the relative damage potential of landslide failure (Wong et al., 1997).

Risk analysis is the product of hazard and consequence analysis, and the output is a mathematical expression of risk given by the general term risk estimation. Risk estimation can be expressed in a number of ways such as the cost to save a life (not to be confused with the value placed on life), the probability of life loss (or injury), cost of damage, or the extent of environmental impact.

The calculation of risk is essentially a mathematical manipulation of the probability of failure, P(F), the elements at risk, and the consequences of failure. Numerically, this can be expressed as:

(3) $\text{Risk} = P(F) \times P(C|F)$
where: $P(F)$ = probability of failure
and: $P(C|F)$ = the conditional probability of a consequence occurring given failure has occurred.

Based on the definitions of the conditional probability, $P(C|F)$, provided by formulas 1 and 2, risk can be expressed as (Figure 3):

(4) $\text{Risk} = P(F) \times P(S|F) \times P(T|S) \times V(L|T)$, for people
and:
(5) $\text{Risk} = P(F) \times P(S|F) \times V(P|T) \times \text{value}$, for fixed property

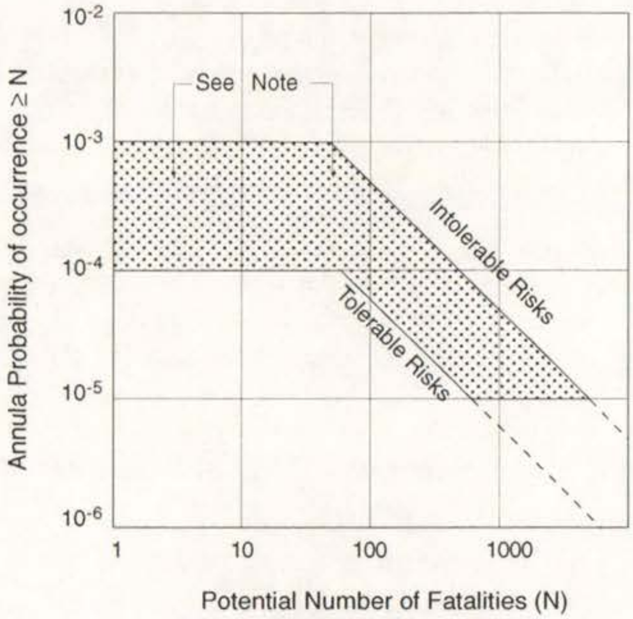
2.2.2 Risk Assessment

Risk assessment is risk analysis considered together with risk evaluation (Figure 1). Risk evaluation requires the calculated risk value (formulas 4 and 5) to be compared against risk-based acceptance criteria for the slope under study, in order to determine the importance of the numerical value. This could include comparison with levels of acceptable death for other activities, or the economic, social and environmental consequences were failure to occur. Typically, the decision as to whether to accept the calculated risk is made by the client, owner or regulators, rather than their technical advisers, whose role is primarily to determine the risk.

The criteria in Figure 2 have been compiled on the basis of a wide range of data and we suggest that they could be used by New Zealand practitioners as a guideline for acceptable risk. The "intolerable risk" range illustrated in Figure 2 is a level in which the risk is viewed as unjustified due to the benefits of accepting the risk being simply not high enough. Conversely, the "tolerable risk" range on Figure 2 is viewed as the level at which the risk is considered to be so small that it is "de minimis" or too small to be worth dealing with or to be held responsible for.

The shaded area on Figure 2 is representative of a risk which is undertaken or accepted only if a benefit is desired. This requires the risk to be reduced to a level when most (but not all) of the public are satisfied (Health and Safety Executive, 1988), considered to be as low as reasonably practicable (ALARP).

Figure 2: Proposed guidelines for assessing risks to life from naturally occurring slope hazards (from Morgan, 1997)



Note: Risks should be reduced in accordance with the ALARP principle with emphasis on those risks falling within the shaded area. The expected annual fatalities should not exceed 5 x 10^{-3} without justification.

Fell & Hartford (1997) provide suggested tolerable risk criteria for the case of the specific identifiable individual at risk from landsliding involving engineered slopes (Table 2):

Table 2
Possible Tolerable Individual Risk Criteria for Landsliding
(for engineered slopes) (from Fell & Hartford, 1997)

Situation	Tolerable risk for loss of life
Existing slopes	10^{-4} person most at risk 10^{-6} average of persons at risk
New slopes	10^{-5} person most at risk 10^{-6} average of persons at risk

Fell & Hartford (1997) consider that the situation for natural slopes affected by landsliding is less clear, but that the general public will tolerate much higher individual risks from natural slopes (compared with engineered slopes), possibly of the order of 10^{-3}.

2.2.3 Risk Management

Risk management, is risk analysis and risk assessment considered together with risk control. Risk control involves the evaluation of options for risk treatment including risk mitigation, risk acceptance, and risk avoidance.

The four case histories presented here are discussed only in the context of risk analysis and risk assessment. The final stage of the QRA process, risk control via the process of risk management, has not formed part of the study as options for treatment are well established and no benefits are achieved by repeating them in this publication.

2.3 Parameter Uncertainty

One of the main limitations of QRA analysis is that almost inevitably there is some uncertainty attached to the various input parameters. This statement is particularly true in relation to the assessment of the frequency of landsliding where considerable uncertainty is attached to parameters such as the annual frequency of occurrence and/or soil strength parameters.

In the context of this research study, parameter uncertainty includes any parameter having a potential influence on the estimated risk value such as the probability of failure, spatial probability, temporal probability and vulnerability of the element at risk. Parameter uncertainty can be accounted for in the risk estimation using Monte Carlo simulation. Monte Carlo simulation is incorporated into this research programme using @RISK Version 3.5e software in conjunction with Microsoft Excel. Analysis inputs about which there is significant uncertainty are specified in terms of probability distributions. There are many different types of distribution that could be used. Triangular distributions have been used (defined by minmising most likely and maximum values) in this instance as they are easily understood and the input data are often insufficiently precise to warrant the use of more sophisticated distributions.

Monte Carlo simulation involves repeating the risk estimation many times, each time using input values selected from their respective probability distributions. As the analysis is repeated the outcomes themselves build up probability distributions. This technique allows the uncertainty in the calculated risk to be considered during the risk estimation stage of the QRA.

3. CASE STUDY A - TAURANGA

3.1 Background

The case study involves a rural residential subdivision lot comprising 24 "life style" lots contained in 10.6227 hectares on the slopes of a dormant volcanic dome. On 7 December 1983, following a period of prolonged rainfall, a landslide was activated on an immediately adjoining property upslope of the subdivision (Figures A1 and A2). The failure overran parts of Lot 14 resulting in destruction of the detached garage and minor damage to the dwelling (Figure A3). Slip material broke through an exterior wall to the dwelling and entered a bedroom where the occupant was asleep. The occupant suffered no physical injury.

Subsequently, a number of the property owners initiated civil proceedings against the territorial authority, the land developers and their engineering consultants on the basis that the land was of inadequate long-term stability. The litigants' evidence forms the basis of much of the background to this QRA analysis.

3.2 Site Description

The subdivision is contained on the lower northeasterly-facing slopes of a dormant volcanic dome that rises 80 m in elevation from valley floor to summit crest (Figure A4). Slopes within the subdivision are inclined at 15° - 30°, steepening up to 45° at the slope crest above the subdivision (Figure A5).

The subdivision topography is characterised by hummocky, undulating and irregularly contoured slopes, which reflects an association with past slope instability.

3.3 Investigations

Following the December 1983 landslide on Lot 14, the territorial authority in which the subdivision is located initiated an over view study of land stability of the local area based mainly on aerial photograph interpretation, from which land use management plans were prepared and subsequently adopted by Council (Tonkin & Taylor, 1984).

Subsequently, expert witnesses acting for the various litigants carried out geotechnical investigations in support of the litigation proceedings (Grocott & Olsen, 1992):

- Data review, including subdivisional plans, topographic plans (1:2000 scale), unpublished reports on local slope instability, and geological information relevant to the local area including publications and unpublished university records
- Interpretation of aerial photo stereo pairs of various dates
- Walk over inspections and engineering geological mapping (Figures A4 and A5), and
- Subsurface investigations including engineering geological logging of 19 test pits and cut slope road batters together with Pilcon shear vane testing of in situ materials.

3.4 Geological Setting

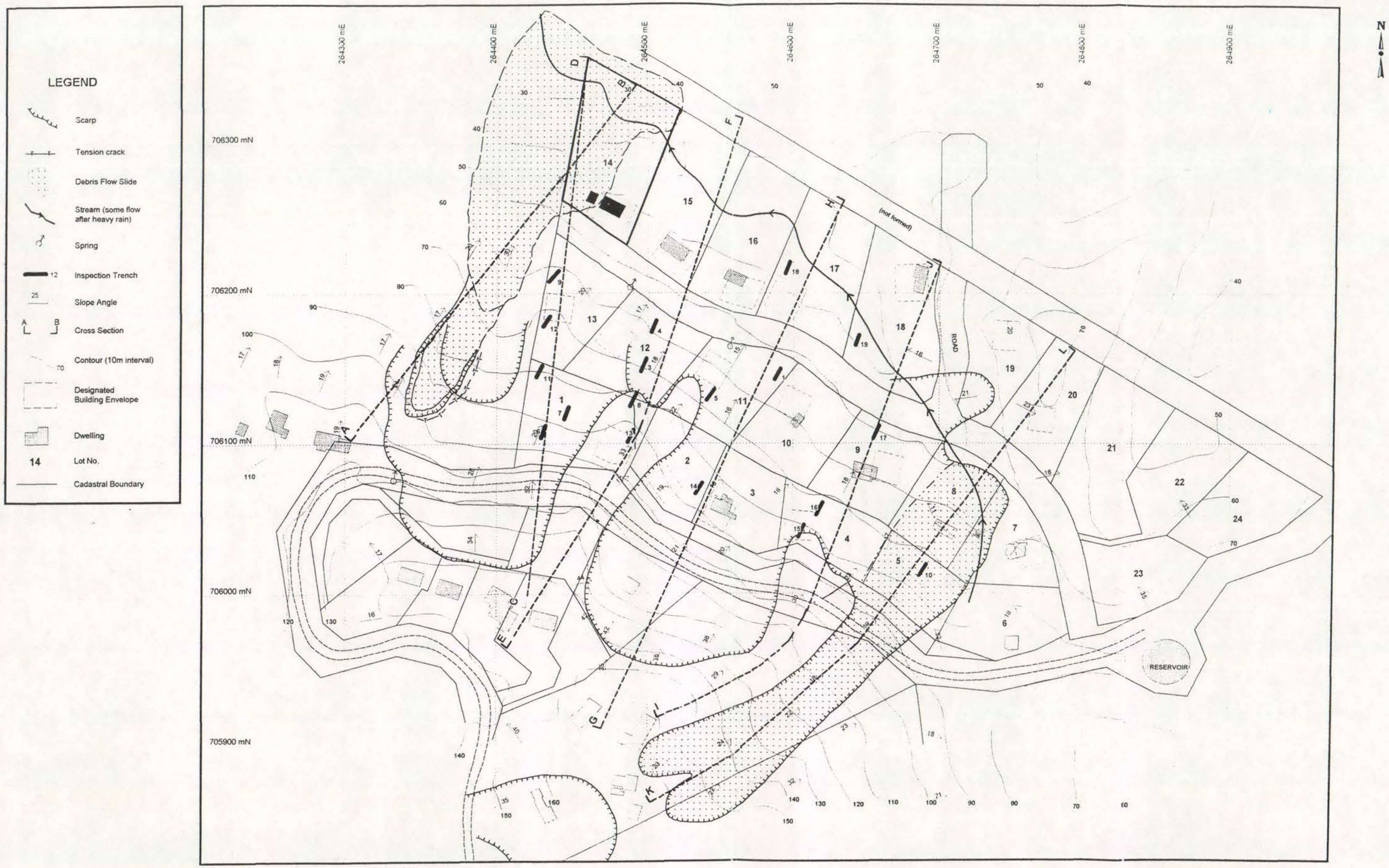
The volcanic dome containing the subdivision is one of a number of similar domes of rhyolitic composition emplaced throughout the Bay of Plenty region approximately 4 million years ago (Healy et. al., 1964). The dome is steep-sided and, apart from the nearly flat summit, forms a highly irregular surface on which younger volcanic materials have been deposited.

Throughout the last 1 million years of the late Quaternary period, the volcanic dome landscape has been modified by erosion and weathering and successively blanketed by numerous rhyolitic ashes erupted mainly from the Rotorua District (Nairn, 1972; Pullar & Birrell, 1973). In the Tauranga area

Figure A1: Landslide of 7th December 1983, damaged house on right of photo (view looking southwest)

Figure A2: View to northeast from top of landslide. Dashed line shows the boundary between the lost ground and the landslide debris

Figure A3: View to east showing damage to house and landslide debris



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SCALE	1:2,500	FILE plan_bw wor A3	PROJECT No 436	REPORT No — FIGURE No A4

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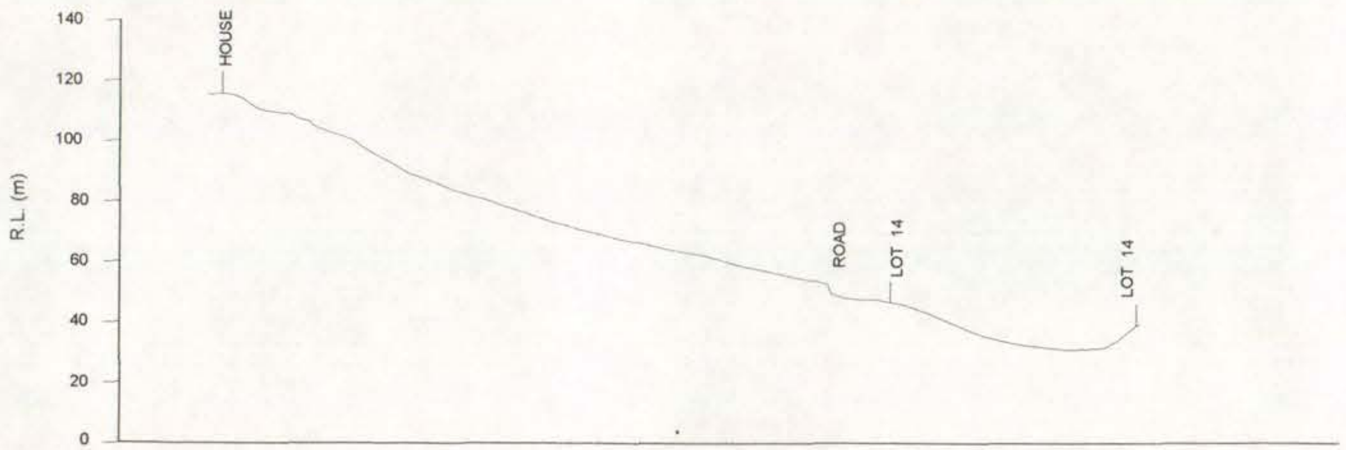

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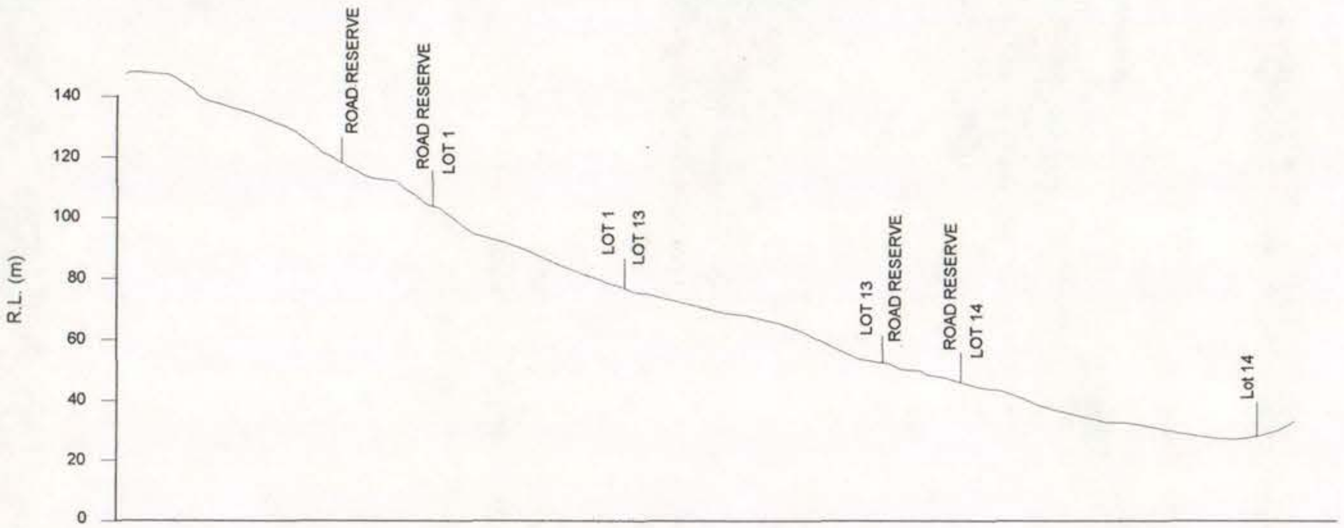

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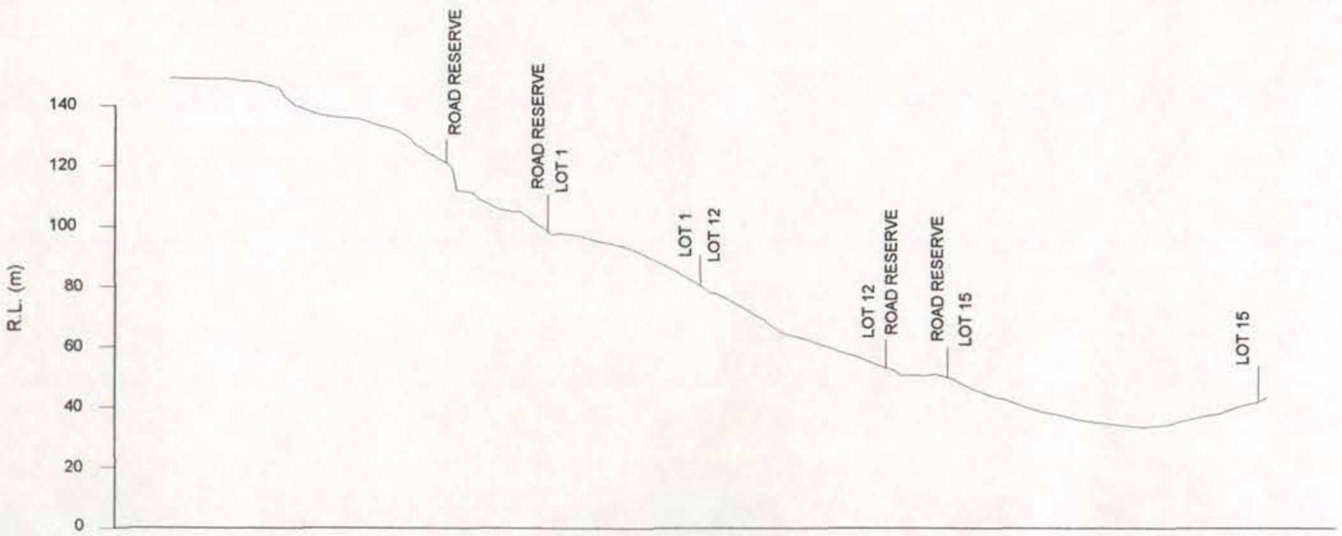

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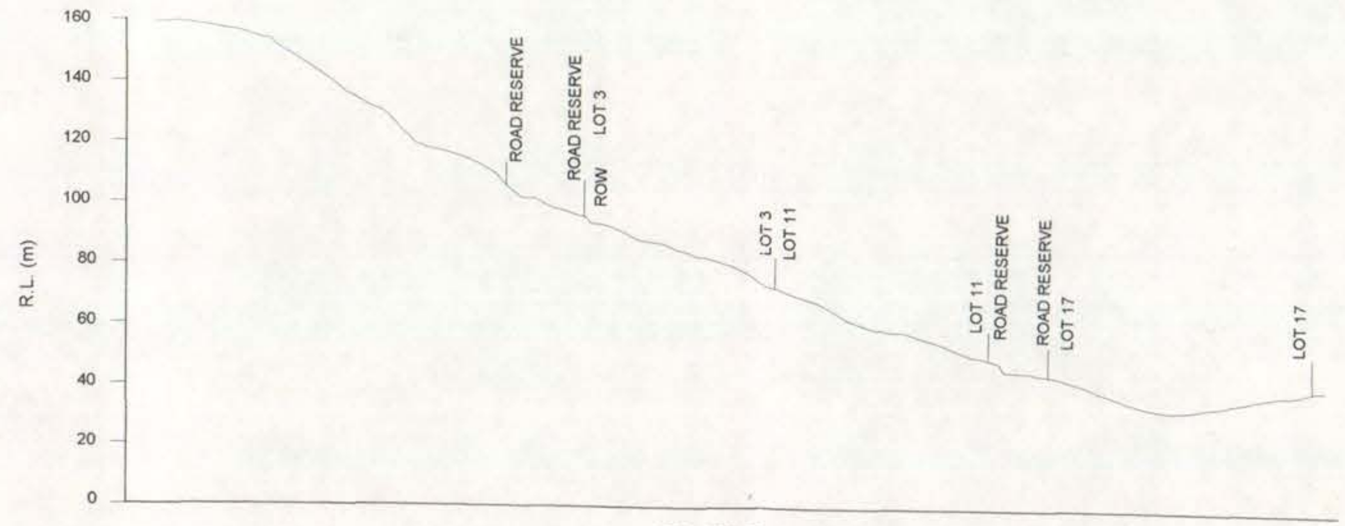

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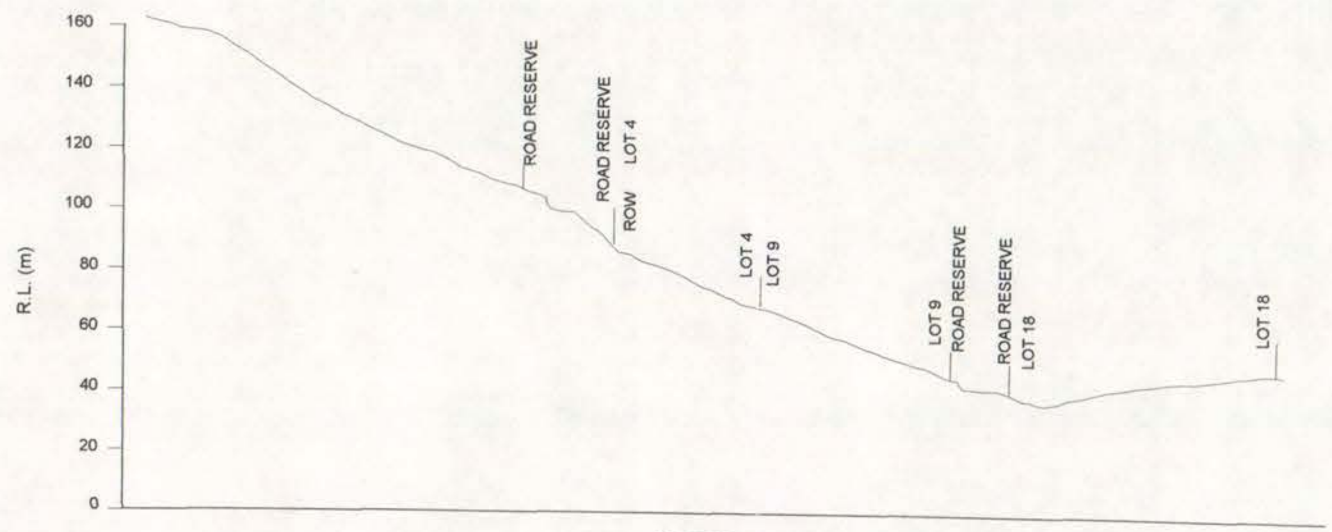

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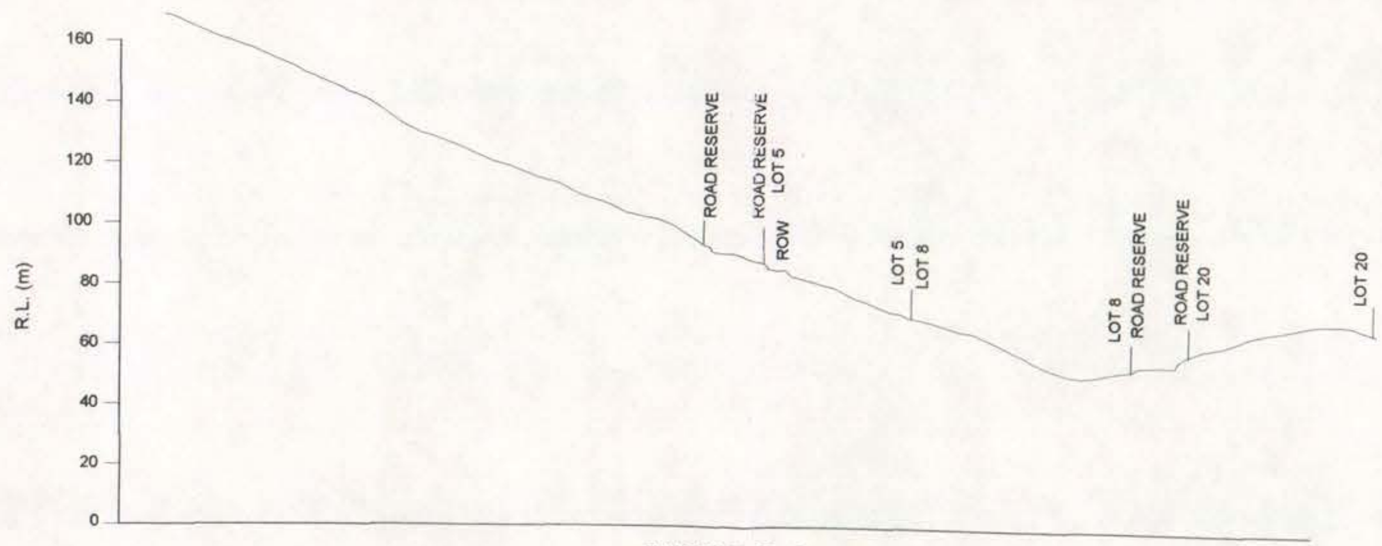

PROFILE K - L

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metres

CLIENT	EQC / BRANZ		PROJECT	CALIBRATION STUDIES FOR QUANTITATIVE RISK ASSESSMENT
DRAWN	MCD	DATE October 2000	TITLE	CASE STUDY A: TAURANGA SUBDIVISION; TYPICAL CROSS SECTIONS
CHECKED	GGG	DATE October 2000		
SCALE	1:2,500	FILE Profiles_bw.wor A3	PROJECT No 436	REPORT No — FIGURE No A5

generally, the near-surface ashes are informally subdivided into two main groups on the basis of distinct lithological differences and these are referred to as the younger and older ash sequences respectively. The older ash sequence, deposited mainly between 75 000 - 300 000 years, is characterised by mainly fine grained lithologies and typically deep residual weathering. By contrast, the overlying younger ashes are lithologically dominated by silt and sand, are less weathered, and include a number of distinctive marker horizons such as the 42 000 years old Rotoehu Ash.

The complete sequence of volcanic ashes is not preserved everywhere on the dome landscape due to erosion processes on the steep side slopes, but typically the younger and older ashes form a soil mantle several metres in thickness overlying volcanic bedrock. On steep slopes where mass movement has occurred, there is colluvium derived from the ashes.

The volcanic ashes and the colluvial materials derived from them are characterised by soil properties that are complex and highly variable. The soil properties that determine the behaviour of the ashes include high natural moisture contents, extreme sensitivity and variable strength and permeability characteristics, the result of which is high susceptibility to mass movement on steep ground.

3.5 Stability Characteristics

3.5.1 Landslide Risk Zoning

A study commissioned by the territorial authority following the December 1983 landslide found that the north-east facing hillslopes of the general area (including the case study subdivision) are characterised by numerous landslips, and field evidence indicates that the debris from these landslips still covers the mid to lower slope areas (Tonkin & Taylor, 1984). The study resulted in a three-tiered zonation of the slope stability risk :

- Area A: moderate to severe risk
- Area B: slight to moderate risk
- Area C: very slight to slight risk

Based upon the steepness of the slopes; the colluvial debris; and the evidence of recent mass movement, the general area was judged to have a moderate to severe risk of slope instability, with the case study subdivision being assigned as Area A,the highest risk area.

Development conditions recommended as being applicable for Area A are "that no building, subdivision or other development, including cutting, filling, removal of vegetation, disposal of stormwater or domestic waste water into or over the area delineated, be permitted unless adequate provision is to be made to prevent erosion or landslippage or unless sufficient investigation has been undertaken to satisfy Council that a building sited in a specific location and subject to specific development criteria is unlikely to be damaged by landslippage" (Tonkin & Taylor, 1984).

3.5.2 Ground and Groundwater Conditions

i) Surface Evidence

Geomorphic evidence (Tonkin & Taylor, 1984; Grocott, 1988; Taylor, 1988) shows the subdivision and higher slopes up to the summit of the volcanic dome comprise a number of different landforms. These include ridge and gully topography on the higher flanks of the volcanic dome which extend into the subdivision, while on lower foot slopes of the subdivision, the land surface is irregular and undulating suggesting mass movement has resulted in the accumulation of colluvium from the higher slopes.

Aerial photos show evidence of landslips in the same gully system but pre-dating the December 1983 landslide (indicating the December 1983 was the was the most recent expression of ongoing landslide activity at this location) (Taylor, 1988), and slips above the subdivision (south of Lot 5), probably related (on the basis of the dates of different aerial photographs) to rainstorm events in June 1961 and May 1962 (Grocott, 1988).

A major landslide feature visible on photographs extends from Lots 5 and 8 to the summit of the volcanic dome above the subdivision (Figure A4). This feature is of considerable proportions (300 m long x 35 - 60 m wide), the boundaries of which are recognisable on the basis of its elevated surface relief compared with the adjoining land and a bulged toe. Its elongated plan profile and toe bulging suggest the mode of failure has been as a major earthflow. The age of this feature is unknown but clearly pre-dates the earliest aerial photographs (i.e. pre-dates 1946).

ii) Precedent and Anecdotal Evidence

The December 1983 storm event that initiated the landslide affecting Lot 14 was also responsible for activation of landslides in an adjoining catchment 500 m southeast of the subject subdivision. A landslide was also initiated during a January 1986 storm event in the same catchment southeast of the subdivision. There is also anecdotal evidence of at least three landslides on slopes near to the subdivision (from a local resident), the timing of which are uncertain but are believed to have occurred in about the 1970's.

iii) Subsurface Evidence

Test pits (Section 3.3) were located on known landslides in the subdivision, in gullies where geomorphic evidence was suggestive of previous colluvium development, and on ridge lines where more stable slope conditions were indicated (Figure A4). The test pits revealed a complex and variable range of ground conditions, generally consistent with surface geomorphology. .

Groundwater was encountered in 6 of the 19 test pits at depths ranging from 1.5 to 5 m below ground surface. In most trenches groundwater was confined to a perched seepage path, or less commonly multiple seepage paths, rather than there being a true unconfined groundwater table. Typically the seepage paths are horizons of higher permeability on the basis of their grading characteristics than the surrounding materials. There was no indication of any interconnection of groundwater between adjacent test pits, suggesting that seepage paths are discontinuous and variably distributed throughout the colluvium (Grocott, 1988).

3.5.3 Description of Slope Failures

Two distinct landslides types were identified from the various studies carried out (Grocott & Olsen, 1992). Firstly, the landslides generated by the December 1983 and the January 1986 storms (Section 3.5.2) involved rapid downhill movement of surficial (typically 1 - 5 m deep) ash colluvium accompanying periods of heavy and commonly prolonged rainfall. These failures are the predominant type of mass movement, characterised by the rapid displacement of small to moderate volumes (from several hundred up to several thousand cubic metres of displaced ground) of fluid earth type materials, combined with considerable travel path distances.

The December 1983 landslide affecting Lot 14 involved an estimated 3 000 to 5 000 m^3 of rapidly moving earth materials. The velocity of the displaced mass was not estimated at the time of failure (Grocott & Olsen, 1992), but due to its extensive travel path (250 m from the initial point of failure) combined with the deposit area being several tens of metres in elevation above the valley floor on the opposite side of the valley (Figures A2 and A4), a considerable velocity must have been attained. Observations indicate that the failures of the January 1986 storm involved smaller volumes (several hundred cubic metres , but debris velocities and travel paths are judged to have been comparable to the December 1983 landslide case study landslide.

Based on the predominance of fine-grained engineering soils comprising the displaced mass, and the rates of movement, the term earth flow has been used (Varnes, 1978) for instability of this type. The features that characterise earthflows include high water contents sufficient to allow the failed mass to behave in part as a liquid, and high mobility.

The second landslide type, identified on Lots 5 and 8 of the case study subdivision, extends to the summit of the volcanic dome (Section 3.5.2i) is apparently unique in the area due to its considerable dimensions (300 m long x 35 - 60 m wide) and large estimated volume (50 - 60 000 m^3). The landslide form appears largely preserved, suggesting movement may have been more as a relatively slow (compared with the smaller very rapid earth flows discussed above) translational earth slide on the basis of Varnes' (1978) criteria.

3.6 Quantitative Risk Assessment

3.6.1 Objectives, Methods and Limitations

The objectives of the QRA which follows in this section is to retrospectively examine the risk to the principal element at risk (that is, Lot 14 of the case study subdivision) affected by the December 1983 landslide. The following objectives have therefore been established:

- Establish the specific risk of injury and or death to the single most exposed occupant of the dwelling on Lot 14 from the December 1983 landslide, and
- Establish the specific risk of physical damage to the dwelling on Lot 14 from the December 1983 landslide.

There are a large number of other risk objectives that could be addressed in addition to those listed above as part of this risk assessment, including:

- Establish the specific risk of injury and or death to all occupants of the dwelling on Lot 14 from all possible landslide failure modes
- Establish the specific risk of injury and or death to the single most exposed occupant on all lots from all possible landslide failure modes, and
- Establish the risk to all other existing and proposed structures located on the subdivision, and the risk of damage and destruction to other structures such as access roads.

These other risks have not been individually considered here due to the large number of discrete risk calculations that would be required.

A key objective of the studies carried out by the various litigants was to establish the frequency of occurrence of landslide activity (probability of failure) (Grocott & Olsen, 1992), as this influences all other risk calculations. As slope instability at the particular case study site is of a very destructive type, potentially useful volcanic ashes were not preserved and therefore could not be used for dating of specific instability events in this instance. Furthermore, the highly complex nature of the volcanic ground conditions at the site meant that a deterministic approach using the factor of safety method was also of limited usefulness in this instance.

Accordingly, some of the litigants developed alternative approaches based on a review of the rainfall record to assess the frequency of landslide-inducing rainfall events. While such techniques are potentially very useful, they are limited by the quality of the available rainfall data such as the length and frequency of recorded data or the proximity of the gauge to the subject area.

3.6.2 Risk Analysis

i) Probability of Failure

The return period of slip-inducing rainfall events was evaluated with respect to the closest available continuous (13 years) recording rainfall gauge, in this case located 5 km northwest of the case study subdivision.

The analysis involved examination of the 10 highest one, two and three-day rainfall events (Tables A1, A2 and A3), and comparison with known slip-inducing rainfall events (Male, 1988). Of the ten highest recorded rainfalls two events, only those of December 1983 and January 1986 are known to have resulted in landslide activity in the study area generally. For these two storms, the maximum one-day rainfalls rank as the highest and fourth highest ever recorded (Table A1). However, when the preceding ten days rainfall is included, the ranking changes to the highest and second highest rainfalls (Table A1).

Table A1. Ten Highest One Day Rainfalls (mm)

Date	1 Day Rainfall	10 Previous Day Rainfall	Total 11 Days
7 December 1983*	250	91	341
27 May 1974	154	34	188
03 December 1974	127	28	155
04 January 1986*	114	113	227
15 March 1980	112	1	113
09 March 1979	105	20	125
01 August 1979	104	118	222
13 April 1981	104	114	218
19 April 1983	96	9	105
20 January 1981	91	74	165

* Date of slipping events

Table A2. Ten Highest Two Day Rainfalls (mm)

Date	1 Day Rainfall	10 Previous Day Rainfall	Total 12 Days
06-07 December 1983	263	94	357
12-13 April 1981	206	12	218
03-04 January 1986*	180	49	229
27-28 May 1974	161	34	195
04-05 February 1979	138	2	140
14-15 June 1974	137	50	187
22-23 May 1977	135	17	152
09-10 March 1979	122	21	143
23-24 December 1982	119	43	162
14-15 March 1980	113	1	114

* Date of slipping events

Table A3. Ten Highest Three Day Rainfalls (mm)

Date	1 Day Rainfall	10 Previous Day Rainfall	Total 13 Days
05-08 December 1983*	264	94	358
11-13 April 1981	219	0	219
02-04 January 1986*	206	30	236
16-18 April 1974	168	23	191
25-27 May 1974	162	36	198
16-18 February 1984	150	47	197
01-03 December 1974	148	11	159
21-23 May 1977	143	10	153
03-05 February 1979	138	2	140
20-22 March 1979	137	100 (205 in previous 11 days)	237 (342 in previous 14 days)

* Date of slipping events

The two slip-initiating storm events of December 1983 and January 1986 rank as the first and third heaviest rainfalls for both two and three day durations (Tables A2 and A3) (Male, 1988). However, when the preceding 10 days of rainfall is taken account of, the rainfalls rank as the first and second highest two and three-day events ever recorded (Tables A2 and A3). Inspection of Tables A2 and A3 shows that the main difference between the two slip-inducing events (December 1983 and January 1986) and the April 1981 rainfall (being the second highest recorded 2 and 3 day rainstorm) is the low rainfall recorded during the preceding 10 days.

The assessment showed that at least two rainfall factors have therefore played a part in the generation of landslide activity, namely that the catchment soil must undergo a period of days of pre-wetting followed by a significant rainfall event with a threshold of about:

- a 2 day rainfall exceeding 160 mm and a preceding 10 days of rainfall exceeding 40 mm, or
- a 3 day rainfall exceeding 200 mm and a preceding 10 days of rainfall exceeding 25 mm were shown to be significant.

A frequency analysis of the slip-inducing rainfalls showed these to be relatively frequent events with a recurrence interval for the 2 day and 3 day rainfall thresholds of approximately 2.5 and 3.9 years respectively (Table A4). These events are therefore relatively frequent occurrences and occur on average less than four years apart. However, the conditional probability of a mixed process involving some 10 days of pre-wetting means that in practice the average recurrence interval of slip-inducing storms would be longer than this. The 38 years of rainfall records was considered too short to be definitive about the recurrence interval for a conditional probability mixed rainfall process, but suggested it was likely to be of the order of every 7 to 8 years (Male, 1988; Hollands, 1988).

Table A4. Predicted Rainfall Return Periods

Return Period (Years)	2	5	10	20	50
1 Day Rainfall	142 mm	165 mm	183 mm	201 mm	224 mm
2 Day Rainfall	188 mm	219 mm	243 mm	266 mm	297 mm
3 Day Rainfall	209 mm	243 mm	269 mm	295 mm	329 mm

Accordingly, because some uncertainty is attached to the recurrence interval for the mixed rainfall process, the recurrence interval has itself been treated as a probability distribution. This has been treated as a skewed triangular distribution, with values ranging from a minimum of 4 years (P(F) = 0.25) equal to the three day rainfall threshold interval, a most likely value of 8 years (P(F) = 0.125)equal to the suggested recurrence interval for the mixed rainfall process, up to a maximum of 100 years (P(F) = 0.01) based on a judgement of this being the maximum recurrence interval).

ii) Spatial Probability

The spatial probability of a landslide originating from the catchment above Lot 14 and impacting upon the dwelling is required to be estimated. This is a particularly difficult value to assess, as factors such as proximity to landslide source, surface geomorphology as well as the size of the potential landslide will dictate the probability of spatial impact. Considerable subjective judgement is therefore required to be applied to attain a realistic spatial probability value.

Empirical methods of estimating runout distances for various slope angles have been published (Finlay, et al, 1999). These are not generally applicable to debris flows that can travel long distances over very gentle slopes due to the liquid nature of the debris.

The debris from the December 1983 landslide extended over an area approximately four times wider than the actual source area (Figure A4). Also, a small ridge immediately upslope of the dwelling did not afford it complete protection, as might have been expected. Based on these factors, the spatial probability (P(S))of a landslip originating in the catchment area upslope of the dwelling and impacting upon it is judged to be of the order of 0.5 (Table A5). As there is obviously spatial uncertainty associated with landslides of different scales and proximity to the dwelling, the sensitivity of the risk estimation calculation to changes in the spatial probability has been evaluated by treating the spatial probability as a probability distribution, with the values varying by + 0.1 and - 0.1 above the mean value (0.5) (Section 3.6.2 below).

iii) Temporal Probability

In the event that a landslip impacts the dwelling, death or injury is a possibility only if the house is occupied at the time. The temporal probability (P(T)) of the house being occupied could range from 0 (for an absentee occupier) to almost 1 (for an elderly or confined resident). An average value of 0.5 (plus or minus 0.1) would be reasonable in the absence of a more specific knowledge of the occupier's lifestyle.

The temporal probability for the case of damage to the dwelling is obviously 1, as the dwelling is inanimate, and therefore always present.

iv) Vulnerability

The damage to that part of the dwelling that was impacted by the December 1983 landslide was total destruction. Assuming the magnitude of this event to be representative, which is considered reasonable given the destructive type of instability, a vulnerability value of 0.9-1.0 for death of an occupant given impact is considered appropriate. Clearly, the vulnerability (V) of the building to damage in the event that a landslip strikes it is 1.0.

iv) Risk Estimation

The risk may be calculated as the product of the failure probabilities, spatial and temporal probabilities of impact and the vulnerability of the elements at risk. The calculations are presented in Tables A5 and A6. The calculation may be made using "best estimate" values of the input parameters. By specifying a likely range of values for each input parameter a simple Monte Carlo simulation may also be carried out to determine the probability distribution of each of the risks assessed. This approach is demonstrated in Tables A5 and A6, which includes a plot of the probability distribution of each risk.

The calculated risk of death to the most exposed occupant of the dwelling due to landslip ranges from a maximum of 0.09 (one in 11) to 0.0014 (one in 694), with an average of 0.03 (one in 33), indicating that the risk is very sensitive to the assumed probability of landslide failure.

The calculated risk of damage to the dwelling ranges from a maximum of 0.15 (one every 7 years) to 0.004 (once every 250 years), with an average of 0.06 (once every 16 years).

3.7 Risk Assessment

The assessed landslide risk (Section 3.6) indicates that the risk of death to the most exposed individual occupying the dwelling on Lot 14 is much higher than acceptability guidelines (Section 2.2.2). This result holds true even when the probability of failure is assumed to be equivalent to the maximum value, that is, a recurrence interval of 100 years.

There are no acceptability guidelines for property damage. However based on the assessed average interval of damage once every 16 years, most property owners would probably consider such a risk to be unacceptably high.

Table A5: Risk of Death to Occupant due to Landslip

	Min	Most Likely	Max
Probability of Failure P (F)	0.01	0.125	0.25
Spatial Probability P (S)	0.4	0.5	0.6
Temporal Probability, P(T)	0.4	0.5	0.6
Vulnerability V	0.9	0.95	1

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.0014	Minimum frequency of occurrence	694 years
Maximum risk	0.09	Maximum frequency of occurrence	11 years
Average risk	0.03	Average frequency of occurrence	33 years

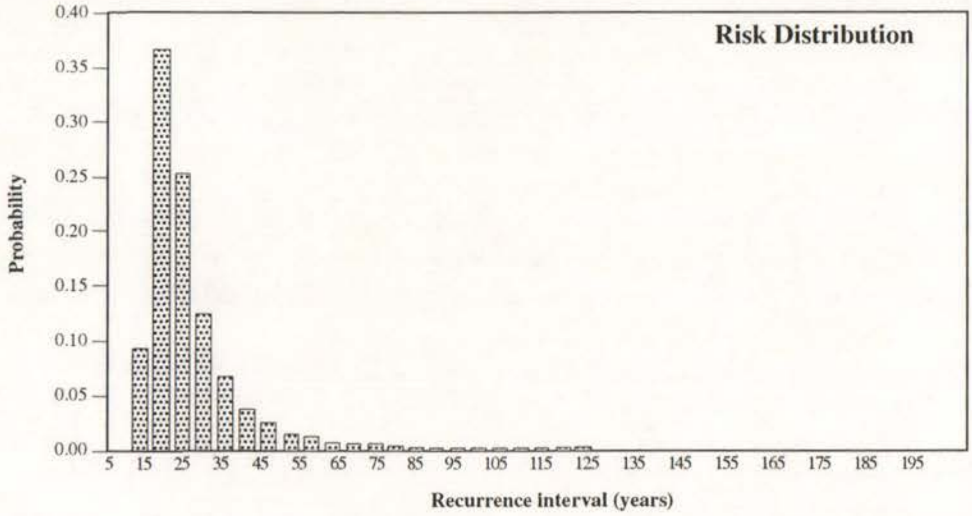
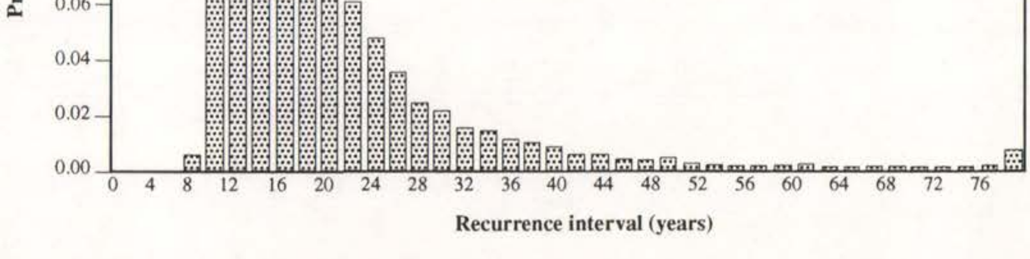


Table A6: Risk of Damage to dwelling due to Landslip

	Min	Most Likely	Max
Probability of Failure P (F)	0.01	0.125	0.25
Spatial Probability P (S)	0.4	0.5	0.6
Temporal Probability, P(T)	1.0	1.0	1.0
Vulnerability V	1.0	1.0	1.0

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.004	Minimum frequency of occurrence	250 years
Maximum risk	0.15	Maximum frequency of occurrence	7 years
Average risk	0.06	Average frequency of occurrence	16 years



4. CASE STUDY B - QUEENSTOWN

4.1 Background

During excavation of building platforms for townhouse construction on a sloping site on 6 September 1994, a substantial slope failure occurred (Figure B1). The landslide significantly affected the development site, encroached up to 8.5 m on to an immediately adjoining cross-slope residential site, and also threatened a residential property upslope.

The proposed development involved construction of two cascade-style townhouses on a stepped excavation towards the foot of the slope. Excavation was mainly carried out by hydraulic digger with some drilling and blasting to loosen some of the harder rock near the base.

4.2 Site Description

The site is on the lower southeast-facing slopes of a hill. Pre-failure topography shows the site to slope at about 20° in the upper part, at ca 58° in the middle ("rock face"), and at about 24° in the lower part. Fill from the immediately upslope property was placed across the upper boundary of the development site, and fill is also inferred to have been placed at the bottom of the site.

At the time of failure, approximately 550 - 600 m^3 of rock had been excavated for building platforms. The failure involved about 2 000 m^3 of mostly rock debris and lesser amounts of residual soil cover. The failure mainly affected the mid slope area of the site, extending from the west side boundary to some 8 m across the east boundary, a horizontal distance of about 45 m.

4.3 Investigations

A detailed surface inspection was made of the slope failure and surrounding terrain, and geological conditions were recorded (Bryant, 1994; Riddolls & Grocott Ltd, 1994). An engineering geological site plan and two cross sections are shown in Figures B2 and B3 respectively.

4.4 Geological Conditions

Rock exposures, resulting from the failure, and also in the vicinity of the site, provide a good indication of subsurface geological conditions. As shown in Figures B1 to B3, schist bedrock occurs close to the surface, overlain by brown-grey colluvium (schist fragments up to 200 mm across within a sandy matrix) up to 2 metres thick in the lower part of the slope, and about 0.5 metres of yellow brown gravelly sand in the upper part.

The schist bedrock is a greenish grey material of high strength, thinly laminated and moderately fissile. The foliation (mineral layering) dips to the southwest at about 20°. The head scarp is formed by a persistent irregular joint plane which strikes across the slope, and dips at 60° in the downslope direction.

The uppermost 1.5 m of schist bedrock at the west end of the head scarp exhibited closely spaced fractures parallel to foliation. The toe failure surface was obscured by rock debris excavated from the building platform, but it is very likely to be formed by a thin clayey crushed seam (foliation shear).

A number of steeply inclined joints striking up/down slope were also observed at the margins of the failure, which may have acted as lateral release surfaces for the block slide.

Slight seepage of groundwater was visible at the site inspection following failure down the head scarp originating from the contact between the surficial deposits and the bedrock, and also at the base of the closely fractured schist at the west end of the scarp. The water source was believed to be from a thin

Figure B1: Oblique view of slope failure

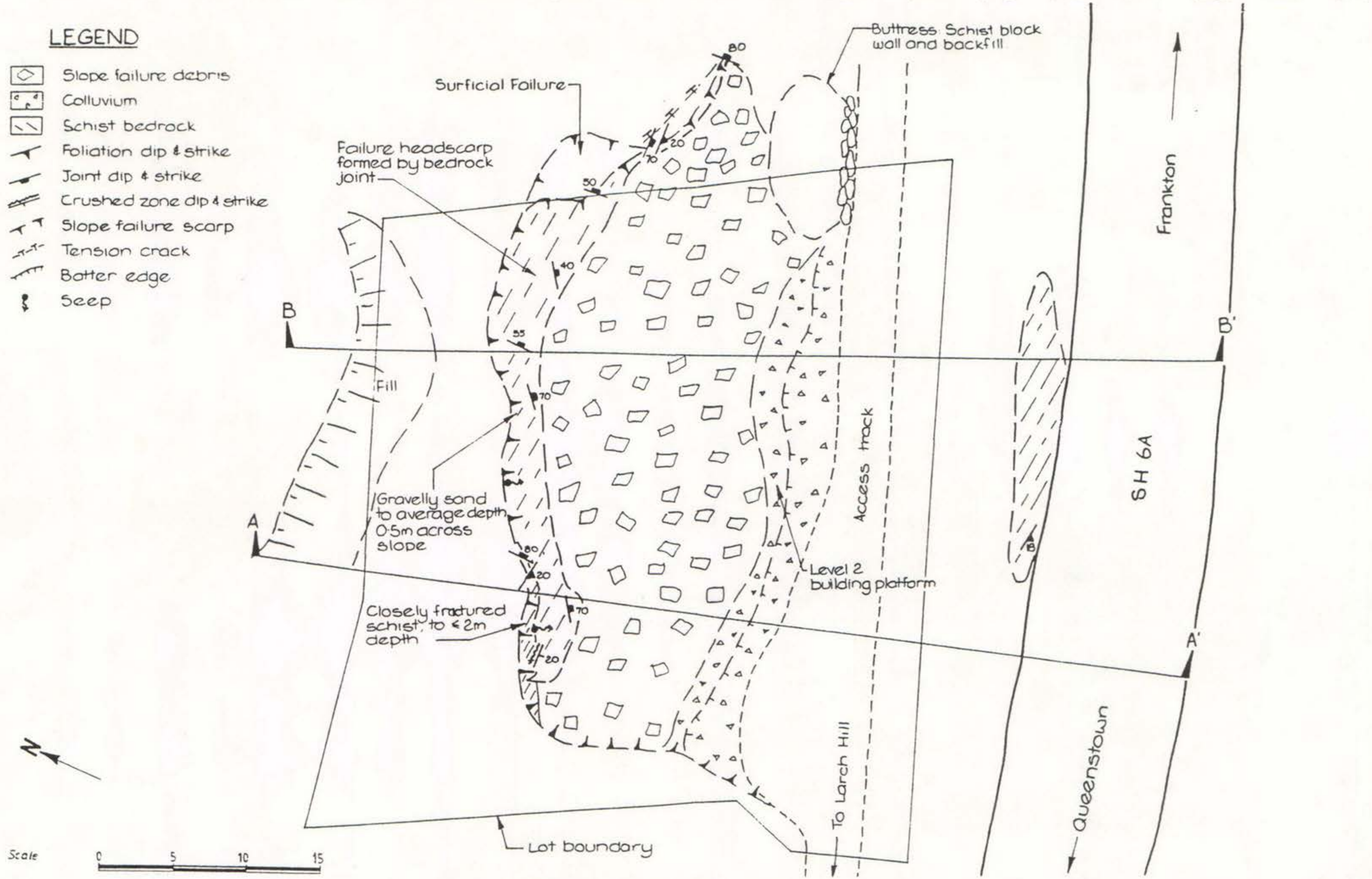


Figure B 2: Geological plan

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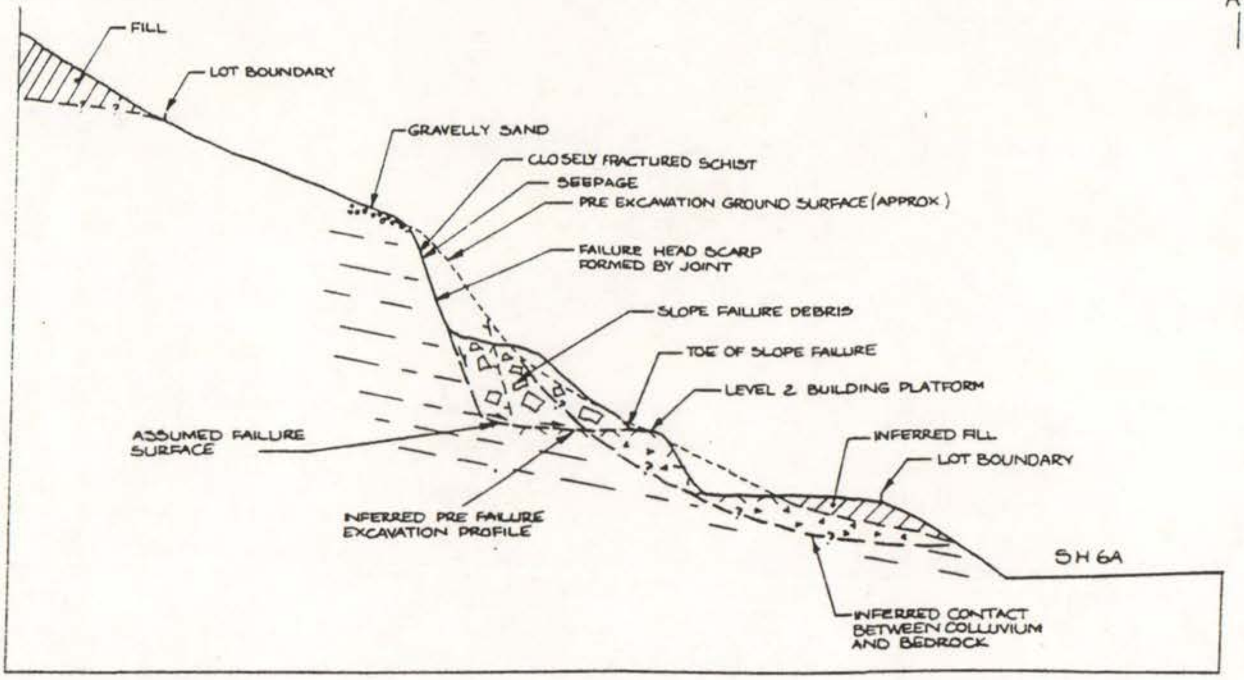

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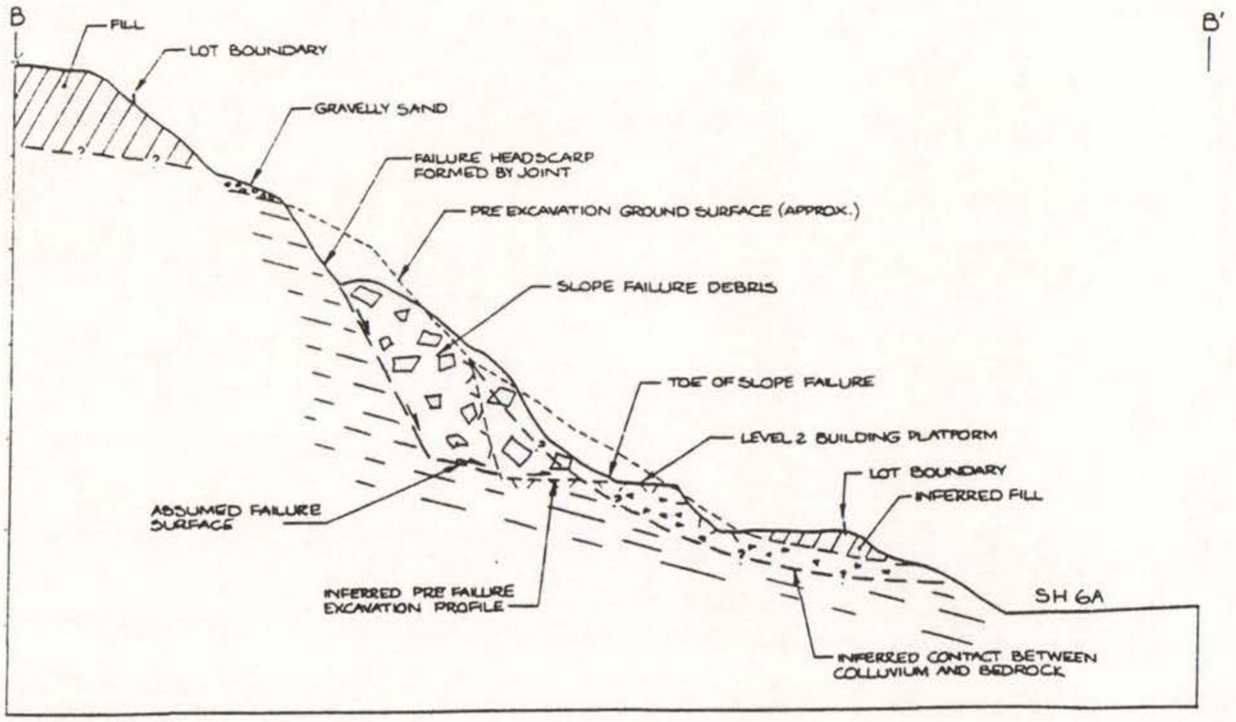


Figure B3: Geological sections A - A', B - B'

water table perched on the interface separating schist bedrock surface from the overlying colluvial soil (Bryant, 1994).

The lowest part of the slope, which is essentially level is inferred to be fill (Figure B3).

4.5 Stability

An inspection of the unaffected parts of the site and the adjacent properties revealed no surface evidence of deep-seated slope instability.

The failure is understood to have occurred just as building platform excavation was nearing completion, which would have resulted in removal of some toe support to the slope (although there is no data available on the ground profile immediately preceding the failure). The failure was not associated with or preceded by any significant rainfall. The volume of debris has been estimated from Figures B2 and B3 to be in the order of 2700 m^3.

The failure of the slope evidently took place as a translational block slide within the schist bedrock, presumably daylighting at the level of excavation (Figure B3).

An engineered buttress fill was subsequently placed against the failure, and the development abandoned. The site is not suitable for residential development in its present state, and will require major stabilisation to restore its development potential.

4.6 Quantitative Risk Assessment

4.6.1 Objectives

The objective of this case study is to determine if quantitative assessment of the risk of landsliding would have proved useful at this site. The risk to be assessed may therefore be defined as:

The element at risk - the site itself

The hazardous event - defect-controlled landslide in schist bedrock

The consequence - loss of the development potential and associated profits

In this case, the occurrence of a landslip inevitably resulted in the consequence being realised (if the magnitude variable is ignored). The temporal and spatial probabilities are 1.0, as is the vulnerability. The risk is therefore equivalent to the probability of landslide failure.

In additional to the commercial risk, the safety of construction workers at this site is also an issue. QRA could be used to provide a means of demonstrating a duty of care on behalf of the developer and designer with respect to health and safety of site workers. These risks have not been addressed in the following example.

4.6.2 Risk Analysis

i) Probability of Failure

As the existence of a failure plane was not known prior to the failure, numerical stability analysis alone is insufficient for an *a priori* determination of failure probability. An alternative approach based on back analysis of the slope model is discussed below.

Back analysis of the actual failure that occurred using two-dimensional limiting equilibrium methods has been carried out. The following assumptions were made.

- Drained slope (i.e. no groundwater pressure acting upon the failure planes)
- The slope failed along pre-existing defects
- Two dimensional analysis

The analysis indicates that shear strength, along the failure plane mobilised at failure, comprises a friction angle of 25° and cohesion of 25 kPa (Figure B4). These values are high when compared to published strength data for rock mass defects in Otago Schist rock (Riddolls & Grocott Ltd). Repeating the analysis using upper bound values (cohesion = 8 kPa, friction angle = 18°) gives a factor of safety of 0.53 (Figure B5). Adopting an arbitrary standard deviation of 3 kPa for cohesion and 5° friction angle, a probability of failure of 99.9 % is indicated. Back analysis shows that had the existence of the failure plane been known limiting equilibrium analysis using published data would have identified the very high probability of slope failure (close to 1.0).

ii) Presence of Failure Plane

It would be reasonable to assume that failure cannot occur through intact schist rock, which has an unconfined compressive strength of typically 100-200 MPa. The existence of the failure plane was not known prior to development of the site. The fact that foliation dipped out of the slope at a shallow angle was known however. It would therefore be reasonable to consider a failure mechanism where a defect parallel to foliation formed part of the failure plane. As foliation dips at a shallower angle than the original slope (20 deg v 35 deg), failure along foliation alone could not occur. A steeper intersecting defect would be required to allow the failure plane to daylight at the head of the slip.

This potential failure mechanism could therefore have been identified in principle prior to development. A quantitative assessment of the likelihood that the required defects would be present may be carried out as follows.

Foliation shear (toe breakout)

Foliation shears develop as a result of tectonic or slope movement along a foliation plane. Based on a general knowledge of the geology of the area it is estimated that a persistent foliation shear could be expected to occur every 20 m within the rock mass. Consideration of the two-dimensional failure geometry (Figure B6) indicates that a foliation shear would have to occur within a 4 metre vertical interval for failure to develop. The probability of this may therefore be expressed as 4/20 = 0.2.

Steeply dipping joint (head release)

One or more joints sets are present within the rock mass. In order for failure to occur, a joint persisting right across the top of the site is required. Joints with orientation between 90/147 and 45/167 (dip/dip direction) are considered to meet this criterion. The locus of poles of a defect within this range is shown on a stereographic projection in Figure B7. In the absence of any site specific information on preferred joint orientation, the probability of a joint having an orientation within this range is calculated as:

$$((90 - 45) / 90) \times ((167 - 147) / 360) = 0.0277$$

On average we assume there would be two joint sets present within the rock mass, and the spacing between persistent joints within each set would be 10 metres. It is further assumed that failure could occur if one of these features was present within 15 m of the toe of the slope prior to failure (Figure B6). The probability of a suitable head release joint being present is therefore estimated as:

$$0.0277 \times 2 \times (15/10) = 0.0831$$

Figure B4: Results of stability analysis assuming typical strength parameters for discontinuities in Otago Schist

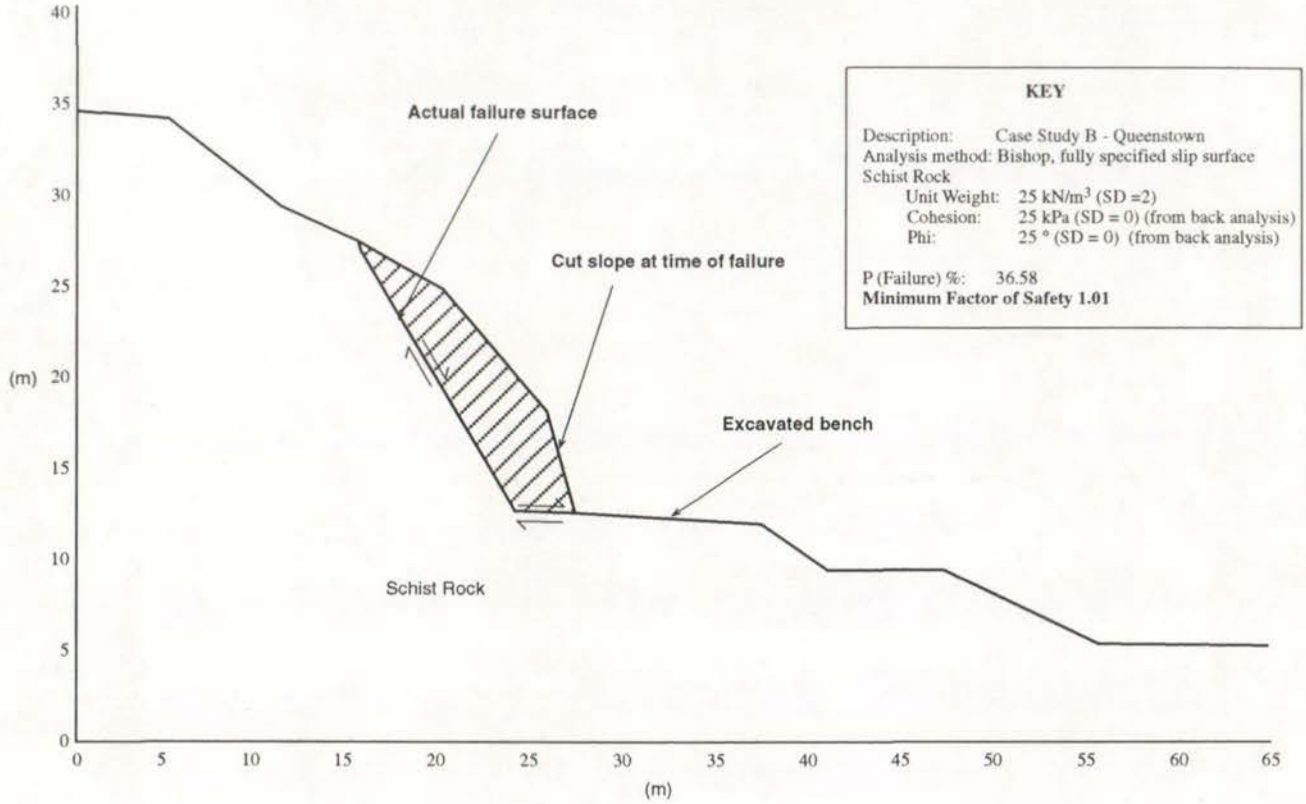


Figure B5: Results of back analysis showing actual strength parameters

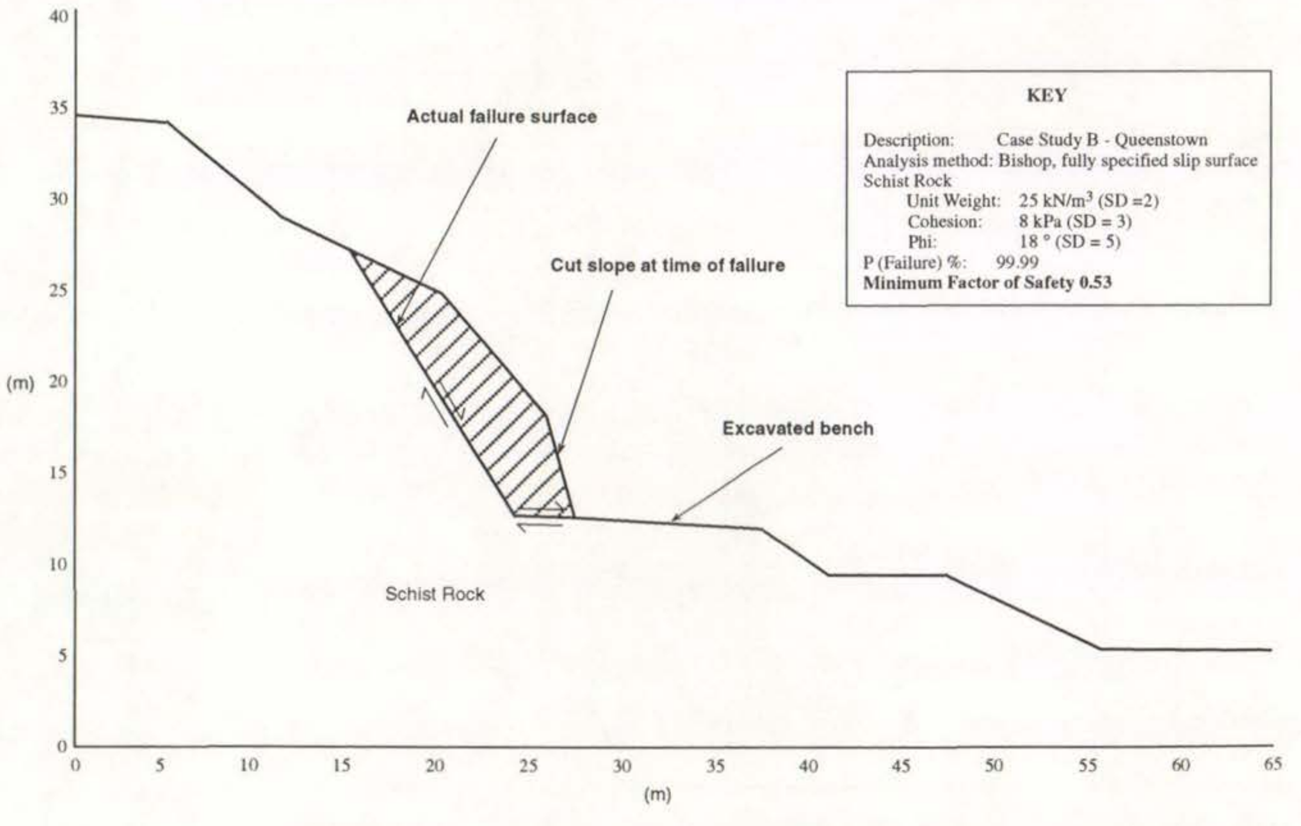


Figure B6: Range of potential failure geometries

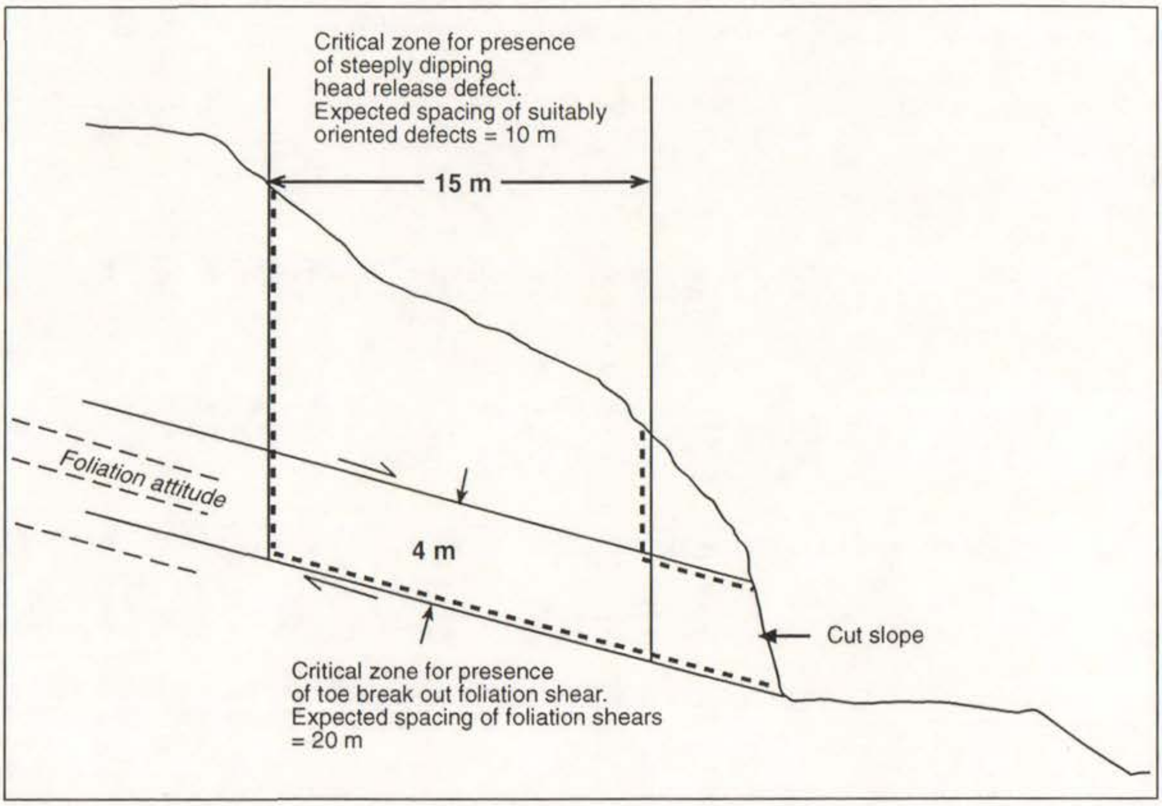
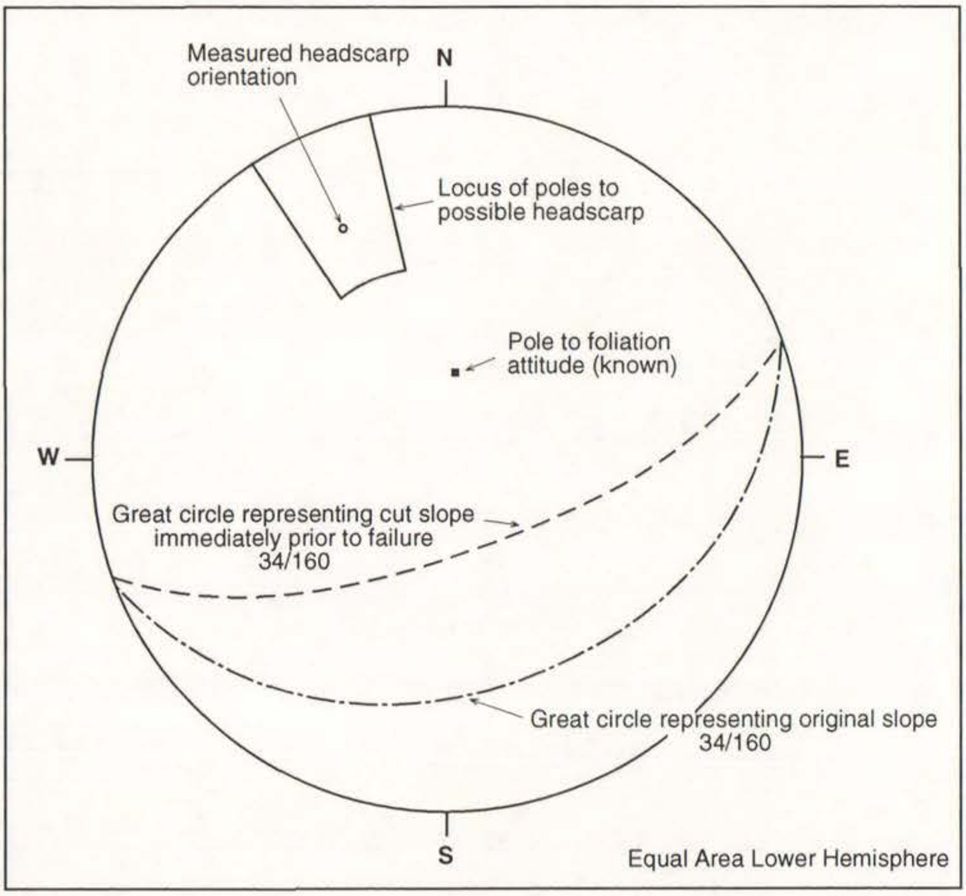


Figure B7: Stereoplot showing slope failure mechanism



Lateral release surfaces

By limiting the possible upslope extent of the slip (i.e. only considering failures that are wide in relation to their length), the role of lateral release surfaces (i.e. at either end of the failure) is correspondingly small, and may be ignored for the purposes of this assessment.

Overall probability

The overall probability that the identified failure mechanism is present is therefore:

$$0.0831 \times 0.2 = 0.017$$

ii) Risk Evaluation

Numerical slope stability analyses indicate that failure was inevitable given the existence of adversely orientated rock mass defects (i.e. probability is 1.0). The probability of the defects being present in the required configuration is assessed as 0.017. The risk of a landslide occurring at the site during construction is therefore the product of these two probabilities, 1 x 0.017 = 0.017, or odds of 1 in 59.

The assessed risk calculated here assumes a minimal knowledge of the specific ground conditions at the site, but experience of general rock mass conditions in the regional area. More detailed site investigation prior to the failure may have allowed the probability of occurrence of adverse defects to be better constrained. It further assumes that the likely failure mechanism could have been identified in principle prior to failure.

4.6.3 Risk Evaluation

An example of how commercial risk can be accommodated within a simple economic analysis is as follows. The costs used are hypothetical.

Item	**Cost**
Site Purchase	\$200,000
Development costs	\$500,000
Slope failure risk	
Slope failure remediation	\$400,000
x Assessed risk	0.017
	\$6800
Total development costs (including allowance for risk)	\$706,800
Value of completed development	\$1,000,000
Expected Profit	\$293,200 (41%)

This analysis suggests that the risk may have been acceptable in simple economic terms.

Whilst the risk calculated above is only likely to be accurate to an order of magnitude, the assessment would however have appraised the developer of the commercial risk associated with basement excavation on the site, and may have afforded an opportunity to revise the design or construction programme. Alternatively, the risk could have been accepted subject to health and safety requirements being met.

5. CASE STUDY C - NELSON

5.1 Background

Following heavy rainfall in July 1998, a landslip occurred on natural slopes above a residential site. Debris from the landslide encroached onto the property, and extended to the downslope boundary. It narrowly missed the single storey timber dwelling on the site.

5.2 Site Description

This site is situated near the toe of a slope forming the eastern side of the lower reaches of a river valley. Slope angles are moderately steep (up to 30°), and highly irregular and undulating reflecting the effects of past slope instability (Figure C1).

5.3 Investigations

Engineering geological investigations were carried out immediately following the July 1998 event, and more recently for the purposes of this study. A topographic survey was undertaken and engineering geological mapping of surface features carried out (Figure C1). 6 hand auger holes were bored to investigate subsurface ground conditions and to determine groundwater levels. Hand shear vane measurements were made in cohesive (clayey) soil to determine undrained shear strength.

5.4 Geological Setting

The slopes are underlain by a 2-3 m thickness of weathered colluvium, which is in turn underlain by weathered rock of the Brook Street Volcanics.

5.5 Stability Characteristics

A landslip occurred on the property after heavy rainfall on 2 July 1998. The landslip occurred within the colluvium at an approximate depth of 1.5 m deep. Debris up to 2 m thick extended to the downslope boundary of the property. A tension crack also formed extending across the slope from the headscarp of the failure. As a result of the slip the affected house was considered at risk of imminent loss due to further landslip, and was removed (Figures C2, C3).

A large number of older landslip features are present on the slopes above the dwelling. No information is however available concerning the timing of these events.

5.6 Quantitative Risk Assessment

5.6.1 Objectives

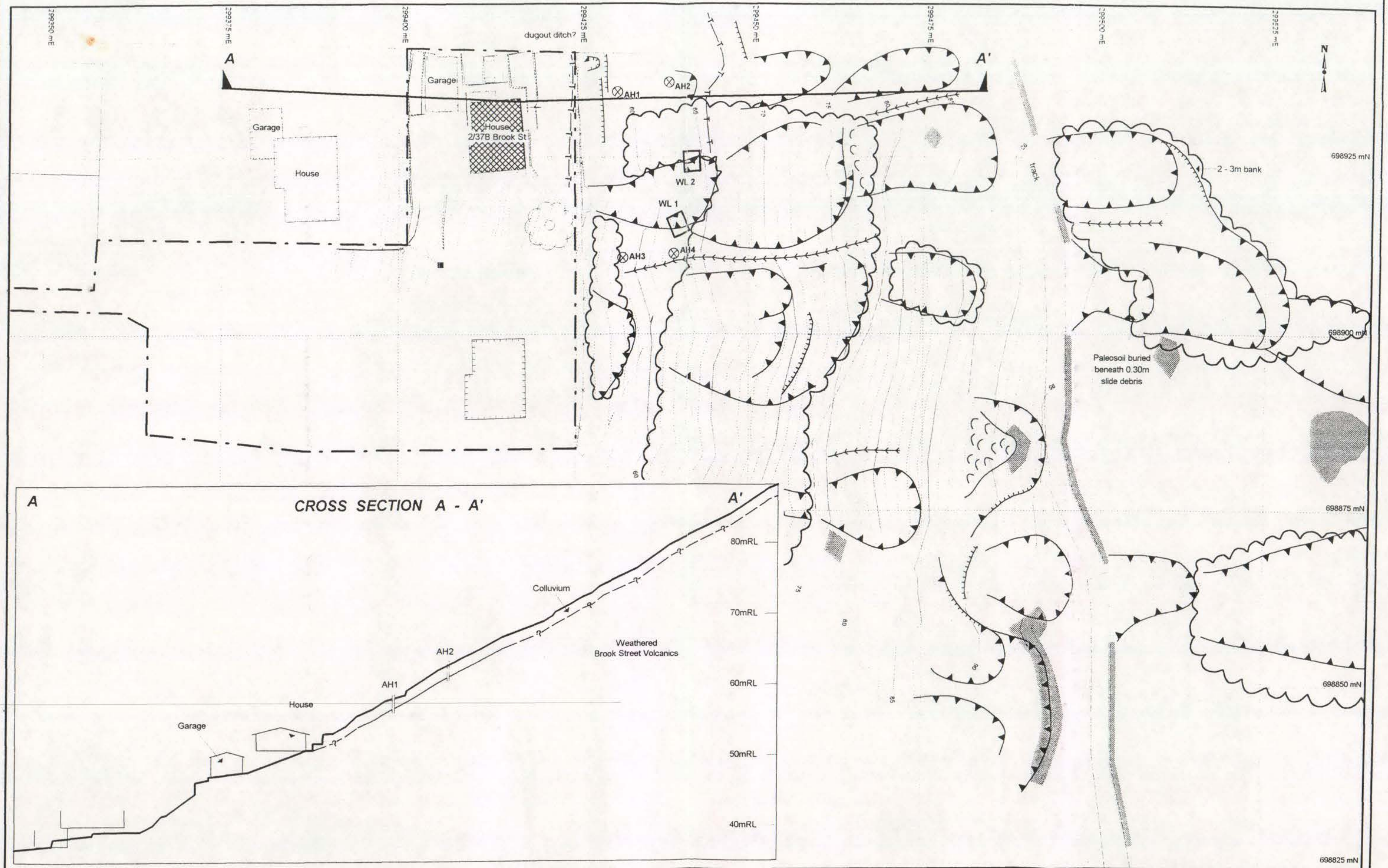
The objective of the quantitative risk assessment for this site is to assess the risk due to landsliding of:

- damage to the dwelling and
- death or injury to the most exposed occupant.

5.6.2 Risk Analysis

i) Probability of Failure

In this instance, the probability of the triggering event, namely the July 1998 rainstorm, is equated with the frequency of occurrence of landsliding itself.



LEGEND

AH1 Auger Hole	Steep Bank	Track	
WL1 Wall Log	Moderate Slope	Rock Outcrop (Sandstone)	
Tension Cracks	Ridge Line	Bush / Scrub	
Hummocky Ground	Surface Topography Contour (1m interval)		

5 0 5 10
metres

CLIENT	EARTHQUAKE COMMISSION	PROJECT	CALIBRATION STUDIES FOR LANDSLIDE QUANTITATIVE RISK ASSESSMENT
DRAWN MCD	DATE October 2000	TITLE	CASE STUDY C: NELSON; ENGINEERING GEOLOGICAL SITE PLAN
CHECKED PJH	DATE October 2000		
SCALE 1:500	FILE SitePlan500.wor A3	PROJECT No 436	REPORT No / FIGURE No C1

Figure C2: July, 1998 Landslip

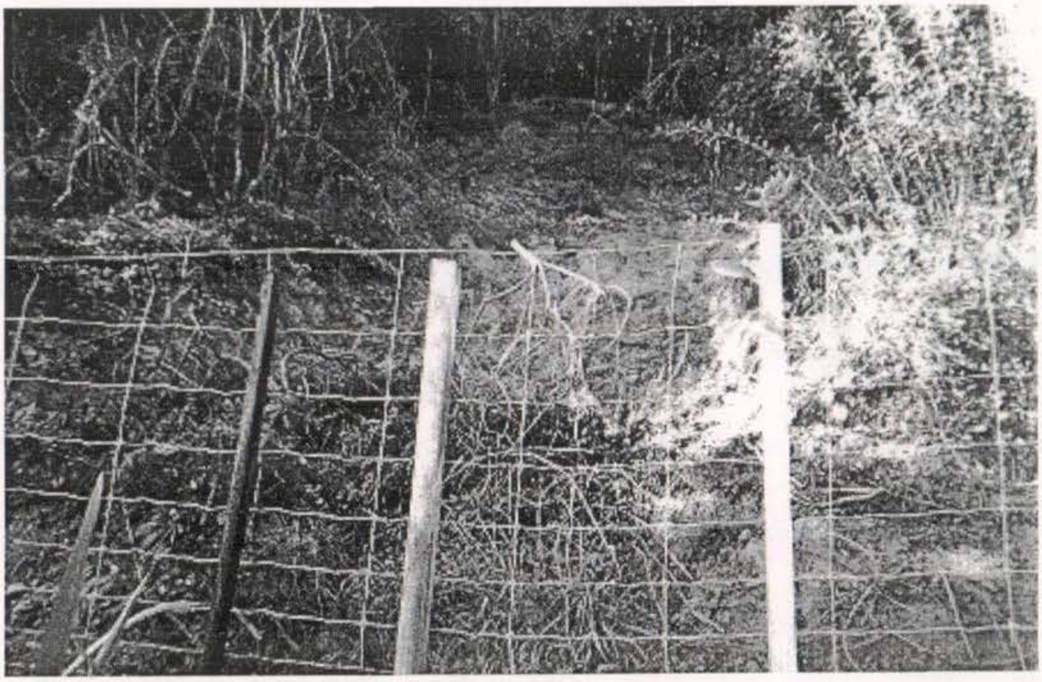
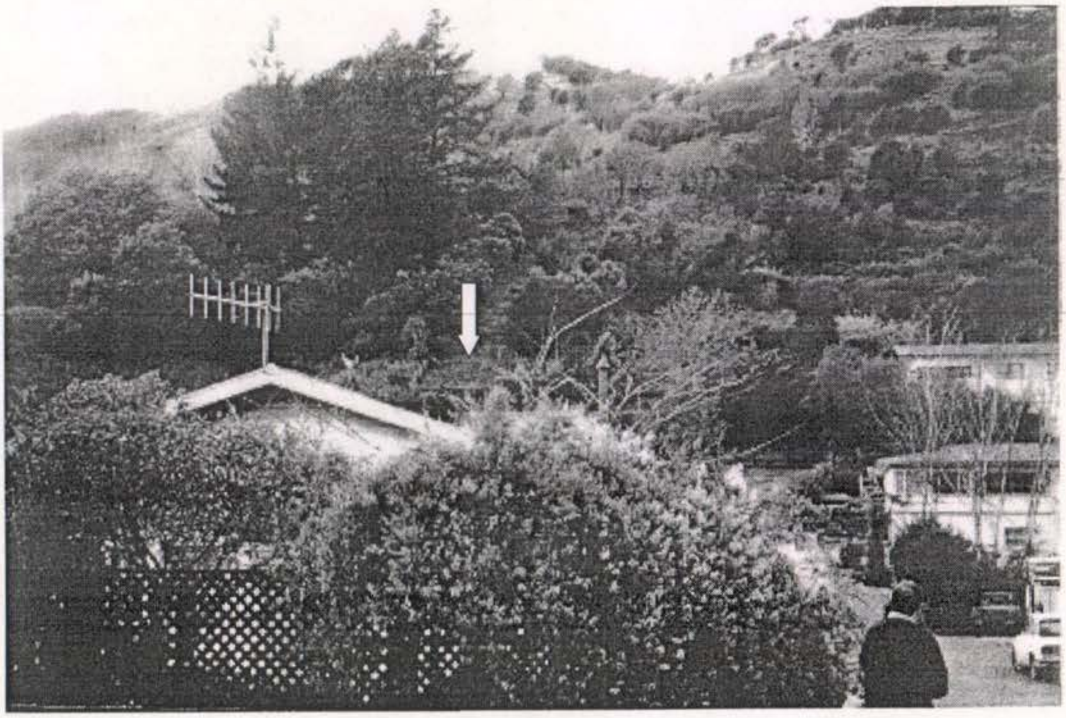


Figure C3: Slope viewed from below. Former house site arrowed



A high intensity rainfall initiated the failure of 2 July 1998. Records obtained for the Nelson Airport rain gauge indicates 173 mm of rain fell in the 48 hours preceding the landslip of July 1998 (Figure C4). Comparison with NIWA's High Intensity Rainfall Design System (HIRDS) database indicates an event of this magnitude to have a recurrence interval of approximately 14 years (Figure C5). By assuming that, on average, a rainfall event of this magnitude will always result in a slope failure of similar size to the 1998 event, a failure probability of 0.05 to 0.25 (i.e. 1 in 4 to 1 in 20 years) appears reasonable.

Engineering geological mapping (Figure C1) has identified a number of well-defined previous slope failures of similar size to the 1998 event. This provides further evidence that landslips have occurred repeatedly during the past 100-200 years, which is consistent with the recurrence interval calculated above.

ii) Spatial Probability

Not all slope failures occurring on the slopes above the dwelling will impact upon it. Spatial probability of a landslide impacting upon the dwelling must be estimated. There are a number of difficulties in determining the spatial probability. Firstly, development of the site has removed geomorphological evidence of any previous failure, which may have affected the site. Secondly, whilst evacuated slip scarps are identified on the slopes above the property, the actual travel distance associated with these failures is not often evident from the existing surface features.

Empirical methods of estimating runout distances for various slope angles have been published. Generally these are more appropriate when the landslip will run out onto a horizontal surface. Estimates for travel distances for landslips moving across sloping ground are more problematic. Investigators have found better correlations for engineered slopes (cut and fill slopes) than for natural slopes. In view of these limitations a subjective judgement has been made that there is a 1 in 5 to 1 in 20 chance that a slope failure occurring on the slopes directly above the site will impact upon the dwelling.

iii) Temporal Probability

In the event that a landslip hits the dwelling, death or injury may only occur if people are present in the house at the time. The temporal probability of the house being occupied could range from almost 0 (for an absentee occupier) to almost 1 (for an elderly or confined resident). An average value of 0.5 would be reasonable in the absence of a more specific knowledge of the occupier's lifestyle.

The temporal probability for the case of damage to the dwelling is obviously 1, as the dwelling is inanimate, and therefore always present.

iv) Vulnerability

In the event that a landslip impacts upon the house when it is occupied, death to the occupant will not necessarily occur.

Based on the engineering geological of the magnitude of previous landslips at the site, destruction of the building or complete inundation is not considered likely. A vulnerability between 0.2 and 0.4 for death of an occupant given that a landslip strikes the house is considered appropriate.

Clearly the vulnerability of the building itself to damage in the event that it is struck by a landslip is 1.0.

Figure C4: 24 hour antecedent rainfall , Nelson Airport

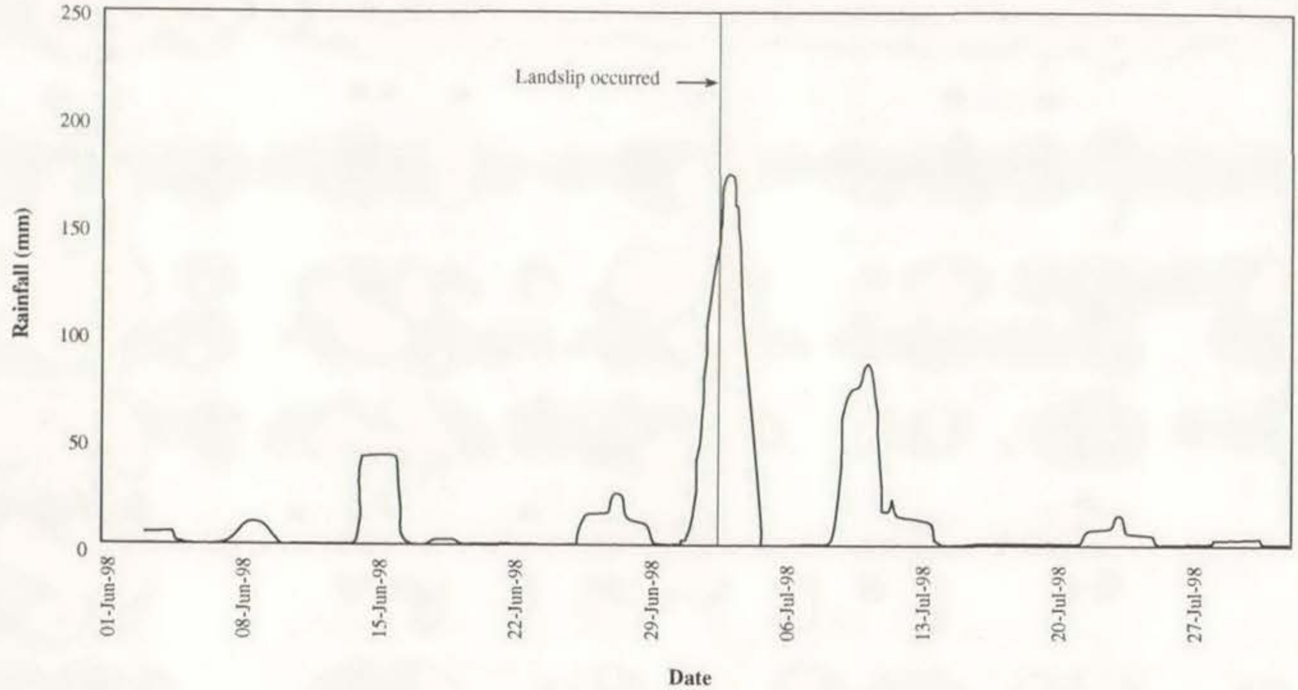
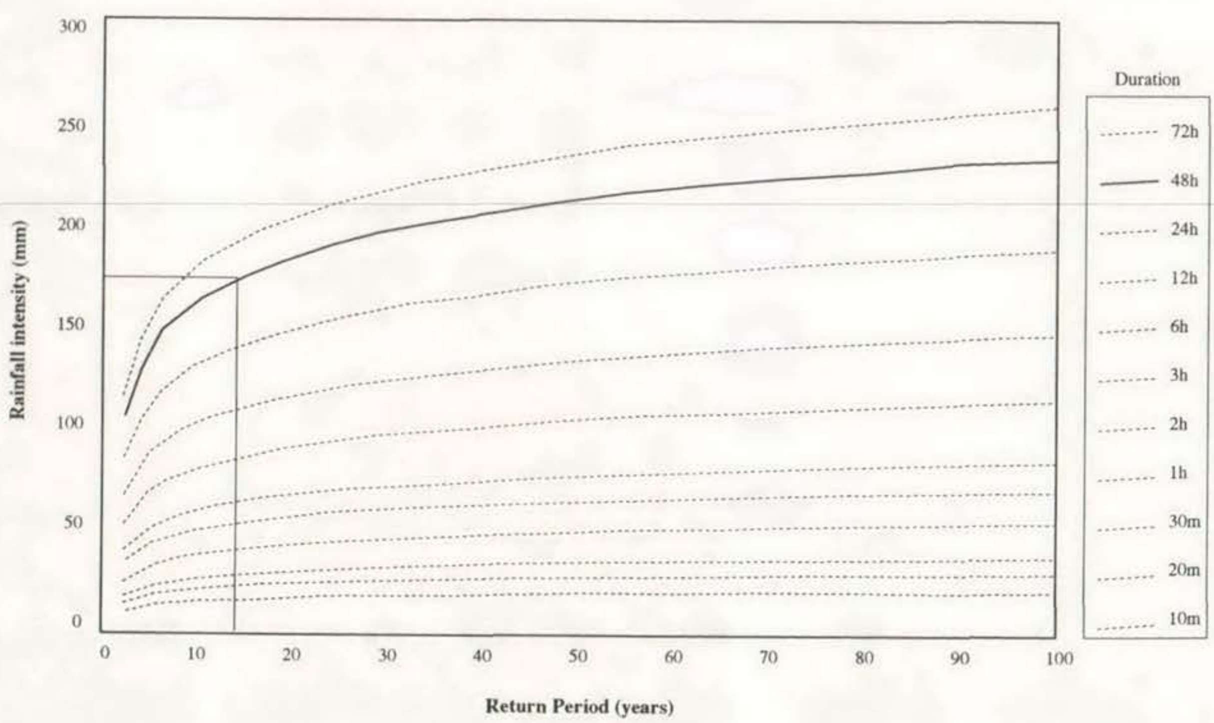


Figure C5: Rainfall event return periods, Nelson



v) Risk Estimation

Once the probability of failure, the spatial and temporal probabilities of impact and the vulnerability of the elements at risk are identified the risk may be calculated as the product of these four parameters. The calculations are presented in Tables C1 & C2. The calculation may be made using "best estimate" values of the input parameters. By specifying a likely range of values for each input parameter a simple Monte Carlo simulation may also be carried out to determine the probability distribution of each of the risks assessed. This approach is demonstrated in Tables C1 & C2, which include a plot of the probability distribution of each risk

The risk posed by a landslip prior to July 1998 has been assessed as:

Risk of death to an occupant (mean): 0.0014 (1 every 711 years)

Risk of damage to the dwelling (mean): 0.0093 (1 every 107 years)

5.6.3 Risk Evaluation

Comparison with acceptability guidelines (Figure 2), shows that in this instance the assessed risk of death to the occupants is unacceptably high. This would have alerted the relevant territorial authority responsible for the issue of subdivision and or building consents that either avoidance of the risk or mitigation of the hazard would have been required to allow the site to have been developed.

While there are no guidelines for the risk of damage to the dwelling, if the assessed risk of damage had been known to a prospective purchaser, they would probably have considered that damage to the house once during its design life (that is, once every 100 years) is unacceptably high.

Table C1: Risk of death to occupant due to Landslip

	Min	Most Likely	Max
Probability of Failure P (F)	0.05	0.125	0.25
Spatial Probability P (S)	0.05	0.125	0.2
Temporal Probability, P(T)	0.4	0.5	0.6
Vulnerability V	0.2	0.3	0.4

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.0002	Minimum frequency of occurrence	5000 years
Maximum risk	0.0048	Maximum frequency of occurrence	208 years
Average risk	0.0014	Average frequency of occurrence	711 years

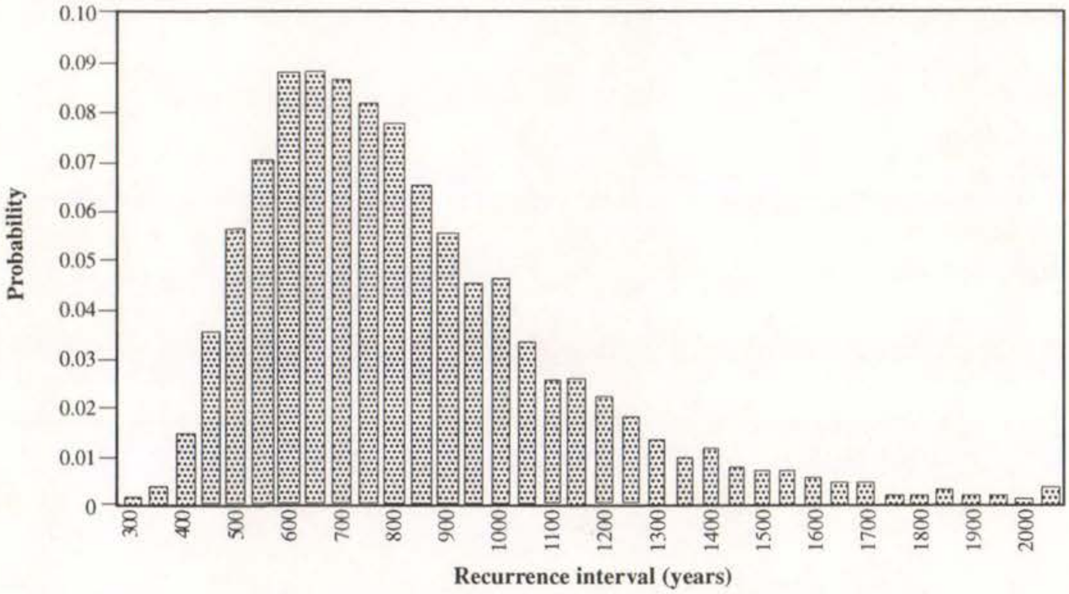
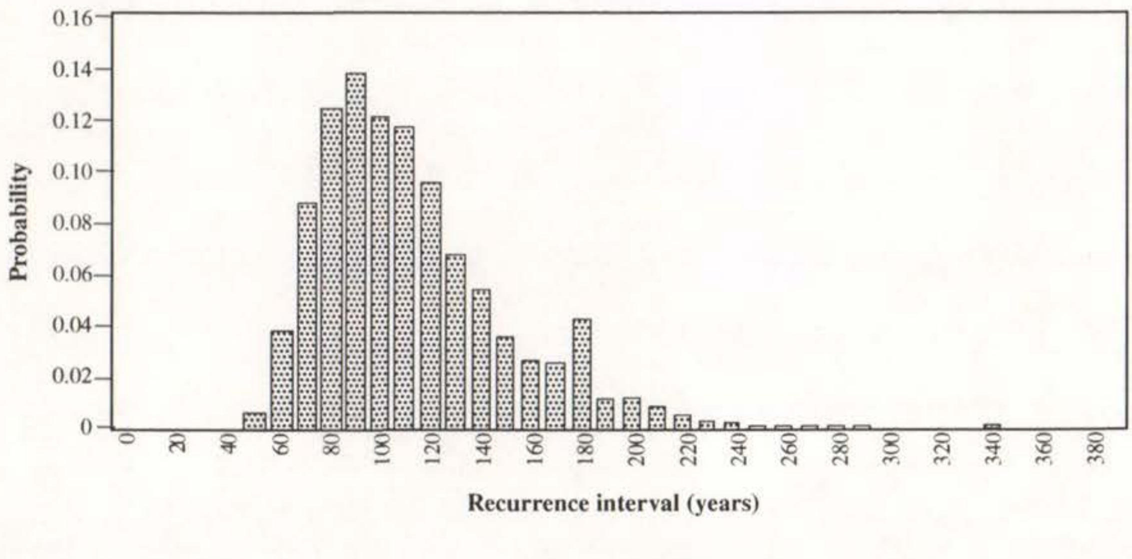


Table C2: Risk of damage to dwelling due to Landslip

	Min	Most Likely	Max
Probability of Failure P (F)	0.05	0.075	0.1
Spatial Probability P (S)	0.05	0.125	0.2
Temporal Probability, P(T)	1.0	1.0	1.0
Vulnerability V	1.0	1.0	1.0

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.0025	Minimum frequency of occurrence	400 years
Maximum risk	0.02	Maximum frequency of occurrence	50 years
Average risk	0.0093	Average frequency of occurrence	107years



7. CASE STUDY D - NELSON

7.1 Background

A landslip comprising predominantly soil material occurred at this site in July 1998. The slip occurred on a cut slope at the rear of a residential site, and impacted slightly upon a wooden single storey dwelling.

7.2 Site Description

The moderately steep (33°) cut slope affected by the landslip is located to the rear of the dwelling (Figures D1 to D4). The slope was first excavated in 1986, and the existing house built around the same time. It was excavated further in 1997 to increase the available level area. Prior to failure the slope was approximately 8 m high.

7.3 Investigations

Available reporting was reviewed, and engineering geological mapping carried out (Figures D3, D4).

7.4 Geological Setting

The slope is formed predominantly of yellow-brown weathered greywacke gravel and minor sand lenses of the Port Hills Gravel Formationof upper Tertiary age. The formation underlies much of the land in the Nelson urban area. A light grey slightly carbonaceous (organic) and slickensided (polished from previous movement) silty clay layer up to 300 mm thick interbedded with the greywacke gravel outcrops near the toe of the slope, and dips at 20° out of the slope. (Figures D3 and D4).

7.5 Stability Characteristics

7.5.1 Instability history

Failure of part of the cut slope occurred on 2nd July 1998 during a period of heavy rainfall resulting in debris accumulating very close to the house.

7.5.2 Ground and groundwater conditions

The failure appears to have occurred as a result of movement along the silty clay layer described above, which forms a plane of weakness in the Port Hills Gravel mass. The mode of failure is thus essentially planar, with lateral release surfaces occurring through intact Port Hills Gravel.

Groundwater seepage was observed in the lower part of the slope (Figure D3), coincident with the silty clay layer, suggesting the layer acted as an aquitard.

7.6 Quantitative Risk Assessment.

7.6.1 Objectives, Methods and Limitations

The purpose of the analysis is to determine whether QRA techniques could be applied as a predictive tool to assess the potential for landsliding prior to the event of 2 July 1998. The objective of this case study is therefore to assess the landslide risk of

(i) damage to the dwelling, and
(ii) death of an individual occupant.

Figure D1: View showing slip debris against house

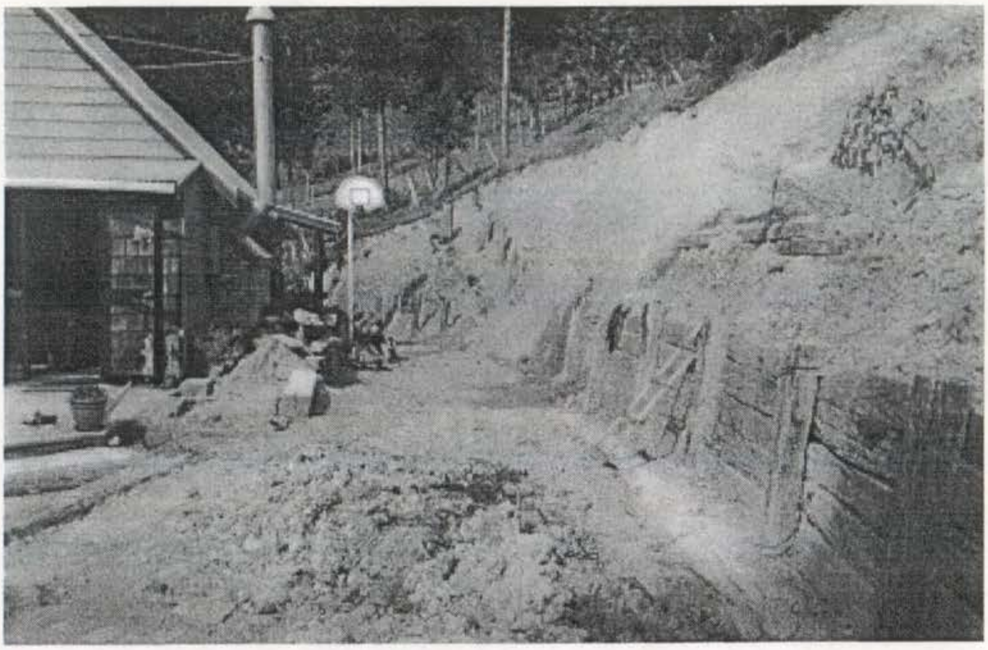


Figure D2: View showing slip

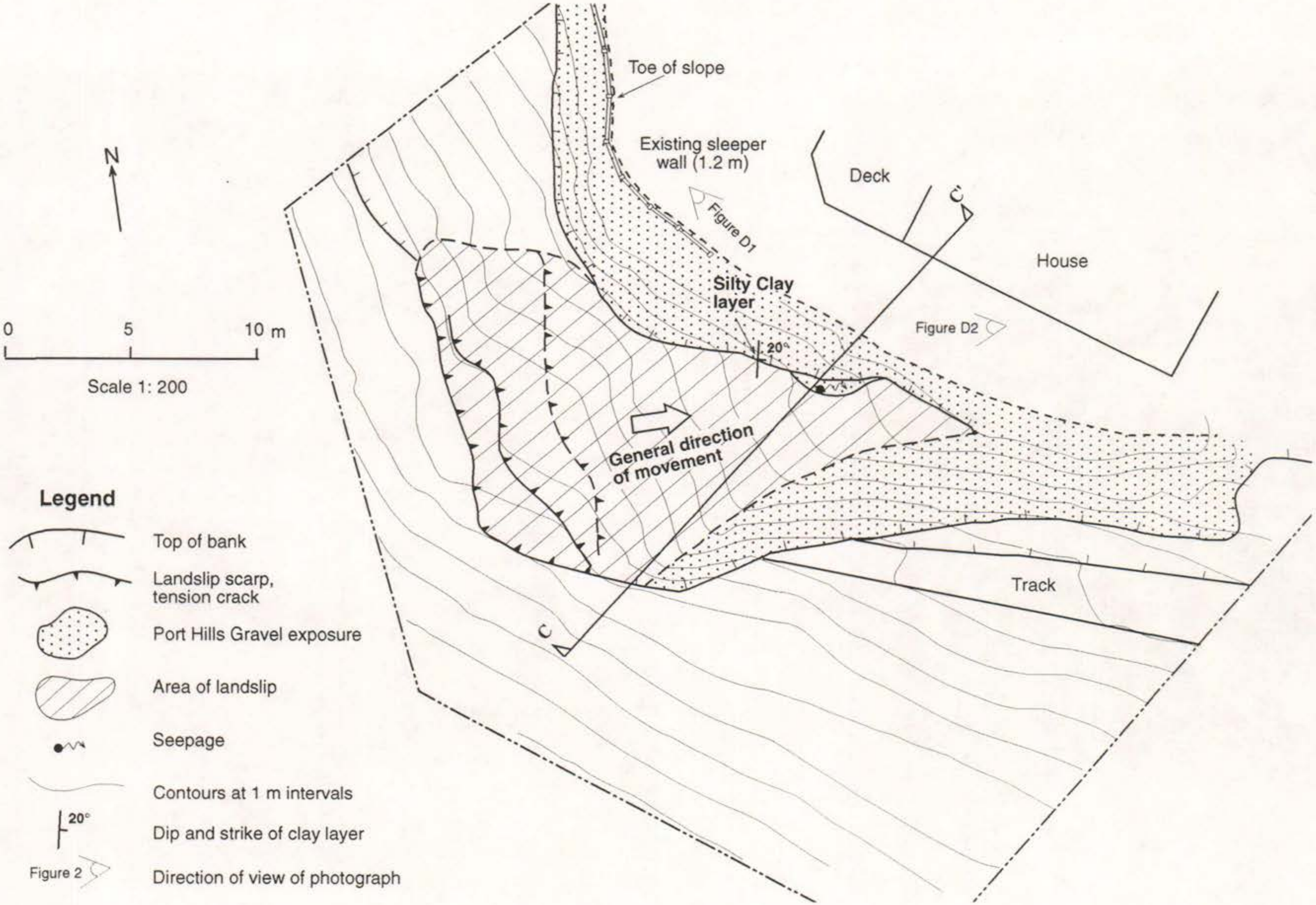


Figure D3: Site Plan

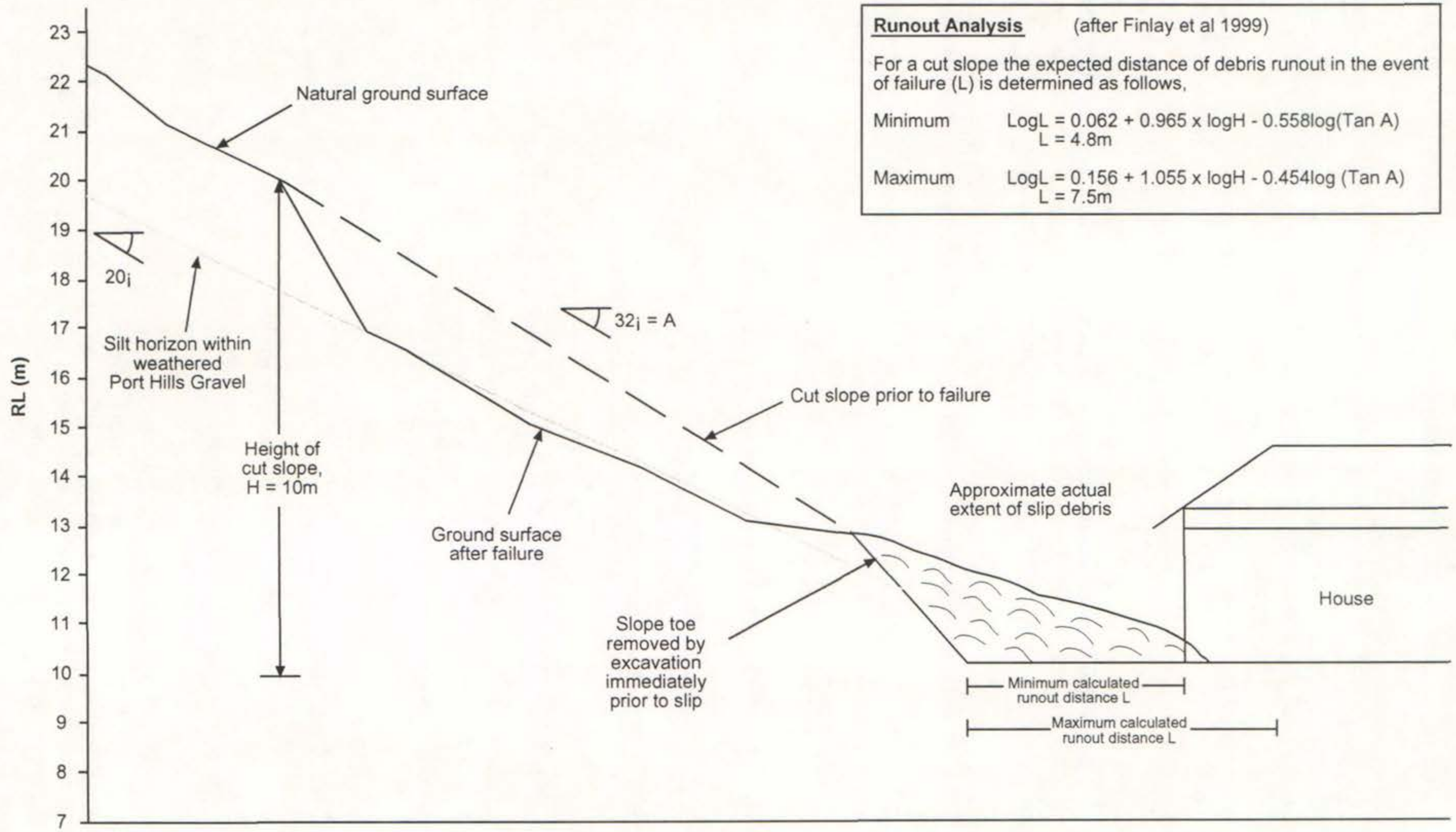


Figure D4: Cross section C-C' sketch, and run out analysis

The QRA analysis presented here has a number of limitations that are discussed more fully in other sections. In particular, the analysis is based upon a deterministic slope stability assessment in which the input parameters are necessarily assumed. Such assumptions are a common limitation of geotechnical analysis, and are not considered to be a significant impediment to QRA analysis as sound judgement is commonly used by many practitioners to evaluate the likely engineering properties of the affected slope materials.

7.6.2 Risk Analysis

i) Probability of failure

The slope failure occurred during removal of the toe of a cut slope that had been formed 12 years earlier. It also followed a period heavy rainfall. The cut slope had not shown signs of instability prior to the initiation of failure. The low angle silt horizon would have been visible in the cut slope prior to failure, as would any seepage perched above it. The silt horizon comprised the basal failure surface, whilst the eastern lateral release surface was formed by shearing through much stronger intact Port Hills Gravel. Whilst the possibility of failure along the silt horizon could have been identified prior to failure, the location of the lateral release surface could not have been reliably predicted. It is therefore reasonable to consider planar sliding on the silt layer as the likely slope failure mode.

The probability of failure has been estimated using a deterministic slope stability approach, with the inputs provided in terms of probability distributions. In this instance, in the absence of measured shear strength data, shear strength parameters have necessarily been assumed. Analyses have been carried out to assess the sensitivity of the slope model to changes in piezometric pressures and removal of toe support as a result of toe excavation.

The analysis assumptions are as follows:

- Planar sliding along the silt horizon
- Groundwater almost coincident with ground surface
- Effective cohesion 4 kPa, Effective friction angle 28°
- Unit weight of soil 20 kN/m^3
- Presence of a tension crack at the top of the cut slope

The analysis has been carried out for the slope both before and after toe removal. Derivation of a failure probability requires information about the variability of the input parameters. Using assumed standard deviations for the soil input parameters, Monte Carlo simulation has been carried out to derive a failure probability (Figures D5, D6). Given that both the mean and standard deviation of the input parameters are assumed this approach has some major limitations in this case. Factors of safety and failure probabilities calculated for the slope before and after toe removal are as follows.

	FOS	**P(F)**
Slope prior to toe removal	1.23	0.155
Slope After toe removal	1.11	0.279

The above analysis assumes the slope is fully saturated. Inspection of rainfall data indicates the storm event that preceded the slope failure to have a return period between 10 and 20 years (refer Case study C, which occurred following the same rainfall event). The annual probability of occurrence of the groundwater conditions that preceded failure is therefore 0.05 to 0.1. The probability of failure values determined from slope stability analysis must therefore be multiplied by these values to determine the actual probability of the failure occurring (see figure QRA calculation)

Figure D5: Results of stability analysis for slope before removal of toe support

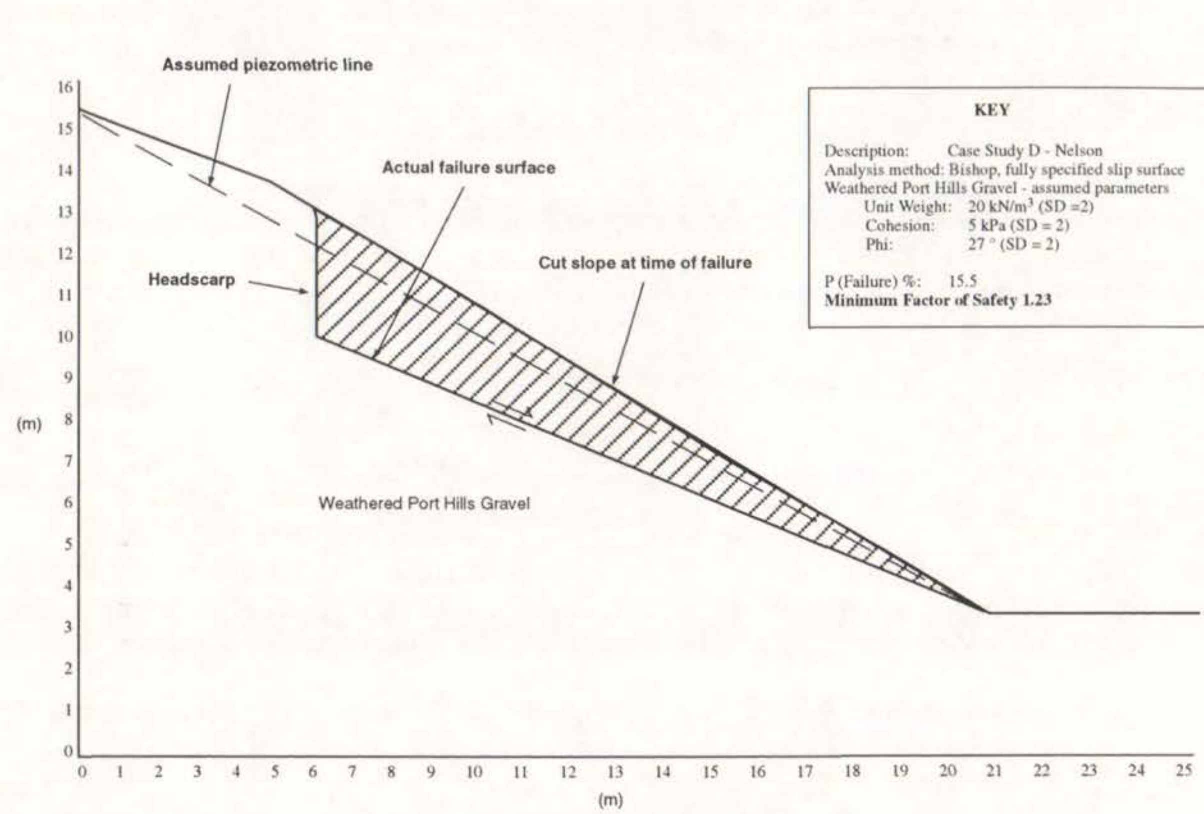
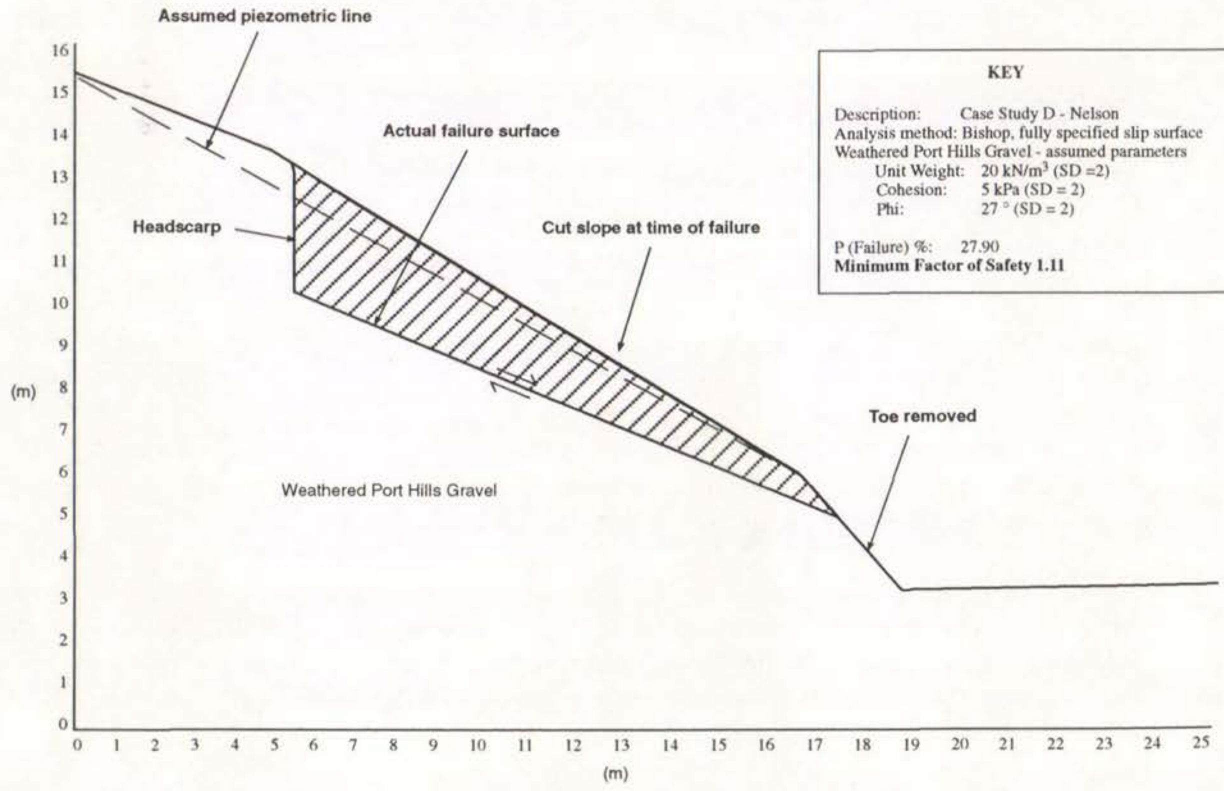


Figure D6: Results of stability analysis for slope after removal of toe support



ii) Spatial Probability.

Using the criteria of (Finlay et al, 1999) the expected runout distance has been calculated for this site as ranging from 4.8 to 7.5 m beyond the toe of the slope (Figure D4). Given that the rear wall of the house is 4.5 m from the toe of the slope, the spatial probability P(S) of the slide debris impacting upon the house is effectively 1.

iii) Temporal Probability

In the event that a landslip hits the dwelling, death or injury may only occur if people are present in the house at the time. The temporal probability of the house being occupied could range from almost 0 (for an absentee occupier) to almost 1 (for an elderly or confined resident). An average value of 0.5 would be reasonable in the absence of a more specific knowledge of the occupier's lifestyle.

The temporal probability P(T) for the case of damage to the dwelling is obviously 1, as the dwelling is inanimate, and therefore always present.

iv) Vulnerability

In the event that a landslip impacts upon the house when it is occupied, death to the occupant will not necessarily occur. Assuming that a landslip impacts the house, a vulnerability value of between 0.2 and 0.4 for death of an occupant is considered appropriate in this instance.

Clearly the vulnerability (V) of the building to damage in the event that it is struck by landslip is 1.0.

v) Risk Estimation

Calculation of risk is shown in Figures (Tables D1-D4). The results show that the mean risk of damage to the dwelling increased from 0.011 (1 in 89 years) prior to toe removal, to 0.02 (1 in 49 years) after toe removal. Similarly, the mean risk of death to a single occupant increased from 0.0016 (1 in 593 years) prior to toe removal, to 0.003 (1 in 329 years) after toe removal.

The risk calculations indicate that removal of the toe significantly increased the risk of both house damage and death of an occupant.

7.7 Risk Evaluation

The assessed risks of death to the most exposed occupant of this property are considerably higher than the suggested guidelines provided for new engineered slopes of 10^{-5} to 10^{-6} (Section 2.2.2; Table 2) for both before and after toe removal. While there is some uncertainty with the input parameters for the deterministic slope stability analysis and hence the failure probabilities, these would need to be wrong by several orders of magnitude for the results to significantly change the result. As well, the value of QRA in this instance has been to demonstrate quantitatively that toe removal will result in an approximate doubling of the numerical risk value (from 1 in 593 years to 1 in 330 years), which is very significant.

Similarly, the assessed risk of damage to the house is also high, and would probably have been considered unacceptable by the owner had the risk been known.

Quantitative methods therefore have applicability in gaining an understanding of the relative changes in risk associated with a particular course of action, in this case, removal of the slope toe. If the risk had been identified prior to failure, a number of risk management strategies may have been applicable, including overall flattening of the cut slope, progressive retaining or relocation of the house further from the toe of the cut.

Table D1: Risk of damage to dwelling due to landslip prior to toe removal

	Min	Most Likely	Max
Probability of failure assuming saturated slope	0.15	0.15	0.15
Annual probability of rainfall event	0.05	0.075	0.1
Probability of Failure P (F)	0.0075	0.01125	0.015
Spatial Probability P (S)	1.0	1.0	1.0
Temporal Probability, P(T)	1.0	1.0	1.0
Vulnerability V	1.0	1.0	1.0

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.0075	Minimum frequency of occurrence	133 years
Maximum risk	0.015	Maximum frequency of occurrence	67 years
Average risk	0.011	Average frequency of occurrence	89 years

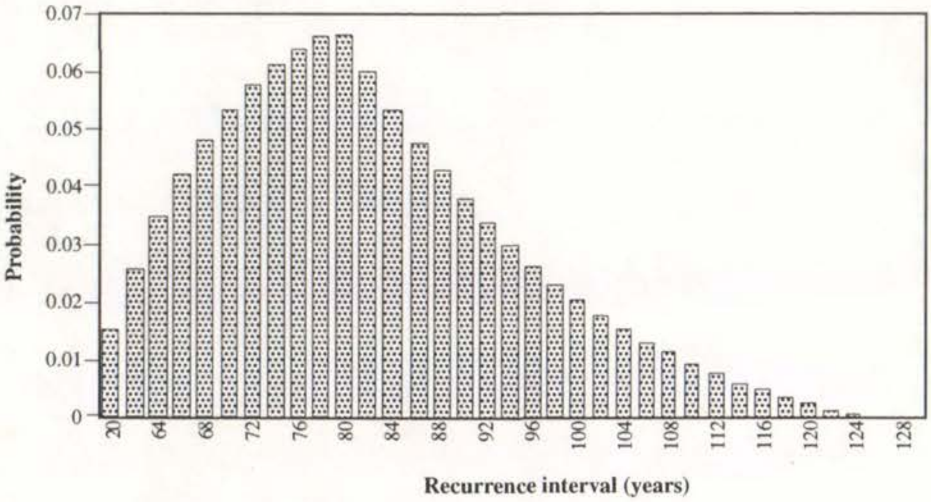


Table D2: Risk of damage to dwelling due to landslip after toe removal

	Min	Most Likely	Max
Probability of failure assuming saturated slope	0.27	0.27	0.27
Annual probability of rainfall event	0.05	0.075	0.1
Probability of Failure P (F)	0.0135	0.02025	0.027
Spatial Probability P (S)	1.0	1.0	1.0
Temporal Probability, P(T)	1.0	1.0	1.0
Vulnerability V	1.0	1.0	1.0

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.0135	Minimum frequency of occurrence	74 years
Maximum risk	0.027	Maximum frequency of occurrence	37 years
Average risk	0.02	Average frequency of occurrence	49 years

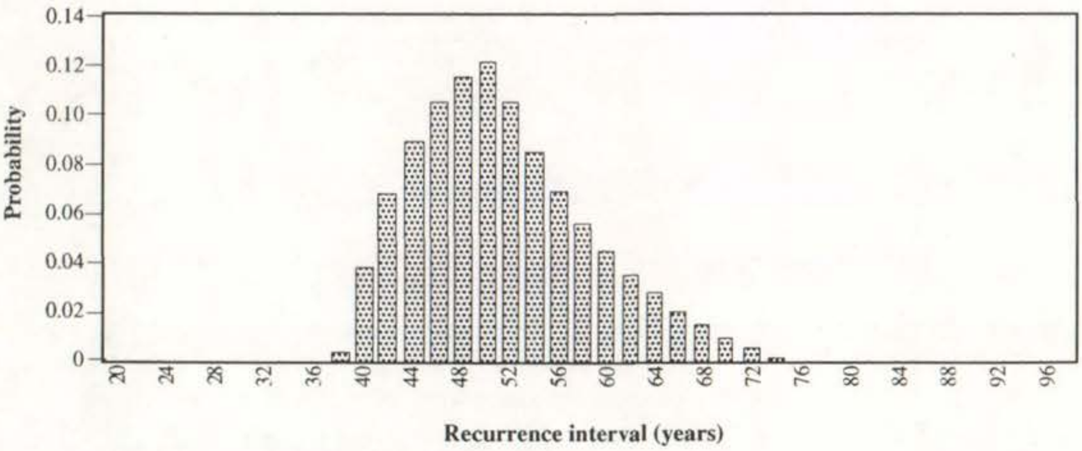


Table D3: Risk of death to occupant due to landslip prior to toe removal

	Min	Most Likely	Max
Probability of failure assuming saturated slope	0.15	0.15	0.15
Annual probability of rainfall event	0.05	0.075	0.1
Probability of Failure P (F)	0.0075	0.01125	0.015
Spatial Probability P (S)	1.0	1.0	1.0
Temporal Probability, P(T)	0.4	0.5	0.6
Vulnerability V	0.2	0.3	0.4

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.0006	Minimum frequency of occurrence	1667 years
Maximum risk	0.0036	Maximum frequency of occurrence	278 years
Average risk	0.0016	Average frequency of occurrence	593 years

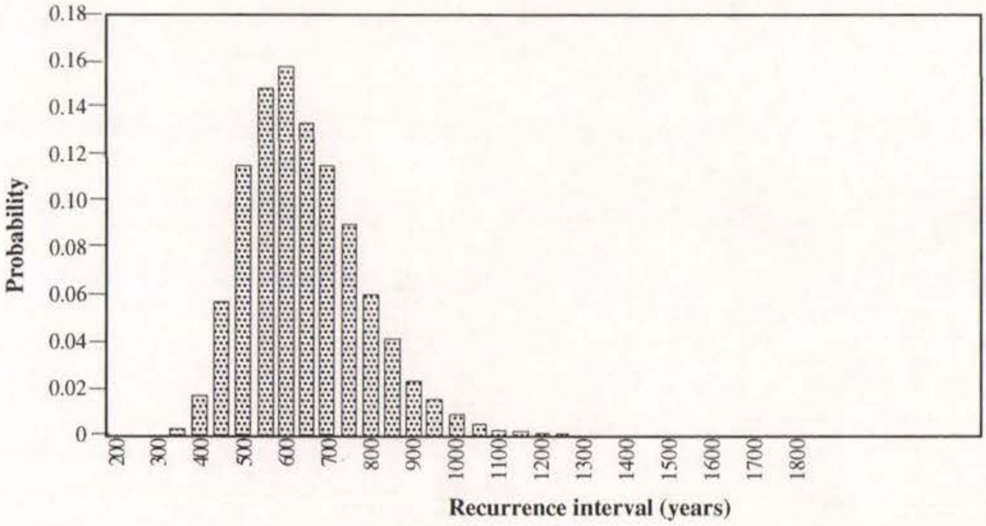
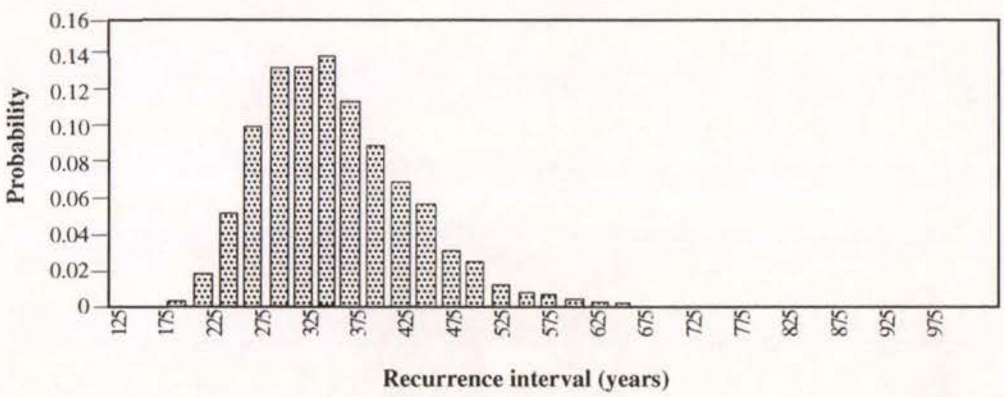


Table D4: Risk of death to occupant due to landslip after toe removal

	Min	Most Likely	Max
Probability of failure assuming saturated slope	0.27	0.27	0.27
Annual probability of rainfall event	0.05	0.075	0.1
Probability of Failure P (F)	0.0135	0.02025	0.027
Spatial Probability P (S)	1.0	1.0	1.0
Temporal Probability, P(T)	0.4	0.5	0.6
Vulnerability V	0.2	0.3	0.4

Risk = P(F) x P(S) x P(T) x V

Minimum risk	0.001	Minimum frequency of occurrence	926 years
Maximum risk	0.006	Maximum frequency of occurrence	154 years
Average risk	0.003	Average frequency of occurrence	329 years



8. ASSESSING IMMINENT LOSS CLAIMS

8.1 Introduction

The Earthquake Commission Act (1993) allows EQC to consider a claim where physical loss or damage to a property is, in the opinion of the commission, imminent as a direct result of a natural disaster that has occurred. A further objective of this research project is to determine if QRA methods could help in assessing if loss is "imminent".

The intention of this provision of the Act is to allow EQC to fund remedial work to prevent further property damage rather than waiting until damage occurs. EQC typically relies upon the recommendation of consulting geotechncial professionals to help it decide whether immediate action should be taken.

Clearly an unambiguous definition of the term "imminent" is required, although this is not included in the Act. EQC (or more specially their legal advisers) have defined an imminent event as one that is expected to occur within the time frame of one year (i.e. the annual probability of occurrence is 1.0). The rationale behind this, is that it allows for four seasons to pass, therefore represents a reasonable range of climatic conditions, which is of particular relevance to landslides. EQC have developed a checklist for use by their geotechnical advisors in assessment of imminent loss claims (See Appendix 1).

In order to determine whether loss is imminent under the above definition, the risk of further loss or damage to property must be quantified.

8.2 Quantitative Risk Assessment and Imminent Loss

Assessments of imminent loss claims are essentially an attempt by the geotechnical practitioner to predict the timing of a future landslip event. Whilst quantitative risk assessment can assist in deriving the odds of the event occurring (i.e. the probability), it has no power to determine the actual timing of the event. The use of QRA analysis is therefore a means for providing a structured assessment of the cause and effect relationships between the probability of landsliding and its likely consequences should failure occur.

Determining, for example, if loss is expected to occur within the next 11 months or the next 13 months is well beyond the resolution of a quantitative landslide risk assessment in most cases.

However, the elements of a quantitative landslide risk assessment may be applied to assist the engineering judgements that are typically used for the assessment of imminence. Using the above definition of "occurrence within 1 year ", loss or damage is imminent if:

$$P(F) \times P(S) \times P(T) \times V > 1$$

where: $P(F)$ = Annual probability of (further) failure

$P(S)$ = Probability of failure impacting upon the property

$P(T)$ = Probability of the property being present (always = 1 for fixed property)

$P(V)$ = Vulnerability of the property to further damage given impact

The following check list of items may be considered by the geotechnical professional in determining the risk of further loss or damage due to landsliding, all of which are required to be evaluated for a formal QRA analysis:

- assessment of type of landslide failure (input required to determine failure probability)

- deformation monitoring by either inspection or surveying (input required to determine failure probability)

- presence of seepage or adverse groundwater conditions (input required to determine failure probability)

- steepness of the slope (input required to determine failure probability)

- the types of slope forming materials (input required to determine failure probability)

- prevailing or forecast weather conditions (input required to determine failure probability)

- location of the property in relation to the likely landslide path (input required to assess consequences of failure)

- likely magnitude of future landslides (input required to determine failure consequences)

- location of all properties with respect to the suspect slope (input required to determine failure consequences)

- type of property construction (input required to determine failure consequences)

Assessment of the landslide risk in QRA terms will assist the practitioner to understand the level of risk involved. If quantitative expression of risk is given by the geotechnical professional the limitations and uncertainties of the risk assessment should be clearly communicated.

8.3 An Alternative Approach to Imminent Loss

A suggested alternative definition of "imminent loss" may include a closer focus on the consequences of further slope failure. The present definition considers "loss or damage" to "property" within a prescribed time frame, but does not consider the likely severity of the "loss or damage".

The intent of the imminent loss provision is to allow remedial work to be undertaken prior to a landslip occurring, rather than waiting until the event occurs and repairing the damage. The test as to whether or not this should be done is therefore " is it better value to remediate prior to a landslip or after it has occurred". This will differ from claim to claim.

In order for EQC to decide on a course of action the following information would be required from the geotechnical professional (perhaps in conjunction with a professional valuer). Again considerable judgement is required in determining these inputs.

- What is the risk of loss or damage to the property (R)
- What is the expected value to fix the property if loss or damage occurs (C)
- What remedial measures could be carried out to reduce the landslide hazard
- What would they cost (C1)
- What is the mitigated risk of loss or damage to the property (R1)

In simplistic economic terms, if $C \times R > C1 + (R1 \times C)$, immediate remediation is favoured. If $C \times R < C1 + (R1 \times C)$, no immediate action is favoured.

In a global sense the decision must be made mindful of factors beyond simple monetary costs of remedial measures and consequences. Other factors may include safety of occupants and the general public, security of adjacent properties, and disruption to occupants. The extent to which EQC, as an insurer of physical property, is obligated in these areas is beyond the scope of this study.

The above approach may be illustrated by a simple hypothetical example.

Say a house is threatened by a landslip which is judged to have a 20 % probability of failing and causing loss or damage to a house. If failure occurs it is estimated that repair of the house will cost \$30,000. On the other hand, if \$3000 dollars is spent on drainage works, the probability of the slope failing and damaging the house is estimated to reduce to 1 %. Following the above equation:

C x R = 30,000 x 0.2 = 6000 (the expected cost of the "wait and see" approach)
and
C1 + (R1 x C) = 3000 + (0.01 x 30,000) = 3300 (the expected cost of the "fix now" approach

As C x R is greater than C1 + (R1 x C) the immediate installation of drainage is the preferred course of action.

8.4 Discussion

The main benefit of QRA analysis is in the structured methodology that it employs, in that it allows all of the inputs into the "imminent loss" decision making process to be addressed in a systematic approach. While the assessed risk value will probably be based mainly on engineered judgement, the calculated level of risk provides the practitioner with a guide as to risk acceptability.

All recommendations to EQC as to whether loss is imminent must still be based primarily on judgement and experience.

An alternative definition of "imminent loss" is proposed which considers the overall cost of remediation prior to a landslip against cost of restoration after the event has occurred.

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