NEW ZEALAND GEOTECHNICAL SOCIETY INC
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SLOPE STABILITY GEOTECHNICAL GUIDANCE SERIES

UNIT 1 – GENERAL GUIDANCE

AN INDUSTRY REFERENCE DOCUMENT COMPILED AND PUBLISHED BY THE NEW ZEALAND GEOTECHNICAL SOCIETY (NZGS)

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QUICK GUIDE

The diagram below summarises the structure of Unit 1, for use as a quick guide. Each part of the guidance provides more detail of the summary provided below, as well as references and links to many external documents.

PART 1: INTRODUCTION

This section outlines the scope of Unit 1, the intended audience as well as existing codes, standards and other documents that relate to slope stability assessment in a New Zealand context.

PART 2: MOVEMENT TYPES AND PROCESSES

Provides an understanding landslide types and processes. This is vital for hazard anticipation, risk assessment and mitigation.

PART 3: RECOGNITION AND IDENTIFICATION

This section discusses common surface features associated with landslide movement and available techniques for recognising and identifying landslides

PART 4: LANDSLIDE INVESTIGATION METHODS

This section covers subsurface investigations and monitoring techniques to complement surface observations.

PART 5: ENGINEERING GEOLOGICAL MODELS

The Engineering Geological Model serves as the foundation for hazard and risk assessment, slope stability modelling and mitigation design.

PART 6: RISK ANALYSIS

Risk analysis considers the probability of occurrence of a landslide hazard and the consequences should that hazard occur. It provides stakeholders with a systematic understanding of landslide impact potential, helps identify management actions and facilitates communication with non-technical stakeholders.

Special Landslide Types

PART 8: ROCKFALL

Part 8 provides a description of the site assessment and aspects of rockfall modelling, to allow assessment of rockfall hazard and design parameters.

PART 9: DEBRIS FLOW

Part 9 provides an overview of elements that should be considered in an assessment of debris flow hazards. It provides a description of the site assessment to allow determination of the debris flow velocity, height and runout distance for risk assessment and mitigation design purposes.

PART 7: SLOPE STABILITY MODELLING

This section focusses on stability modelling of soil and heavily jointed rock slopes under static and seismic conditions. It discusses soil strength, pore water pressures, methods, conditions, and details of slope stability analysis as well as back analysis and probabilistic assessment.

PART 10: MITIGATION AND DESIGN PRINCIPLES

The design process for mitigating instability typically involves identifying and selecting mitigation measures that meet stakeholder requirements including cost, effectiveness, and environmental impact.

PART 11: EMERGENCY RESPONSE

Provides an overview of activities that could be required following a natural disaster, including initial response / rapid impact assessment, infrastructure assessments, Rapid Building Assessment (RBA), RBA placard re-assessments and interim use evaluation, as well as detailed damage evaluation and insurance assessments.

PART 1

INTRODUCTION

Abbotsford Landslide, Dunedin, 1979

Tahunanui Landslide, Nelson, 1929

PART 1
INTRODUCTION

1 INTRODUCTION

New Zealand is highly vulnerable to natural hazard risks. It is one of the most vulnerable economies in the world to the impact of natural disaster as a percentage of GDP and, at the time of writing, is ranked third most vulnerable of 42 countries after Bangladesh and Chile[1]. As described in Rosser et al (2017), since 1843 there have been at least 600 deaths (and perhaps up to 1800) from landslides in New Zealand, compared with 458 from earthquakes, and a lower estimate of the national annual cost associated with landslides is NZ $250–$300 M/year[2]. However, they remain a 'forgotten natural hazard' (Rosser et al, 2017) compared to earthquakes and volcanos.

While landslides are a natural geological process, they become increasingly significant when infrastructure development either encroaches on unstable land or destabilises otherwise stable land.

Residential and other infrastructure in New Zealand frequently occurs in steeper landscapes, especially in urban areas, due to the lack of level space for development or to preserve productive land. These areas are vulnerable to issues associated with slope instability, either naturally inherent or created by human development and activity. Slope stability issues that may be already evident under ambient conditions can be expected to be exacerbated and widespread in cyclones or severe storms as well as seismic events and may be compounded by the effects of Climate Change.

As our world changes, there is an increasing need for thorough and robust assessment of slope instability hazards to help manage risks to society and the environment. As New Zealanders, this forms part of our Kaitiakitanga (or stewardship) of the environment.

1 Lloyd's Global Underinsurance Report compiled by the Centre for Economics and Business Research Ltd, October 2012.

2 $300 - $370 million in 2023

FIGURE 1.1. **Golds Landslide, Wellington to Petone shoreline; following the 1855 Wairarapa Earthquake** ***[Gold, Charles Emilius] 1809-1871: Landslip caused by earthquake near Wellington N. Zealand Jan 1855. Ref: B-103-016. Alexander Turnbull Library, Wellington, New Zealand. / records/22330780)***

2 PURPOSE AND CONTEXT OF THE SLOPE STABILITY GUIDELINES

These Slope Stability Guidelines summarise current good practice in geotechnical engineering with a focus on New Zealand conditions, regulatory framework, and established methodologies. The purpose is to provide technical and practical guidance to geoprofessionals (engineers, engineering geologists, and other professionals involved in assessing and managing the stability of slopes) in a New Zealand context. The guidance document helps to ensure that slope stability assessments are performed in a **competent** manner, using established good practices and current technical knowledge. Almost all the content of Unit 1 is derived from existing literature, and sources are referred to as appropriate so that users can refer to the original publications.

These guidelines are not intended to be a book of prescriptive rules—users are assumed to be qualified, practicing geotechnical professionals with sufficient experience and knowledge to apply professional judgement in interpreting and applying the recommendations that they contain.

3 UNIT 1 SCOPE AND STRUCTURE

Unit 1 is intended as an overarching document with a series of future sub-units planned which will provide further details on critical aspects. Unit 1 provides overview recommendations with its broad structure outlined as follows:

PART 2 – SLOPE MOVEMENT TYPES AND PROCESSES

Including:

- Landslide Classification: the modified Varnes classification is suggested as the systematic basis for classifying landslides, however the term given to a particular landslide is from the perspective of the investigator.
- Types of Material: a suggested terminology is provided, based on IAEG Commission 37 with some modification to reflect the NZGS Soil and Rock description guidelines.
- A suggested terminology for water condition, both in the displaced mass and in relation to the failure plane. Again, this follows the recommendations of IAEG Commission 37.
- A suggested terminology for landslide velocity and velocity class, which is related to destructive significance.
- Style of movement, and
- State of Activity.

PART 3 – LANDSLIDE RECOGNITION AND IDENTIFICATION

- An introduction to techniques and methods used to undertake an initial assessment of landslide type.
- Assessment of the geotechnical and geomorphological characteristics of the slope instability and surrounding area. A key component here is the identification of landslides based on a holistic assessment of geomorphology, subsurface conditions and historical evidence.
- Applications of remote sensing.
- Recognition of the potential for instability in construction works.

PART 4 – LANDSLIDE INVESTIGATION METHODS

- Surface Survey and Geomorphic Mapping.
- Guidance for Subsurface Investigations, including defining appropriate methods and planning investigations.
- Field Instrumentation, including instrument selection for groundwater and ground displacement monitoring.

PART 5 – THE ENGINEERING GEOLOGICAL MODEL

This closely follows IAEG Commission 25 and includes:

- Conceptual v Observational Model.
- The critical importance of an outside-in view: starting with an understanding of the regional geological and geomorphic conditions and working into the detail that subsurface investigations provide.
- The appropriate level of development of the Engineering Geological Model, which is related to both the geotechnical complexity of the site and the complexity of the ‘project’ (which may involve a new or existing infrastructure, house etc.).
- The assessment of the likelihood of slope failure, frequency and magnitude (termed the Geohazard model) and assigning appropriate material strengths as part of a geotechnical model.

PART 6 – RISK ASSESSMENT

- Qualitative v Quantitative Assessments.
- Estimating Probability of slope failure.
- Frequency/Magnitude Assessment.
- Estimating Vulnerability, in particular acknowledging the critical importance of landslide velocity.

PART 7 – MITIGATION STRATEGIES AND DESIGN

- Considerations in selecting mitigation strategies, which could include results of risk assessment, target performance criteria (which might include a minimum factor of safety, level of acceptable risk or tolerable displacement), cost / benefit analysis, sustainability and trade-offs.
- Mitigation or remedial options to improve the stability of the slope, such as slope stabilisation measures, erosion control, and slope reinforcement.
- Recommendations for monitoring and maintenance of slopes over time to ensure ongoing stability.

PART 8 – DESIGN APPROACHES/MODELLING

- Understanding of the principles of slope stability analysis, including methods of evaluation of the stability of the slope, the determination of factor of safety, and displacement under seismic events.

PART 9 - ROCKFALL

- This part of the guidance focusses on hazard identification and provides some additional information on the assessment of rockfall bounce height and energy for risk assessment and mitigation design purposes. It is intended to be a complementary section to the MBIE passive design guidance.

PART 10 - DEBRIS FLOW

- Steep channel Morphology and Morphometric Assessment.
- Assessment of Debris Flow Hazards (Catchment, Channel and Fan characteristics).
- Estimation of Debris Flow Magnitude, peak discharge, flow height and velocity: assessment of these aspects is required to assess vulnerabilities.

PART 11 - EMERGENCY RESPONSE

- Empathising with people who have suffered trauma.
- Rapid Building Assessment (application and subsequent removal of red or yellow placards).
- Assessments under Section 124 of the Building Act.
- Other assessments that geoprofessionals may be required to be involved with.

3.1 FUTURE UNITS

A series of future units are proposed, but have not yet been developed, that will complement Unit 1. Units 2 – 8 (provisionally) will provide greater detail on certain elements as follows:

Unit 2	Landslide Recognition, Identification and Field Investigations
Unit 3	Slope Stability Analysis (in more detail, including worked examples)
Unit 4	Mitigation Strategies (in more detail, including vegetation effects)
Unit 5	Rockfall (assessment and analysis) to complement the existing Guidance on Passive Rockfall Protection Works
Unit 6	Debris Flow (assessment, analysis and mitigation)
Unit 7	Special Cases and Materials

Note that the documents that will ultimately form the completed slope stability guideline are referred to throughout as *Units*. This is to provide a differentiator from the existing Earthquake Engineering Series *Modules*.

3.2 CASE HISTORIES

It is also intended that a series of case histories be developed on some of the more notable landslides that have occurred historically in New Zealand. When produced, these documents will sit outside of the Guidance Units as stand-alone documents.

4 AUDIENCE

This Slope Stability Guidance document is primarily aimed at:

Engineering Geologists: Professionals who specialise in the application of geological principles and knowledge to assess and manage geological and geotechnical risks in engineering and infrastructure projects, as well as aspects of land-use planning, identifying and mitigating geological hazards.

Hydrogeologists: Professionals who specialise in understanding the interaction of groundwater with soil and rock (the geology).

Geotechnical Engineers: Professionals whose primary focus is on understanding the behaviour of soil, rock, and groundwater and using this knowledge to provide engineering solutions that ensure the safe and stable design, construction, and maintenance of infrastructure.

The collective term used throughout the guidance for these three groups is 'Geoprofessional'. There is a large overlap between engineering geology, hydrogeology and geotechnical engineering. In simple terms, engineering geologists identify and define a problem, then geotechnical engineers design the solution. In reality, all three disciplines must work together to fully identify, assess and analyse the ground conditions so that suitable mitigations can be implemented and maintained.

Other professionals who may benefit from the Slope Stability Guidance include:

Civil Engineers: Engineers involved in the design and construction of structures, roads, bridges, and other infrastructure projects that may impact slopes.

Environmental Engineers: Engineers who work on environmental issues and challenges, including those related to slope stability and erosion control.

Land-use Planners: Professionals involved in land-use planning policy and consenting, including for the management of slopes and the potential risks they pose to human life and property.

Regulators: Government agencies and other organisations responsible for enforcing regulations and standards related to slope stability and soil erosion.

The guidance document provides technical and practical information for these groups to assist them in making informed decisions regarding the stability and safety of slopes.

5 EXISTING ACTS, CODES, STANDARDS AND OTHER DOCUMENTS

The overarching philosophy of the Slope Stability Guidance is to provide information to geoprofessionals charged with managing slope instability risk to one or more parties. The following are documents with which practitioners may need to have familiarity; not all of the work undertaken by practitioners is to meet regulatory requirements or is covered in legislation, and other codes and standards will need to be considered, dependent on the project.

Acts

1. Building Act 2004 (in particular Sections 72, 124).
2. Resource Management Act (RMA) (in particular Section 106). The Spatial Planning Act, the Natural and Built Environment Act and the Climate Adaptation Act will eventually replace the RMA. There will be a transition period of approximately 10 years before the RMA is replaced by the three new Acts.
3. Earthquake Commission Act (1993). This has a specific focus in regard to insurable land and residential buildings and does not provide minimum acceptable standards for slope stability assessment per se. However, many practitioners undertake assessments under the provisions of this Act over their professional careers and should be aware of its specific requirements for landslide ('landslip') assessment. The EQC Act will be replaced in July 2024 with the Natural Hazards Insurance Act 2023 which will refer to 'landslide'.

Codes and Standards

1. NZS 4431:2022 Engineered Fill Construction for lightweight structures
2. NZS 3604:2011 Timber Framed Structures
3. NZS 4404:2010 Land Development and Subdivision Infrastructure - Part 5: Earthworks and localisations
4. AS/NZS 1170.0:2002 Structural Design Actions Part 0: General Principles
5. AS/NZS 1170.1:2002 Structural Design Actions Part 1: Permanent, imposed and other actions
6. AS/NZS 1170.1:2002 Structural Design Actions Part 5: Earthquake actions

Guidance Documents for Specific Purposes or Clients

1. WorkSafe New Zealand. (2015). Health and Safety at Opencast Mines, Alluvial Mines and Quarries
2. NZSOLD (2015). New Zealand Dam Safety Guidelines
3. Waka Kotahi/New Zealand Transport Agency (2013). Bridge Manual -3rd edition
4. Department of Conservation (2020). Guidelines for natural hazard risk analysis
    a. Part 1 – Risk analysis framework
    b. Part 2 – Preliminary hazard and exposure analysis for landslides
    c. Part 3 – Analysing landslide risk to point and linear sites
    d. Part 4 – Commentary on analysing landslide risk to point and linear sites

Good Practice Guidance Documents

1. IAEG Commission 25 – Engineering Geological Models
2. IAEG Commission 37 (in prep) – Landslide Nomenclature
3. AGS 2007 Guidelines for Landslide Risk Management, which comprise:
    a. AGS (2007a). Guideline for Landslide Susceptibility, Hazard and Risk Zoning for Land Use Management.
    b. AGS (2007b). Commentary on Guideline for Landslide Susceptibility, Hazard and Risk Zoning for Land Use Management
    c. AGS (2007c). Practice Note Guidelines for Landslide Risk Management
    d. AGS (2007d). Commentary on Practice Note Guidelines for Landslide Risk Management.
    e. AGS (2007e). Australian GeoGuides for Slope Management and Maintenance.

These documents are referred to within specific parts of Unit 1 (and subsequent units) as required.

5.1 INTERFACE WITH OTHER GUIDANCE DOCUMENTS

Earthquake Engineering Series

- Module 1 – Module 1 introduces the subject of earthquake geotechnical engineering, provides context within the building regulatory framework, and provides guidance for estimating ground motion parameters for geotechnical design. Part 8 of the Slope Stability Guidance provides a summary of how ground motions may be applied for slope stability assessment.
- Module 2 – This module relates specifically to earthquake site investigation. While this is useful, more emphasis is placed in the Slope Instability Guidance on the requirement for field mapping, as well as remote sensing and displacement monitoring.
- Module 6 – This module relates specifically to earthquake-resistant retaining wall design. Performance objectives as described in Table 4.1 remain relevant for cut slopes; elements in relation to design peak ground acceleration may be relevant in some cases to slope stability assessment.

- ‘Module 7’ – As described in Section 4.4 of Module 1, Module 7 was proposed to be in relation to the seismic assessment of slopes. This module is now wrapped into the Slope Stability Guidance. Elements of seismic slope assessment are discussed in Part 8 of Unit 1 and will be expanded on under proposed Unit 4.

Rockfall – Design considerations for Passive Protection Measures
This guidance was released in 2016 in relation to the design of rockfall protection measures. The guidance provided in Part 9 of Unit 1 and in proposed Unit 5 focuses more on the assessment of rockfall hazard, including hazard identification, assessment of design bounce height and energy, and the consequences these may have to people and infrastructure. This information is intended to be complementary to the earlier guidance such that design criteria for passive structures are more critically assessed.

Landslide Planning Guidance
The draft Landslide Planning Guidance – Reducing Landslide Risk through Land-Use Planning, (De Vilder et al, 2023), sets out how landslide risk can be reduced through consistent land-use planning practices and approaches. Its primary focus is for planning, policy and building compliance staff to help make informed decisions on how the land’s suitability for development can be determined and measures to mitigate, reduce or avoid the effects of identified landslides.

Unit 1 is complementary to the Landslide Planning Guidance – and provides technical detail to inform robust assessment of landslide risk, and to develop appropriate mitigation strategies.

6 REFERENCES

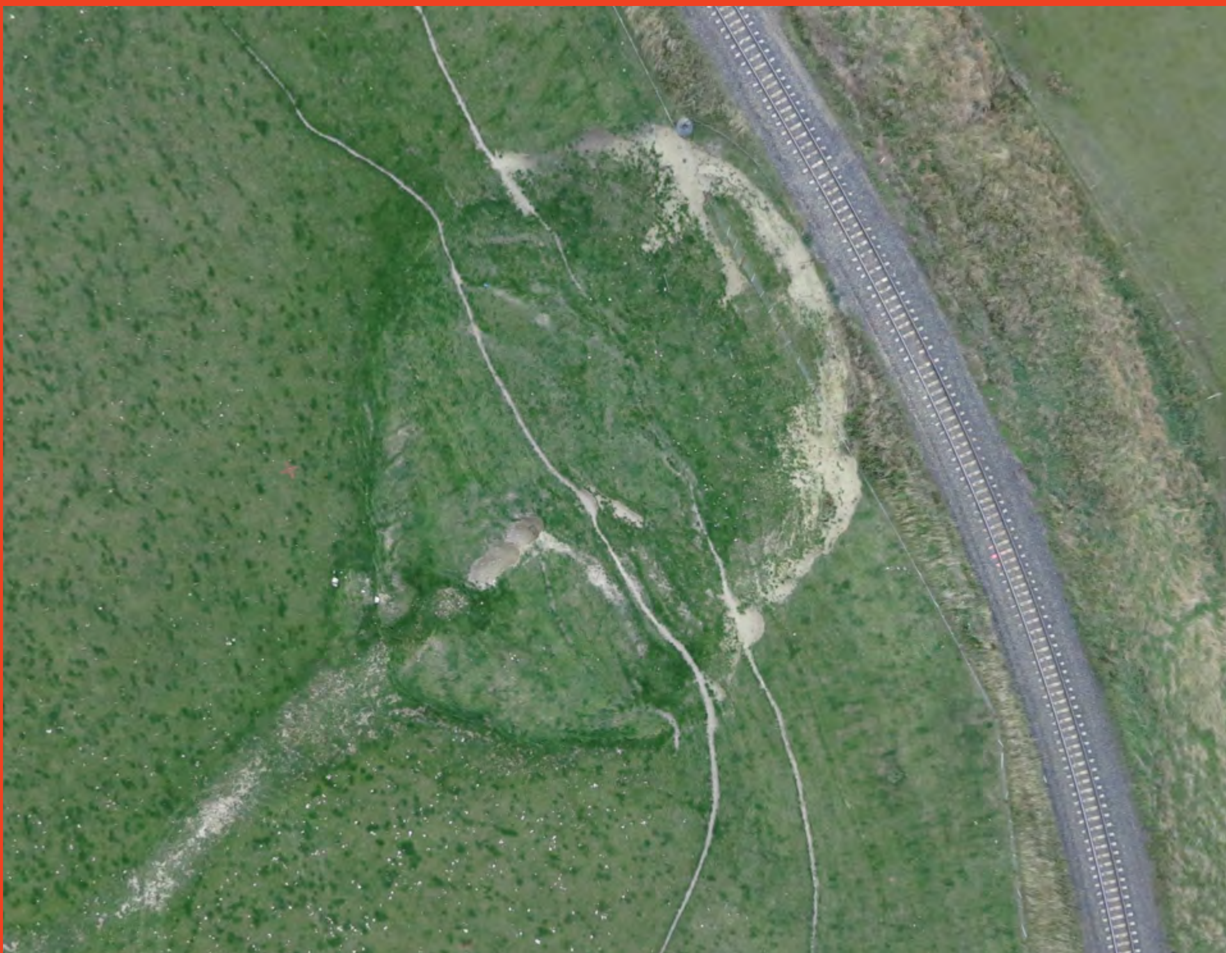
de Vilder, SJ, Kelly, SD, Buxton, R, Allan, S, Glassey PJ (2023). *Landslide Planning Guidance: reducing landslide risk through land-use planning.* GNS Science misc series 144 draft.

Ministry of Business, Innovation & Employment (2016). *Rockfall: Design considerations for passive protection structures.*

Rosser, B, Dellow, S, Haubrock, S, Glassey, P (2017). *New Zealand’s National Landslide Database* Landslides DOI 10.1007/s10346-017-0843-6, May 2017.

PART 2

SLOPE MOVEMENT TYPES AND PROCESSES



Rotational soil slide below railtrack, north of Dunedin

PART 2 – SLOPE MOVEMENT TYPES AND PROCESSES

1 INTRODUCTION

1.1 BACKGROUND

Landsliding is a dynamic process that shapes the earth's surface, constantly reshaping landscapes and influencing the development of various landforms. Landslide movement can be slow and imperceptible, occurring over extended periods (decades or centuries), or it can be sudden and catastrophic.
Understanding different landslide types and processes is crucial to enable geoprofessionals to anticipate potential hazards, assess risks, and develop appropriate mitigation strategies.

THE UBIQUITOUS TERM "SLIP"

It is very common in New Zealand to use the term "Slip" in place of "Landslide", and the term "Slippage" is included within the definition of Natural Hazard in Section 71(3) of the Building Act (2004).

As described by Balance (2009), the term "slip" conceals the fact that several processes may be involved.

While the word "Slip" has widespread usage in non-technical language, it should be avoided as a geotechnical term.

Part 2 of Unit 1 outlines some of the most common types of landslide, types of material, rates and style of movement that should be considered in any assessment of landslide hazard.

The Varnes system of landslide classification, developed in 1978, has become the most widely used system and forms the basis for the recommended system in this guidance (refer Section 2 below). IAEG Commission 37, Landslide Nomenclature, continues to use the Varnes system as the basis for landslide classification systems. At the time of writing, Commission 37 had not released its final documentation, and there is still a possibility that the preferred nomenclature may be subject to some change.

This part of the guidance largely draws on Cruden & Varnes (1996) with some updates suggested by Hungr et al (2014) and IAEG Commission 37.

1.2 DEFINITIONS

Slope Movement is a synonymous term with 'Landslide'. The term Landslide is defined by Cruden and Varnes (1996) as:

The movement of a mass of rock, debris or earth down a slope.

The Natural Hazards Insurance Act 2023, replacing the Earthquake Commission Act (1993) defines a 'landslide' as follows:

...movement (whether by way of 1 or more of falling, sliding, or flowing) of ground-forming materials (being 1 or more of natural rock, soil or artificial fill) that before they moved, formed an integral part of the ground; ...

Massey et al (2019) note

... the history of a mass movement (landslide) comprises pre-failure deformations, failure itself and post-failure displacements. Many landslides exhibit a number of movement episodes, separated by long or short periods of relative quiescence.

Here, the term 'Failure' as described by Hungr et al (2014) is

... the single most significant movement episode in the known or anticipated history of a landslide, which usually involves the first formation of a fully developed rupture surface as a displacement or strain discontinuity (discrete or distributed in a zone of finite thickness)

With this definition, 'failure' describes a critical part of the movement process of a landslide and not the landslide itself.

2 LANDSLIDE CLASSIFICATION

The recommended classification system for the New Zealand context is provided in Figure 2.1.

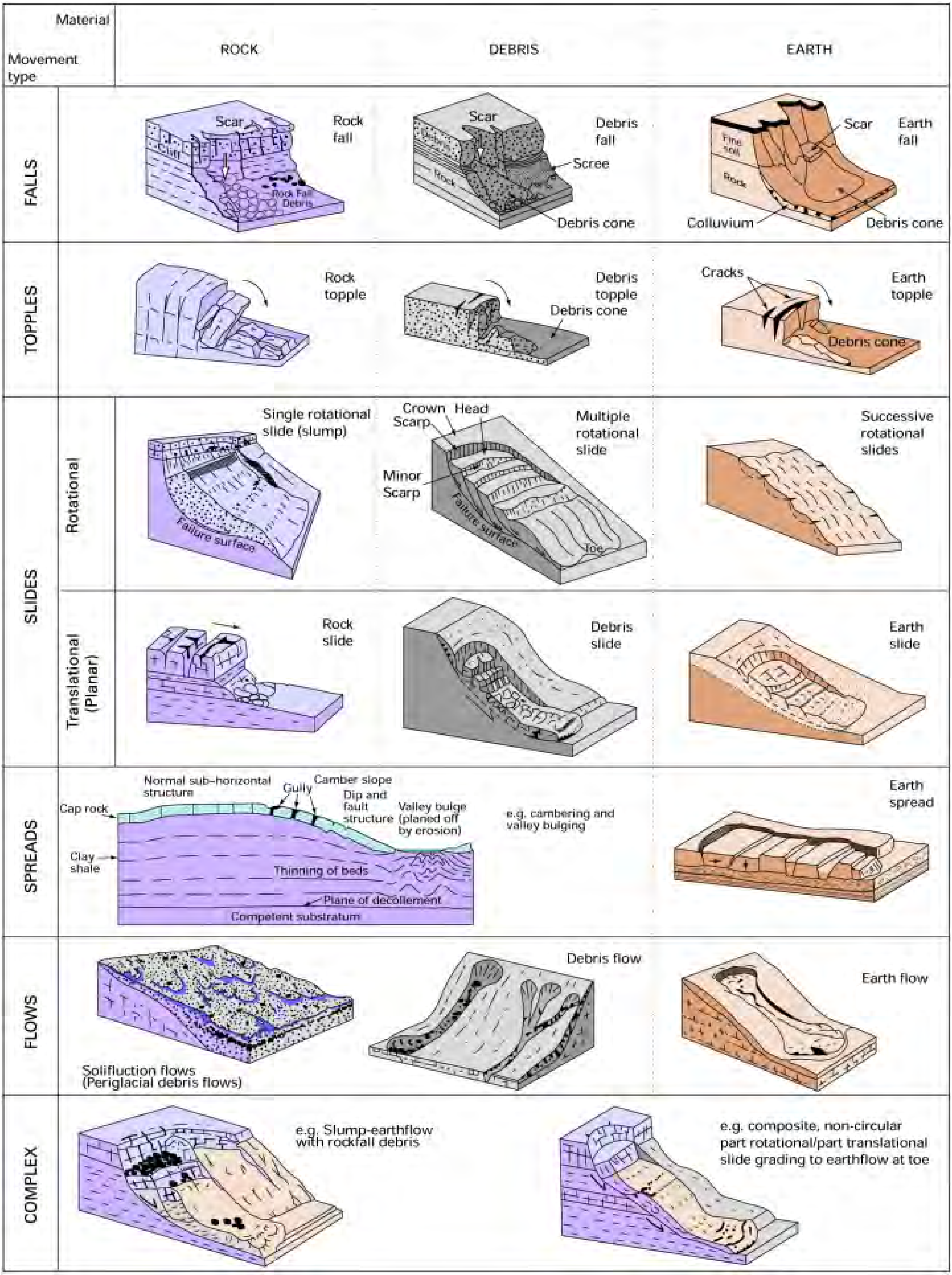


FIGURE 2.1 Landslide Classification System (modifed from Bazin et al, 2012, based on the Varnes classification)

It is important to note that many landslides exhibit a range of different processes, which may vary over distance from source area, and over time. One example is outlined in Hungr et al (2014):

A landslide may begin with slow pre-failure deformation and cracking of surficial soil on a steep hillside. Then a shallow sliding failure develops. The landslide mass accelerates, disintegrates, enlarges through entrainment and becomes a flow-like debris avalanche. The avalanche enters a drainage channel, entrains water and more saturated soil and turns into a surging flow of debris. On entering a deposition fan, the flow drops the coarsest fractions and continues as a sediment-laden flood. This is a complex process. Yet, it is a common one and we should be able to apply the simple traditional term "debris flow" to the whole scenario.

An example of a landslide exhibiting a range of different processes is the 2023 Tahekeroa Landslide in Northland, shown in Figure 2.2. Rather than describe the Tahekeroa Landslide as a "Slide / avalanche / debris flow / flood", which would be over complicated, it is suggested that the dominant terms are assigned reflecting the key mechanisms at the point of concern. In this example, while there is an obvious rotational slide in the upper part of the slope, the term 'debris flow' or 'earth flow' is likely to be most appropriate, as the focus of most assessments would be the impacted road and rail located towards the base of the slope.

A combination of two or more of the above movement or material types, could be termed a "Complex", if the mechanisms critically describe the landslide, for example, Complex Rock Slide/Debris Flow.

FIGURE 2.2. 2023 Tahekeroa Landslide, Northland (courtesy of GNS)

FIGURE 2.3. A view of Buller Gorge showing rock fall. Jones, Frederick Nelson, 1881-1962: Negatives of the Nelson district. Ref: 1/2-070753-G. Alexander Turnbull Library, Wellington, New Zealand. /records/22680450)

A brief description of common landslide types is provided below, with some examples, mostly from New Zealand. Poisel and Preh (2008) provide additional discussion of landslide detachment mechanisms that is useful supplementary reading.

2.1 FALLS AND AVALANCHES

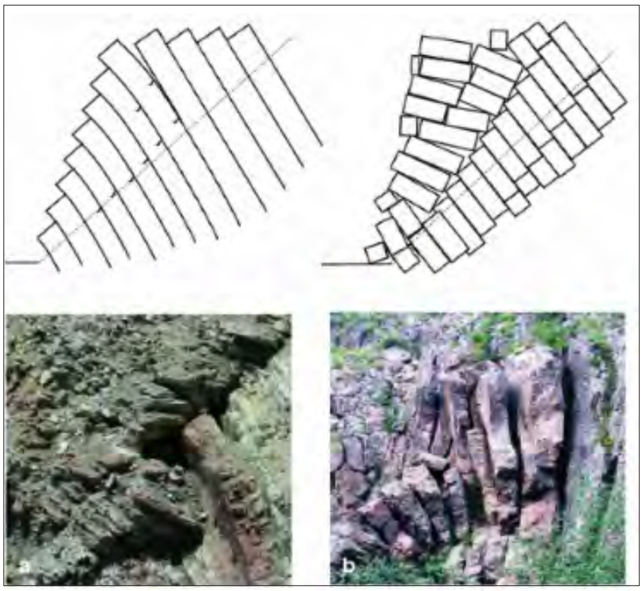
Falls consist of rapid movement of material that becomes detached from steep slopes or cliffs. Separation occurs along discontinuities such as fractures, joints, and bedding planes, while the movement occurs by free-fall, bouncing, and rolling. Falls are usually of a small volume (a few individual blocks).

Avalanches are falls of material where a volume of material cascades down a slope (Figure 2.4). The term is an alternative to 'fall' to be used where it better describes the characteristics of the mass movement. It is typically used to describe larger volumes of failure moving as a slide or flow with complex interparticle action during downslope travel.

FIGURE 2.4. Debris Avalanche, Kaikoura Coast following the 2018 Earthquake, (Source NCTIR)

2.2 TOPPLES

Toppling failures can occur in slopes cut in rock with regularly spaced fractures or bedding planes that strike parallel or subparallel to the slope, and dip into the face. This contrasts with sliding failures which occur when the geological structure dips out of the face. Toppling may be either **Flexural**, where the rock mass deforms in a plastic fashion by bending and forward rotation on closely spaced defects, or **Block**, where the stability of thicker, rigid rock blocks is governed by defect orientations (see Figure 2.5).



(a) (b)

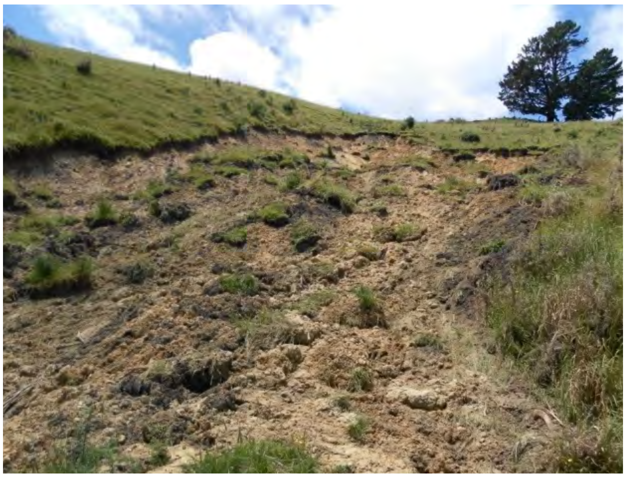
FIGURE 2.5. Rock Topple: (a) Flexural, (b) Block. Amini et al (2012)

2.3 SLIDES

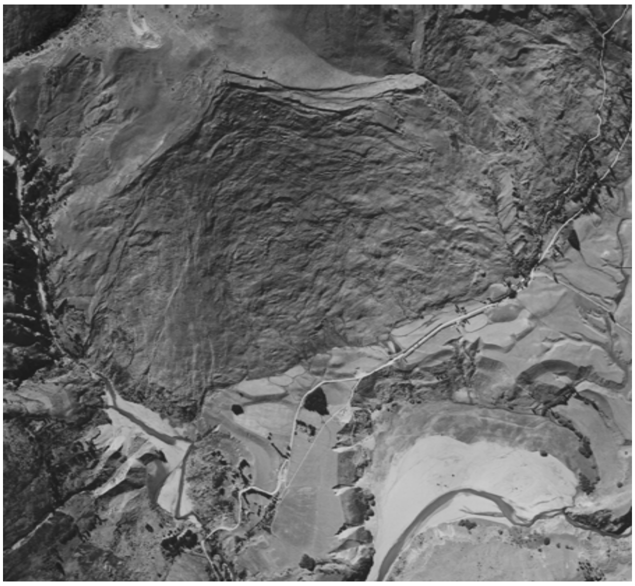
Slides are mass movements occurring on a surface of rupture or a distinct zone of intense shear strain (the basal shear surface) that separates the sliding mass from the more stable underlying material. The shape of the rupture surface allows slides to be categorised into:

- **Rotational slides** that occur along a curved or spoon-shaped surface. Back- tilting may occur near the scarp of the landslide and there is often a toe of displaced material (Figure 2.6).
- **Translational slides** (Figure 2.7) in which the sliding mass moves along a planar or undulating surface with little rotation or backward tilting. A block slide is a translational slide in which the moving mass consists of a single block, or a set of a few closely related blocks, moving downslope as a relatively coherent mass.

FIGURE 2.6 Rotational Slide, becoming a debris flow downslope



(A)



(B)

FIGURE 2.7 Examples of translational landslides (A) Shallow soil failure, central North Island (B) Mt Dewar Landslide, Arthurs Point, Queenstown (1956 aerial photograph)

FIGURE 2.8. Lateral Spreading adjacent to the Avon River during the 2010 – 2011 Christchurch Earthquake (Source: Rapid Post Disaster Building Usability Assessment Field Guide

2.4 SPREADS

This form of landslide is characterised by lateral extension and accompanied by shear or tensile fractures and usually occurs on very gentle slopes. In soils, failure may be caused by liquefaction of susceptible soils, usually triggered by a strong ground motion from high magnitude earthquake events (Lateral Spread), as occurred in several areas of Christchurch following the 2010 – 2011 Canterbury Earthquake Sequence (Figure 2.8).

When coherent material, either rock or (more likely) soil, overlies liquefiable material, the upper unit may undergo fracturing and extension and may then subside, translate, rotate, disintegrate, or flow.

2.5 FLOWS

Flows typically comprise a gravity driven, fluidised mass movement that involves significant internal distortion of the displaced material. Flows are typically elongate (their length is much greater than width) and may be either confined in a steep stream channel, or unconfined on an open slope.

- **Debris Flows** (Figure 2.9) comprise a very rapid to extremely rapid (0.5 to 20 m/s, 2 to 70 km/hr) surging flow of saturated debris. The sediment concentration of debris flows generally is between 70 and 90% in terms of weight and >50–60% in terms of volume. Debris flows are commonly caused by intense surface-water flow caused by heavy rainfall, leading to the erosion and mobilisation of loose soil or rock in gullies on steep slopes.
- **Earthflows** constitute flow-like movement of 'earth' (Refer Section 3 below). This type of landslide usually moves on moderate slopes, under saturated conditions. Dry sand flows are another example.
- **Lahar:** describes a hot or cold debris flow or debris flood composed principally of volcanic material that flows quickly down the slopes of a volcano.

FIGURE 2.9. Gunns Camp Debris Flow, Fiordland in 2020

FIGURE 2.10. Slope creep (terracettes) in soils overlying bedrock on steep slopes near Paekakariki

2.6 SLOPE DEFORMATION

Creep is an imperceptibly slow, downward movement of soil or rock. Movement is caused by shear stresses sufficient to produce permanent deformation, but too small to produce shear failure.

The two main types of creep failure are:

1. **Shallow (soil) creep** that is evidenced by terracettes roughly along the contour on the ground surface (Figure 2.10). It commonly affects only the top 0.5 m of the soil profile (Selby 1993). The creep rate is commonly a few centimetres/year but can develop into shallow failures (earth flows) as a result of extreme rainfall.
2. **Deep-seated** gravitational slope deformation (DSGSD) is creep of rock masses, typically controlled along bedding, foliation or low angle shear zones (Figure 2.11). Movement rates can be imperceptible without monitoring. Well documented examples are found in the Cromwell Gorge upstream from the Clyde Dam where the downslope creep has locally resulted in toe buckling.

FIGURE 2.11. Deep Seated Gravitational Slope Deformation, Flat Top Hill Landslide, Roxburgh Gorge, Central Otago

3 TYPES OF MATERIAL

The Varnes classification divides landslide material into 'Rock' and engineering 'Soil', the latter being further subdivided into Debris and Earth. This system is adopted in IAEG Commission 37 but with modification to subdivide 'Earth' into cohesive (Silt and Clay) and non-cohesive (Sand) components.

It is recommended that the IAEG Commission 37 system is adopted, with minor modification to include the NZGS Field Description for Soil and Rock, as outlined in Figure 2.12. Note that the terms Sand, Silt and Clay can be used if sufficient information is known about the particle size distribution of the landslide material.

Cruden and Varnes (1996) suggest that the material term for the displaced mass should ideally reflect the material before it was displaced. While this works in some cases, rock fall for example, it does not work in others (for example, an earthflow where the parent material may involve weak sedimentary rock, and entrain other material as it flows downslope). Therefore, judgement should be applied as to the most appropriate material term.

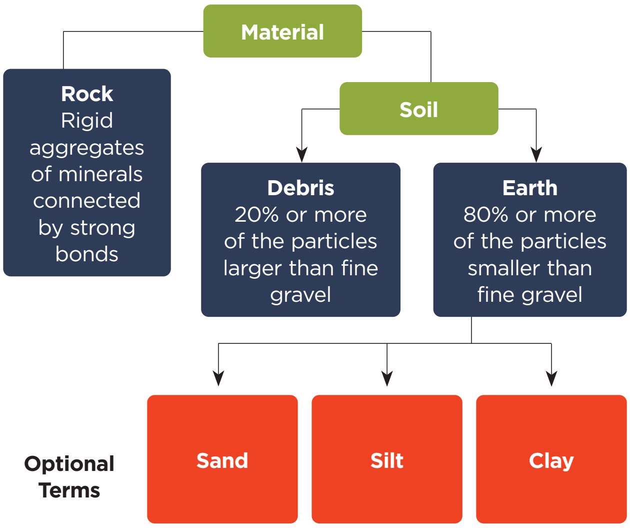


FIGURE 2.12. Proposed Landslide Materials classification (Modified from IAEG Commission 37). Terms 'Sand', 'Silt' and 'Clay' are defined in the NZGS Soil and Rock Description Guide (2005).

4 WATER CONDITION

The water condition of the landslide mass plays a critical role in interpreting the mechanisms of movement. As described in IAEG Commission 37, assessment of the water condition should be made in the field. There are two key aspects that need to be considered:

1. The water condition of the displaced mass
2. The position of the water table in relation to the failure surface, prior to failure (if this can be determined)

4.1 DISPLACED MASS

Descriptors for the water condition of the displaced mass, as recommended in IAEG Commission 37 are provided in Table 2.1.

TABLE 2.1. Water Condition Descriptors (IAEG Commission 37)

WATER CONDITION	DESCRIPTION	PROPORTION OF WATER MAKING UP DISPLACED MATERIAL
Dry (D)	No moisture visible	<5%
Moist (M)	No free water visible; material feels damp to touch	5 – 30%
Wet (W)	The material behaves in part as a liquid, with water flowing from it, and/or supporting bodies of standing water	30 – 50%
Fluidised (Fl)	The material flows as a liquid, discharging water	>50%
Frozen (Fr) (Cold Climate)	Water present as ice	
Thawed (T) (Cold Climate)	Much of the water is in a liquid phase but some ice may still be present	
Steam (St) (Geothermal Environments)	The ground is warm and much or all of the water is present as steam	

4.2 WATER CONDITION OF THE FAILURE PLANE OR ZONE

IAEG Commission 37 indicates that:

A further consideration is the position of the groundwater table in relation to the failure surface prior to failure, where this can be determined from field observations or investigations.

Descriptors for the water condition of the failure plane are provided in Table 2.2.

TABLE 2.2. Failure Plane or Zone Water Condition Descriptors (IAEG Commission 37)

WATER CONDITION	BASAL FAILURE PLANE/ZONE WATER CONDITION
Dry (d)	No moisture visible
Moist (m)	No free water
Wet (w)	Water perched on failure plane
Saturated (s)	Groundwater level above the failure plane

5 LANDSLIDE VELOCITY

The velocity of a landslide is critical in assessing hazard posed by a landslide as described in Part 6 of Unit 1. Rapid landslides may result in loss of life, as there is likely to be insufficient time to evacuate. They also may cause very significant property damage. Slower-moving landslides may not present a threat to life but might affect a significant area and cause significant damage to assets.

Landslide types such as rockfalls, rock and debris avalanches and debris flows are likely to always be rapid. Other landslide types such as rotational and translational slides can be either rapid or slow-moving.

Suggested landslide velocity classes, and associated rates of movement are provided in Figure 2.13.

6 STYLE OF MOVEMENT

In general terms, relatively slowly moving landslide movement can be described in three main stages, as shown on Figure 2.14A.

- Initial displacement, followed by a period of deceleration (A_D – Active but decelerating displacement);
- Steady state or creeping displacement (A_S or C) – involves more or less steady displacement occurring over time; which may (or may not) be followed by
- Accelerated movement (A_A – Active, accelerating displacement) and failure (F)

Alternatively, for a number of landslides, steady state displacement may be followed by

- Suspension (S) – where movement has occurred over the last year, but is currently not moving,

VELOCITY CLASS	DESCRIPTION	VELOCITY (MM/SEC)	TYPICAL VELOCITY	PROBABLE DESTRUCTIVE SIGNIFICANCE
7	Extremely Rapid			Catastrophic damage to buildings impacted by displaced material; Significant risk to life; escape unlikely
		5×10^{3}	5 m/sec	
6	Very Rapid			Significant risk to life; velocity too great to permit all persons to escape
		5×10^{1}	3 m/min	
5	Rapid			Escape evaluation possible; structures; possessions, and equipment destroyed
		5×10^{-1}	1.8 m/hr	
4	Moderate			Some temporary and insensitive structures can be temporarily maintained
		5×10^{-3}	13 m/month	
3	Slow			Remedial construction can be undertaken during movement; insensitive structures can be maintained with frequent maintenance work if total movement is not large during a particular acceleration phase
		5×10^{-5}	1.6 m/year	
2	Very Slow			Some permanent structures undamaged by movement
		5×10^{-7}	15 mm/year	
	Extremely SLOW			Imperceptible without instruments; construction **POSSIBLE WITH PRECAUTIONS**

FIGURE 2.13. Landslide Velocity Classes (modified from AGS 2007)

- Inactive (I), where movement has not occurred within the last year. Inactive landslides can be subdivided into *Dormant*, where the cause of movement is still apparent; *Abandoned*, where the cause of movement is now removed (for example an eroding river changes course away from the toe); *Stabilised*, where artificial remedial measures have stopped movement, or *Relict*, where the landslide has developed under different geomorphic or climatic conditions (Cruden & Varnes, 1996)
- Stick – Slip Movement: Landslides often show different movement patterns over time due to the influence of external factors, such as rainstorm events, earthquakes or changes in vegetation condition. This pattern of movement is called 'stick-slip' and is represented in Figure 2.14(B). Stick-slip movement is characterised by a number of short-term periods of reactivation movement, followed by typically much longer periods of slow or no displacement.

CREEP VS CREEPING

It is common in geotechnical engineering practice to refer to a landslip as 'creeping' if the movement is sufficiently slow. The term 'creep' does not imply anything about the mechanism of movement, rather it simply describes a very slow rate of movement. A potential confusion exists in relation to the use of the term 'creep' as it is also a mechanism of downslope movement which is due to seasonal moisture variation in the near-surface soils. This mechanism is termed 'seasonal creep' and essentially involves a process of expansion normal to the slope during times of high water content (normally the winter months when evapotranspiration is lowest) followed by vertical settlement as the soils dry.

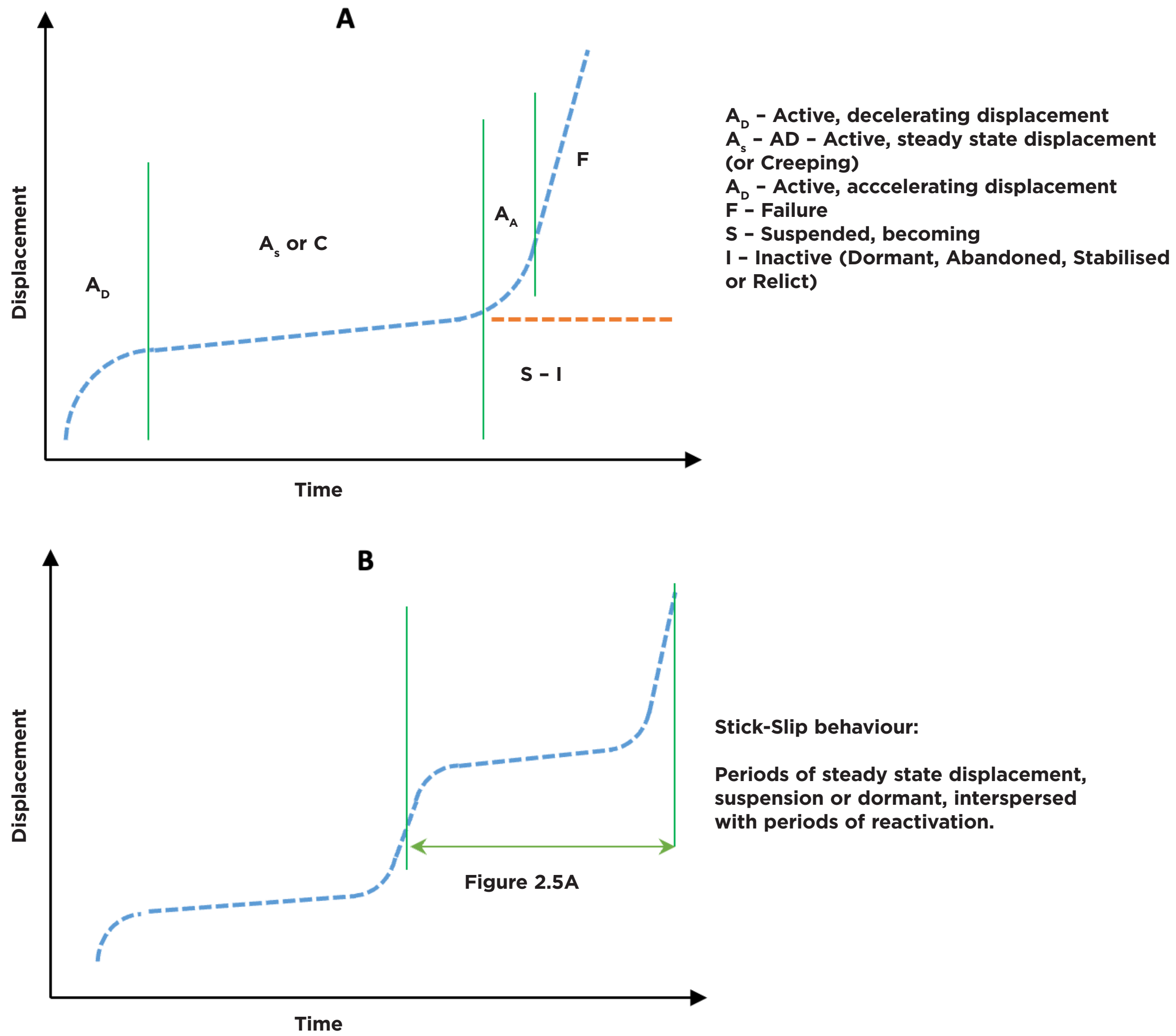


FIGURE 2.14. Landslide Activity States.
(A) Components (B) Stick-Slip behaviour (modified from Cruden and Varnes, 1996)

7 STATE OF ACTIVITY

As movement becomes suspended or dormant, the geomorphology associated with an active landslide becomes subdued. The sharp features become rounded in suspended or dormant states, and with time, pre-existing drainage lines re-form (Figure 2.15). Very old features, which may have been last active several thousand, or tens of thousands of years ago, can be considered ‘Relict’. This concept is important, in that even though the surface morphology becomes subdued, the residual material strengths along the basal shear surface or zone remain. Reactivation may therefore happen should disturbance of the toe area or loading of the head of the landslide (for example, by subdivision earthworks) occur.

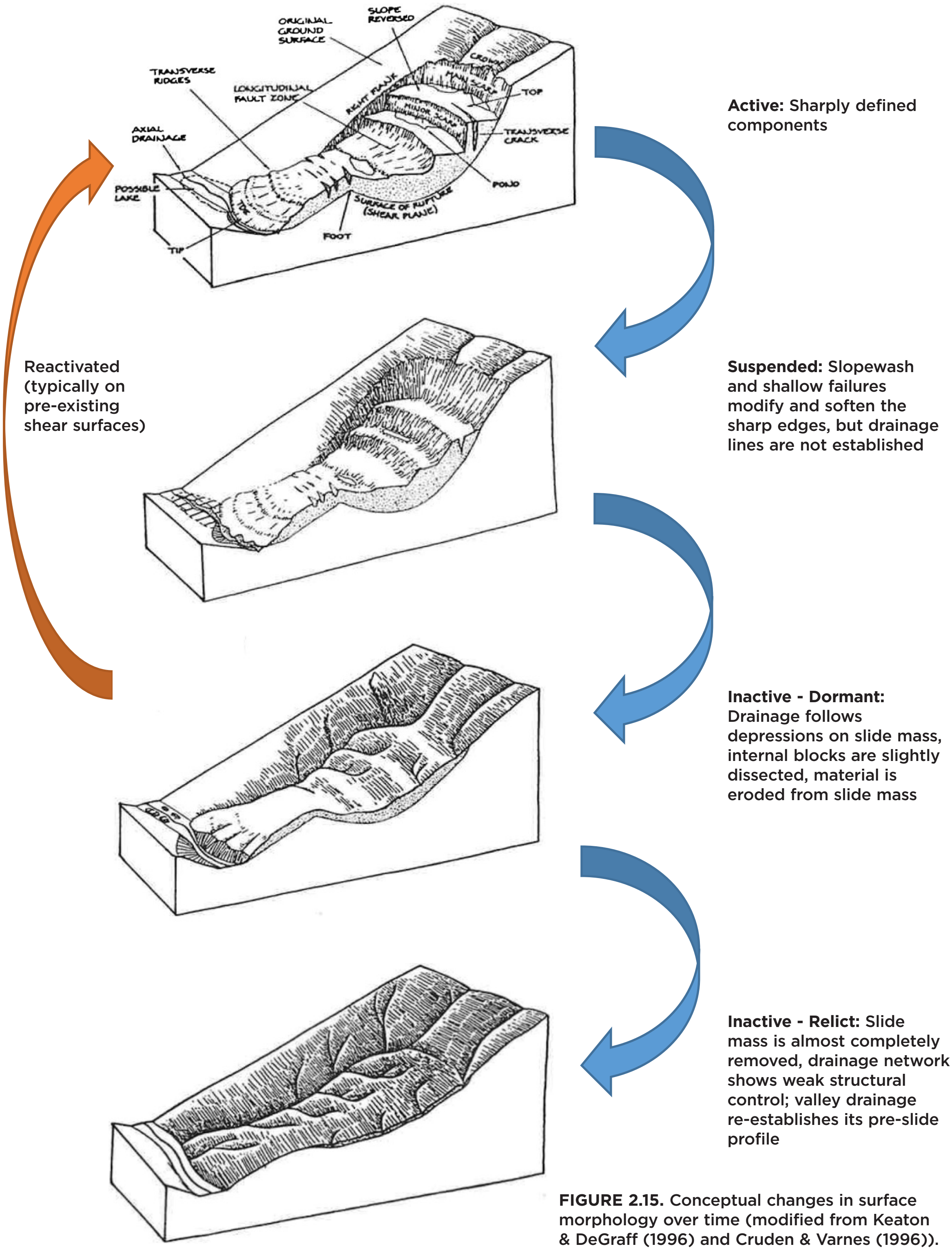


FIGURE 2.15. Conceptual changes in surface morphology over time (modified from Keaton & DeGraff (1996) and Cruden & Varnes (1996)).

FIGURE 2.16. **The Relict Roys Peak Landslide, Lake Wanaka. While the headscarp is relatively clear, the internal geomorphology of the landslide is now very subdued.**

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PART 3

RECOGNITION AND IDENTIFICATION

Broken River Landslide, near Cass, inland Canterbury

PART 3
RECOGNITION AND IDENTIFICATION

1 INTRODUCTION

Any new development on or adjacent to hill slopes must take into consideration the potential for existing and future landslides. Recognition and identification is the first stage of any landslide investigation.

This section describes the surface features that are commonly associated with landslide movement. It also describes some of the commonly available techniques and methods that should be considered to aid in the recognition and identification of landslides.

The techniques and methods used to investigate landslides, particularly subsurface methods, instrumentation and testing are discussed in Part 4 of the guidance. These investigation, testing and monitoring techniques, coupled with methods used to identify and recognise landslides, can then be used to help develop a conceptual engineering geological model which is discussed in Part 5.

GEOLOGICAL SETTING – KNOW YOUR AREA

A sound knowledge of your practising area and geological setting is an important first step in recognising different landslides. The better you know it, the more you will see.

Geology is a governing factor in the type of landslide that can occur. Knowing the local geology is a key way of understanding of the type(s) of landslide that may be present on (or might influence) a particular site.

As examples, slope stability in Wellington is dominated by two principal landslide types: shallow translational slides and debris flows on natural hillslopes of surficial colluvial soils sliding on the underlying Greywacke rock, and shallow rockfalls, planar or wedge failures within slopes cut into the underlying Greywacke rock. In Auckland, deep seated block slides and rotational failures are more common in the Waitemata Group sandstones and siltstones and planar slides with active creep in the extensively sheared Northland Allochthon. A sound knowledge of your practising area and geological setting is an important first step in recognising different landslides. The better you know it, the more you will see.

2 TECHNIQUES AND METHODS

The primary techniques used to determine if there is an actual or potential landslide at the site are:

1. Review published reports and maps
2. Review aerial based imagery
3. Undertake a site walkover and geomorphological mapping (as set out in IAEG Commission 25)

2.1 REVIEW OF PUBLISHED INFORMATION

Published geological maps are useful in determining the underlying geology that will give the geoprofessional the understanding of what type of landslides discussed in Part 2 could be expected. However, in terms of landslide mapping, small scale published geological maps such as the GNS QMAPs rarely show anything other than large landslides, such as the large Quaternary landslides covering many hillslopes in the Marlborough Sounds. 1:50,000 or similar maps have been prepared for some areas and these can be obtained through GNS. These maps may show landslides that the QMAPs don't, but they won't have the site level detail that would normally be required for a project.

Published reports for specific areas have been prepared by GNS following storm events (for example, following the Paekakariki Storm in 2003 or Wanganui Region in 2015) and these may provide additional information on landslides relevant to a project site. In-house consultancy reports may also exist for the proposed site and if available may be held in Council records of previous consent applications. There may also be published papers that contain useful information on the geology or geomorphology of the region or your site or may contain information on slope instability. These may be available online by way of an internet search.

2.2 REVIEW OF AERIAL IMAGERY

The best desk top based technique used to identify landslides is the review of aerial photographs and imagery. Aerial imagery can come in many formats and this include (from easiest to obtain, lowest cost and commonly lowest quality to the more costly and more likely to be obtained specifically for large / complex projects with the highest resolution):

1. Google Earth imagery: This imagery is suitable for an initial identification of landslides within the area of interest but it should not be relied upon for a detailed assessment as the photograph quality typically does not allow subtle detail and important features to be picked up.
2. Publicly available aerial imagery or ground elevation datasets from local councils or other organisations or websites (i.e. Retrolens)
3. Stereographic pair aerial images
4. LANDSAT satellite imagery
5. Site specific DEM/DTM/LiDAR data (refer Section 4 of this part of the guidance)

Aerial images can be used to recognise the distinguishing landslide features discussed in Part 3. They can be used to help identify geological materials that dictate landslide types but more often they are used to identify tell-tale geomorphological features that landslides are present like bowl shaped breaks of slope or scallop shaped escarpments, very steep slopes, hummocky ground or natural ponds.

2.3 SITE WALKOVER

Even if the review of aerial images suggests that no landslide movement has taken place, it is essential that a site walkover is completed to not only challenge the geoprofessional`s conclusions but also to determine whether any proposed work on the site could cause instability.

A site walkover should start by looking at the site from a distance to put it into context with its surroundings. This will also give an indication on whether it has been modified and looks “different” (for example, by way of cuts and fills) compared to its surroundings. Observing a site earlier in the morning or later in the afternoon when the sun is lower in the sky can cast shadows across the site that can make the shape of landforms and landslide features more apparent.

FIGURE 3.1. Hummocky ground on a complex landslide near Pongaroa.
Red arrow = direction of movement

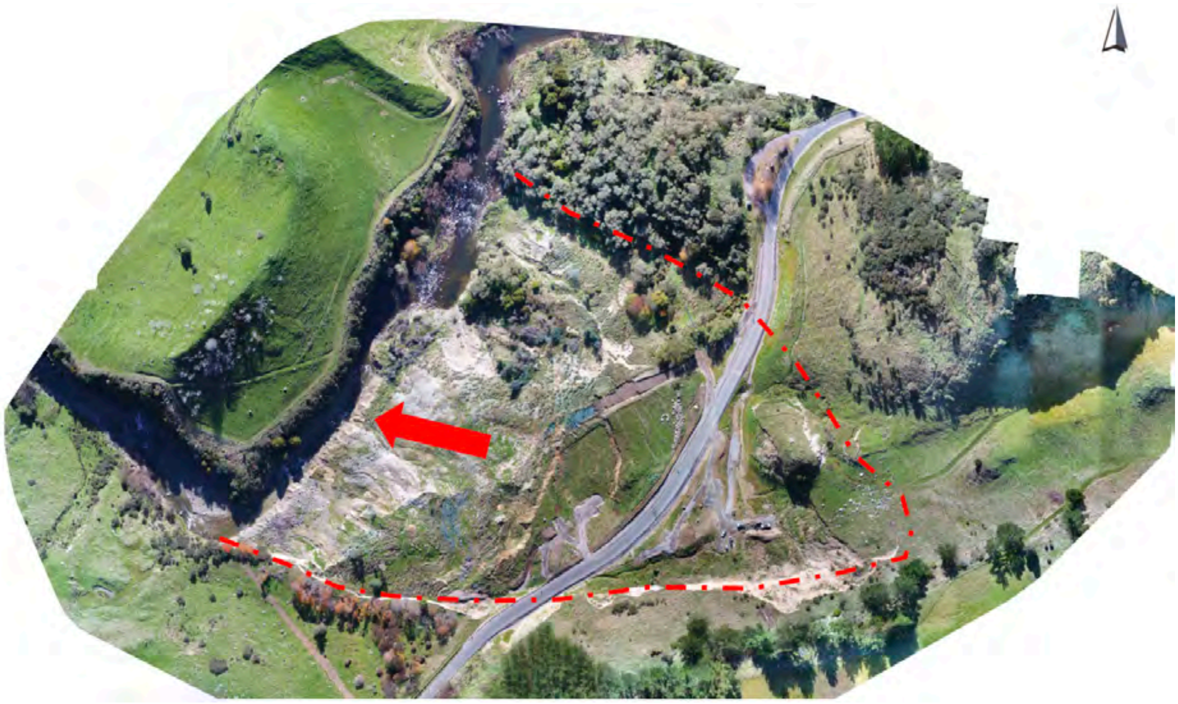


FIGURE 3.2. Te Oreore landslide on SH4 south of Raetihi. The disintegrated old SH4 is on the left of the temporary road alignment. Red arrow = direction of movement

FIGURE 3.3. Displaced gate and fence caused by differential movement within a landslide

FIGURE 3.5. Hummocky ground and rotated trees (in the background, arrowed)

FIGURE 3.4. Displaced fence caused by compressive movement

FIGURE 3.6. Ground stretch around a rigid structure (water bore)

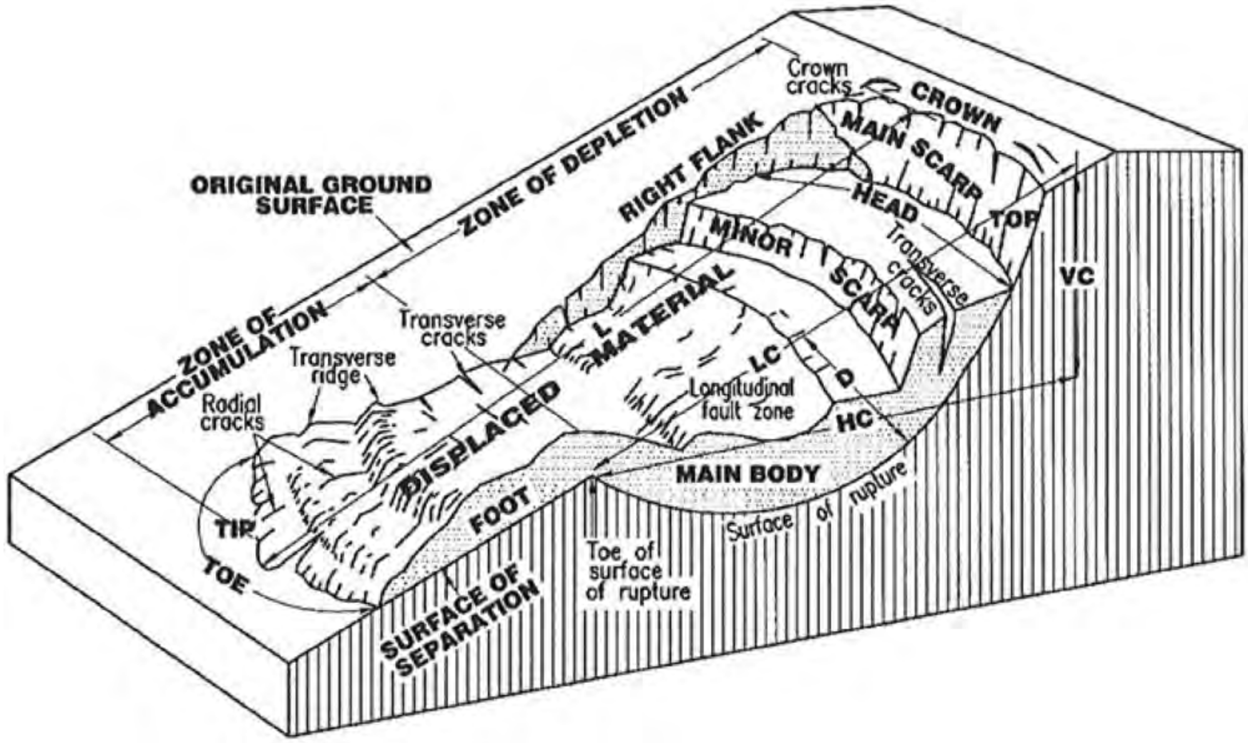


FIGURE 3.7. Idealised Landslide Components (Varnes, 1978). Definitions of the various components are given in Cruden and Varnes (1996).

On closer inspection, the geoprofessional should look for evidence such as:

- ground cracking or scarps
- breaks of slope (concave, convex and cliffs)
- water seepages, standing water, flows or wet ground
- regular small linear ridges indicating shallow soil creep
- small cracks surrounding rigid structures such as fence posts, foundations or power poles which may indicate deeper ground movement
- rotated fences (Figure 3.3 and 3.4), walls, trees or other more rigid features
- hummocky ground or depressions (Figure 3.5)
- steps or back tilting of the ground surface
- broken services
- roadway or pavement settlement
- structures out of alignment, such as buildings, walls, fences or power poles etc.

3 RECOGNISING LANDSLIDES

It is essential to identify the type of landslide that may be present at the project site so that:

- Future patterns and speed of movement can be predicted
- An assessment can be made as to whether the landslide is part of a much larger landslide (for example the Te Oreore landslide on SH4) or whether the observed landslide may further develop beyond its existing extent and movement.
- Effective mitigation or remediation measures can be determined, within project constraints

3.1 LANDSLIDE COMPONENTS

Figure 3.7 is an idealised diagram of a rotational landslide taken from Varnes (1978) which indicates the recommended terminology used to describe parts of a landslide. Depending on the style of movement, not all of these features will be present, as outlined in Table 3.1.

TABLE 3.1. Features that aid in the recognition of active or recently active landslides (adapted from Table 1, Ritchie, 1958)

MECHANISM	MATERIAL	STABLE PARTS SURROUNDING THE SLIDE			PARTS THAT HAVE MOVED			
		Crown	Main Scarp	Flanks	Head	Body	Foot	Toe
Falling: Rock	Rock	Loose rock: probable cracks behind scarp; irregular shape controlled by local joint system	Usually almost vertical; irregular, bare, fresh. Usually consists of joint or fault surfaces	Mostly bare edges of rock	Usually no well defined head. Fallen material forms a heap of rock next to scarp	Irregular surface of jumbled rock, sloping away from scarp. If very large, and if trees or material of contrasting colour are included, the material may show direction of movement radial from scarp. May contain depressions	Foot commonly buried. If visible the foot generally shows evidence of reason for failure, such as underlying weak rock or banks undercut by water	Irregular pile of debris or talus if small. If the rockfall is large, the toe may have a rounded outline and consist of a broad, curved transverse ridge
Soil	Soil	Cracks behind scarp	Nearly vertical, fresh, active, spalling on surface	Often nearly vertical	Usually no well defined head. Fallen material forms a heap of rock next to scarp	Original slump blocks generally broken into smaller masses; longitudinal cracks, pressure ridges, occasional over thrusting. Commonly develops a small pond just above foot	Transverse pressure ridges and cracks commonly develop of the foot; zone of uplift absence of large individual blocks, trees lean downhill	Often a zone of earthflow, lobate form, material rolled over and buried; trees lie flat or at various angles mixed into toe material
Sliding	Soil	Numerous cracks, most of them curved concave toward slide	Steep bare, concave towards slide, commonly high. May show striae and furrows on surface, running from crown to head. Upper part of scarp may be vertical	Striae on flank scarps have strong vertical component near head, strong horizontal component near foot. Height of flank scarp decreases toward foot. Flank of slide may be higher than original ground surface between foot and toe. En echelon cracks outline slide in early stages	Remnants of land surface flatter than original slope or even titled up hill creating depressions at the foot of main scarp in which perimeter ponds form. Transverse cracks, minor scarps, grabens and fault blocks. Attitude and bedding differs from surrounding area. Trees lean uphill.			
	Rock	Cracks tend to follow fracture pattern in original rock	As Above	As Above	As Above	As above, but material does not break up as much or deform plastically	As above	Little or no earthflow. Toe often nearly straight and close to foot. Toe may have steep front
Blockslide	Rock or Soil	Most cracks are nearly vertical and tend to follow contour of slope	Nearly vertical in upper part, nearly plane and gently to steeply inclined in lower part	Flank scarps very low, cracks vertical. Flank cracks usually diverge downhill	Relatively undisturbed. No rotation	Body usually composed of a single or a few units, undisturbed except for common tension cracks. Cracks show little or no vertical displacement	No foot, no zone of uplift	Ploughing or overriding of ground surface

TABLE 3.1. (CONTINUED) Features that aid in the recognition of active or recently active landslides (adapted from Table 1, Ritchie, 1958)

MECHANISM	MATERIAL	STABLE PARTS SURROUNDING THE SLIDE			PARTS THAT HAVE MOVED			
		Crown	Main Scarp	Flanks	Head	Body	Foot	Toe
Rockslide	Rock	Loose rock, cracks between blocks	Usually stepped according to the spacing of joints or bedding planes. Surface irregular in upper part, and gently to steeply inclined in lower part; may be nearly planar or composed of rock chutes	Irregular	Usually blocks of rock	Rough surface of many blocks. Some blocks may be in approximately their original attitude, but lower down, if movement was slow translation	Usually no true foot	Accumulation of rock fragments
Flowing: Dry: Rock Fragment flow	Rock	Same as rockfall	Same as rockfall	Same as rockfall	No head	Irregular surface of jumbles rock fragments sloping down from source region and generally extending far out on valley floor. Shows lobate transverse ridges and valleys.	No foot	Composed of tongues. May override low ridges in valley
Sand Flow	Soil	No cracks	Funnel-shaped at angle of repose	Continuous curve into main scarp	Usually no head	Conical heap of sane, equal in volume to head region	No foot	
Flowing: Wet: Debris avalanche Debris flow	Soil	Few cracks	Upper part typically serrate or V-shaped. Long and narrow Bare commonly straited	Steep, irregular in upper part. Levees may be built up along lower parts of flanks	Typically no obvious head to the landslide	Wet to very wet. Large blocks may be pushed along in a matrix of finer material. Flow lines. Follows drainage lines and can make sharp turns. Very long compared to breadth.	Foot absent or buried in debris	Spreads laterally in lobes. Dry toe may have a steep front a few feet high.
Earthflow	Soil	May be a few cracks	Concave toward slide. In some types scarp in nearly circular, slide issuing through a narrow orifice	Curved, steep sides	Commonly consists of a slump in soil and not in rock	Broken into many small pieces. Wet. Shows flow structure	No foot	Spreading, lobate. See above under "slump".
Sand or silt flow	Soil	Few cracks	Steep, concave toward slide, may be a variety of shapes in outline – nearly straight, gentle arc, circular or bottle-shaped	Commonly flanks converge in direction of movement	Generally under water	Spreads out on underwater floor	No foot	Spreading, lobate

3.1.1 SURFACE CRACKING

Surface cracking is probably the most important evidence for recognising and classifying landslides in the field. The pattern of surface cracking will help the geoprofessional understand whether the landslide is shallow or deep, and within soil or rock, helping to identify the type of landslide discussed in Part 2.

Cracking can vary across an individual landslide and across different types of landslides. It can be obvious or subtle and cracks can indicate many different things.

- Cracking at the head of the landslide is typically normal to landslide movement
- On the sides the cracking is nearly parallel to the direction of movement and may be *en-echelon* (Figure 3.8)
- At the toe the cracks can be both parallel and normal to the direction of movement.

The following sections provide a summary of the main characteristics of typical landslide types and how to recognise them in the field. This is not an exhaustive list and reference should be made to Table 3.1 for more examples.

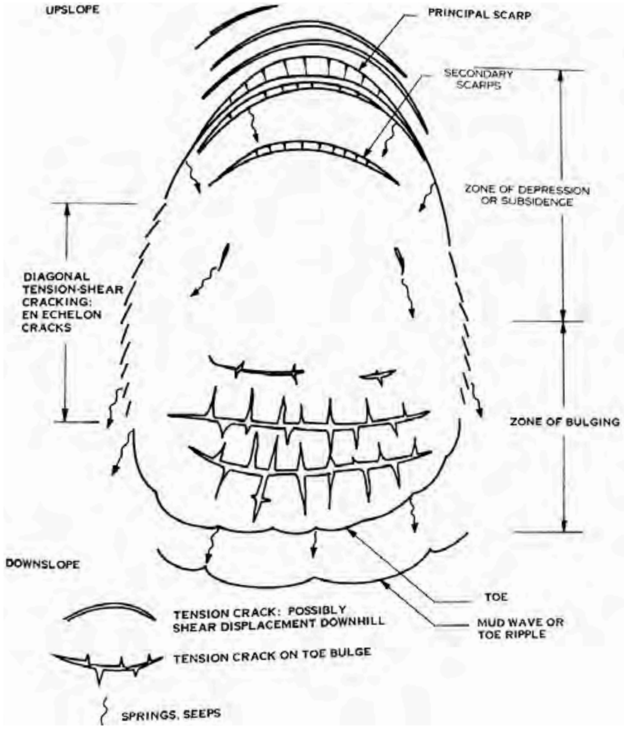


FIGURE 3.8. Example cracking in a soil landslide (taken from Figure 9-14 in Keaton & DeGraff, 1996)

PHOTOGRAPH 3.9. Example of cracking at a landslide head (compare to cracks shown in Figure 3.8)

FIGURE 3.10. Widespread cracking in a large translational block slide at Te Oreore (SH4)

3.2 FALLS

Rock (and soil) falls are characterised by the scattered accumulation of material on the ground surface downslope of the zone of depletion and forming a talus slope at the toe. Most material will accumulate close to the toe of the steep slopes where it has fallen, but some rockfall may come to rest far from its source.

If the rockfall is active or recent the source area is likely to have a fresh surface and sharper edges. The source area and rockfall material will also be free of vegetation. Vegetation damage (snapped or rotated trees or flattened bushes) can be a good indicator of the intensity and severity of a fall.

3.3 SLIDES

Slides can be a little more complex than falls and the shape of these features is dependent on the type of material in which the slide occurs as well as the amount and direction of movement.

Rotational slides are characterised by block rotation whereas translational slides are characterised by lots of lateral displacement but relatively smaller vertical displacement relative to the adjacent ground except where graben structures develop within the blocks (Figure 3.10).

Rotational slides typically occur in finer grained soils (sands, silts and clays) and the head region of the slide is characterised by steep scarps and by visible offsets. Recent activity is indicated by fresh cracks and there may be striations in the soil indicating the direction of movement. Displaced land below the head scarp may also be back rotated (Figure 3.11).

FIGURE 3.11. Simple rotational landslide with head scarp and back rotated block below

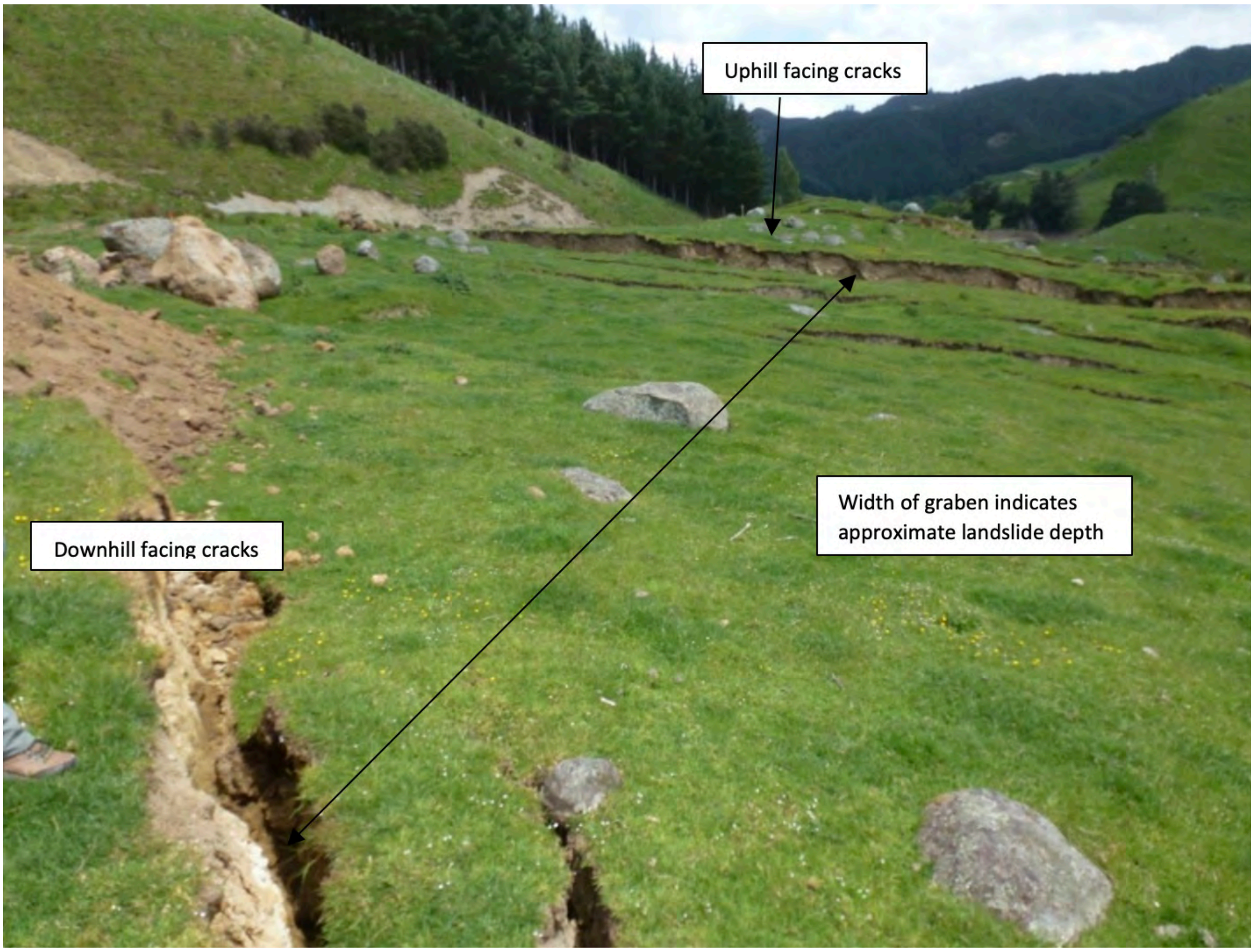


FIGURE 3.12. Cracking defining a graben feature within a large block landslide at Te Oreore. Downslope movement to right.

Ponds and undrained depressions may develop and may occur below the head scarp; this is an indication of grabens which have experienced some rotation.

Cracks that step down but are uphill facing indicate the presence of a graben structure and a deep block slide (Figure 3.12). A simple rule of thumb is the distance between the downhill and uphill facing cracks in a graben gives an indication of its approximate depth.

The toe of the landslide is typically characterised by a zone of compression that results in ground uplift or toe bulging (Figure 3.13). Seeps, springs and marshy conditions can indicate the toe of a slump and trees may tend to tilt downhill rather than uphill like they tend to do near the head of a landslide.

3.4 FLOWS

Flows are very rapid to extremely rapid (5–10 m/s, 15–30 km/hr) surges of saturated debris in a steep channel. The sediment concentration of debris flows generally exceeds 40% and is typically wet rather than dry in New Zealand. They occur in both fine and coarse grained soils, becoming mobilised by an excess of water typically during heavy rainfall events. Flows can travel great distances on relatively modest slope angles.

FIGURE 3.13. Landslide toe with zone of uplift or "toe bulging"

FIGURE 3.14. Earthflow landslide in Hunterville. Note emptying of material at the crown and distance of flow run out downslope.

The headscarp area (or steep creek catchment) comprises the initiating or 'sediment production' area for the particular debris flow or flood. Sediment and debris are typically delivered to the *upper channels* by rock fall, rock slides, debris avalanches, debris flows, slumps and raveling.

Debris flows typically have a *main channel* that comprises the confined transportation zone of the landslide, where sediment bulking via entrainment of bed and bank material typically occurs.

The debris, or *alluvial fan*, represents a depositional landform at the outlet of a steep creek catchment. The alluvial fan comprises the unconfined area of the system and represents the approximate extent of deposition of past flow and flood events.

Debris flow phenomena are discussed in more detail in Part 10 of Unit 1.

4 REMOTE SENSING

4.1 DIGITAL ELEVATION AND HILLSHADE MODELS

Bare Earth Digital Elevation Models (DEM) can provide an excellent means of identifying surface morphology associated with slope instability. As the name suggests, Bare Earth DEMs provide an indication of the ground surface – vegetation and other artifacts have been removed.

In areas where heavy vegetation obscures aerial photography and ground-based mapping is also difficult, hillshade, or greyscale, models derived from the DEM are extremely useful for identifying features which would otherwise not be apparent. An example of this is provided in Figure 3.15.

The most common data source for bare earth DEM is from LiDAR. LiDAR data is publically available for selected areas of the country at https://www.linz.govt.nz/products-services/data/types-linz-data/elevation-data

This dataset has a stated vertical accuracy of ≤20 cm and horizontal accuracy ≤100 cm and a 95% confidence interval.

4.2 SURFACE DEFORMATION VIA INSAR

Satellite based InSAR (Interferometric Synthetic Aperture Radar) provides a useful way of determining rates of movement. InSAR is suitable to measure very slow (16 mm to < 1.6 m/year) to extremely slow (< 16 mm/year) moving landslides. InSAR data is not useful for faster moving landslides, because movement between acquisition dates (12-24 days in NZ) creates a loss of coherence. The application of InSAR to map and monitor slow-moving deformation related to landslides is well documented globally.

Two radar images of the same area collected at different times from similar vantage points in space can be compared against each other. Movement of the ground surface toward or away from the satellite can then be measured, relative to the position of the recording satellite. While this provides a one-dimensional measure of movement, it can allow areas that are moving to be identified. Figure 3.15

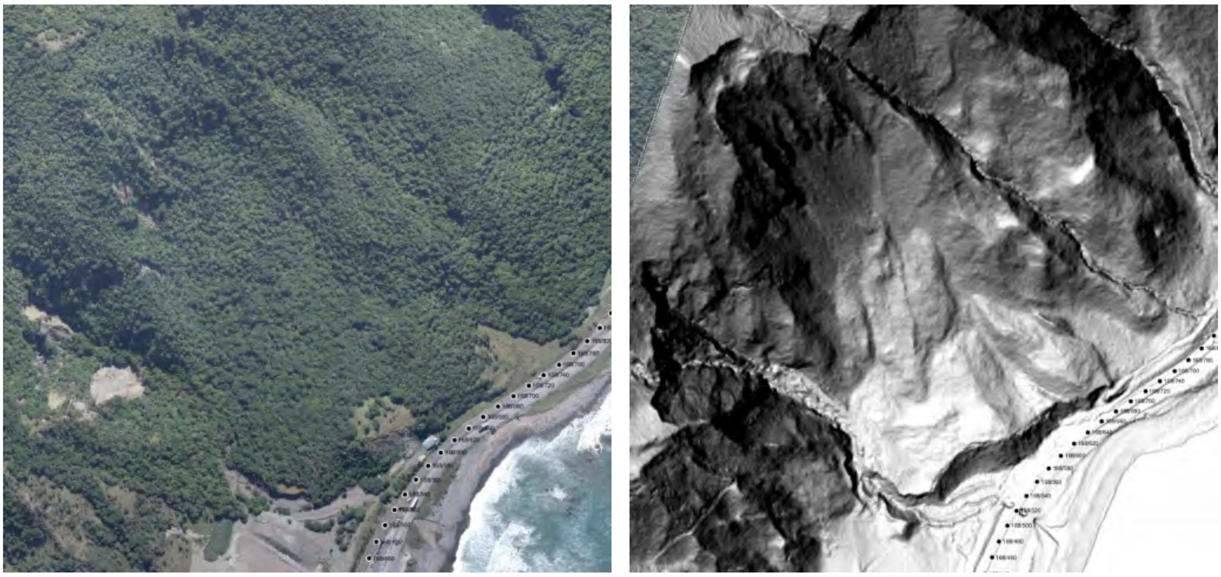


FIGURE 3.15. In the vertical aerial photograph (left hand side), the majority of the surface morphology of this landslide is obscured. However in the hillshade model (right hand side), the morphology of the same landslide is much easier to observe and interpret (source: NCTIR).

shows an example of the measurement point density between low-resolution and high-resolution monitoring approaches; there is an increased probability that areas of movement may be missed with low-resolution data.

Limitations to using InSAR include:

1. the low density of coherent targets over highly vegetated areas;
2. the low sensitivity to slope movements that are perpendicular to the satellite line of sight (e.g. north or south facing slopes);
3. excessively rapid movement such as earthflows and small-scale landslides can lead to complete coherence loss, and these can be challenging to detect depending on the satellite's spatial and temporal resolution.

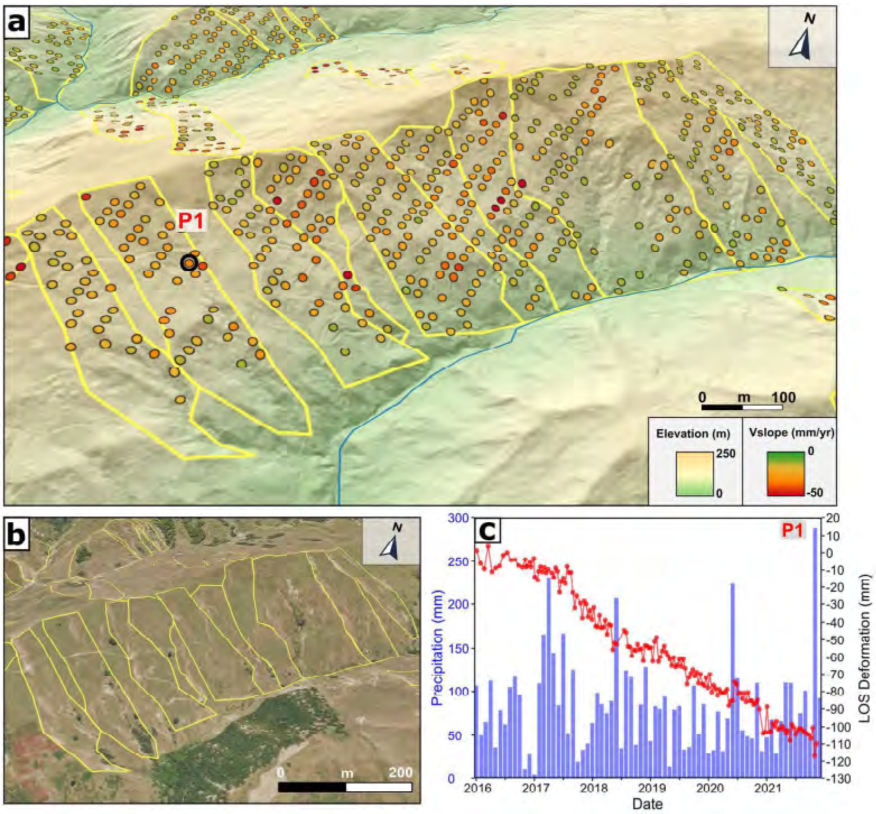


FIGURE 3.16. Example of slope deformation assessment using InSAR. Cook et al (2023)

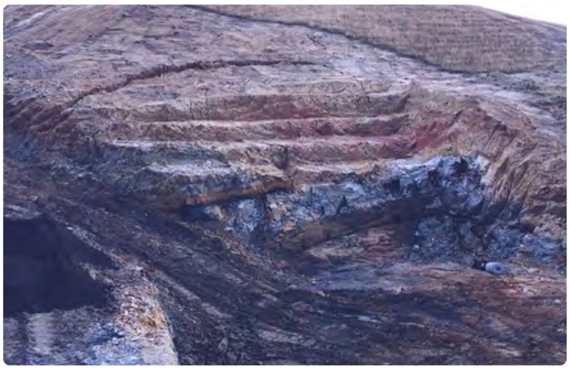
5 RECOGNISING THE POTENTIAL FOR INSTABILITY IN CONSTRUCTION AND DEVELOPMENT WORKS

Every slope has experienced instability at some point in its geological past. As a result, natural slopes typically have a Factor of Safety (FoS) of around 1. The geoprofessional needs to be cognisant of this and that any modifications above, below or to the slope could change this FoS for the better or worse. Questions the geoprofessional should consider are:

1. what type of instability could be expected based on the geoprofessional`s understanding of geology and geomorphology? Is it rotational sliding? a rockfall? deep seated creeping movement? or predominantly surface erosion?
2. how recently and frequently instability occurs? and
3. how sensitive is the slope to landsliding if it is modified?

First-time landslides, or more specifically those slopes that have shown no evidence of instability in living memory, have proven to be deadly in New Zealand (for example, the landslides that caused the fatalities in Auckland and Hawkes Bay in 2023 as a result of storms). Geologically and geomorphologically similar slopes near to or adjacent to the site are a good indicator of whether there is potential for instability within the site area. The geoprofessional must consider whether there are landslides that already exist in the project area: precedence is an indicator that the site may have had, or has, potential for instability.

TABLE 3.2. Construction works that can trigger landslides

CONSTRUCTION WORKS	IMPACT ON STABILITY
Changes to surface water flow paths/drainage by cuts or fills	Altering surface water flow paths may direct surface water to more sensitive parts of the slope with a lower Factor of Safety resulting in an increased likelihood of movement.
Restriction of groundwater flows by fills	Compacted / low permeability fill soils placed over more free draining soils at the toe of the slope can result in a rise in groundwater levels within the landslide if adequate drainage measures are not allowed for.
Overloading of relatively weak underlying soil layers by fill	Where the additional loading increases the existing stress applied to a slope / shear surface within the slope material, so that the ultimate shear strength of the shear surface is exceeded and failure occurs.
Overloading of sloping bedding planes by fill	As above.
Oversteepening of cuts in rock or soil 	Oversteepening the slope can result in the gravitational / sliding forces exceeding the resisting forces within the slope material. **FIGURE 3.17. Block Slide in an excavation into very weak clay-rich sandstone and mudstone, South Auckland. Photo courtesy Warwick Prebble**
Exposure of shear planes or adversely orientated defects in rock cuts	Cuts can expose weak layers in the soil or weak joint / discontinuity surfaces in the rock that were previously not daylighting out the slope face but now are as a result of the cut. Exposure of joints and discontinuities in rocks is common in nearly all sedimentary rocks in New Zealand which exhibit a degree of folding and jointing.
Undercutting of soils or rock, removing toe support and promoting soil sliding along soil / rock interface	A common failure mechanism in the Wellington Region and in Greywacke hillslope geology where cutting slopes exposes the interface between the overlying colluvium soils and the weathered Greywacke Rock.

UNDERSTANDING PAST PRACTICES

The photo below shows excavation practices that were common in the 1930's. Material (in this case, colluvial soil and weathered Greywacke rock) are being cut by hand at probably as steep an angle as possible. Excavated material is then being end-tipped on the outside of the road, with no effort at vegetation stripping benching, or fill compaction.

This common historical practice creates two instability hazards (1) failure of the cut, (2), more critically, failure of the side-cast fill over a decomposed vegetation layer. The potential for historical earthworks to be present at a site, and their effects needs to be carefully understood!

Photograph from Justice et al, (2006)

Any change to a slope by cuts or fills will affect its stability. A well-known example of slope modification contributing to slope instability in a New Zealand context was the Abbotsford Landslide in Dunedin in August 1979 in which instability was inferred to have been aggravated by excavation at the toe of the slope. Therefore, the geoprofessional needs to recognise the potential for landsliding and evaluate the effect that any proposed construction will have on the soil/rock profile, as well as the surface water and groundwater conditions.

Although well considered construction works can have a positive effect on slope stability, by for example, reducing groundwater levels and removing shallow instability, ill-considered works can cause instability. Common works that may trigger landslides on cut or fill slopes are indicated in Table 3.2.

Examples and scales of investigation methods used to assess whether the proposed works could result in landsliding are discussed in Part 4.

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PART 4

LANDSLIDE INVESTIGATION METHODS

Drilling rig at Te Oreore Landslide, SH4

PART 4 – LANDSLIDE INVESTIGATION METHODS

1 INTRODUCTION

Part 3 of Unit 1 covered fundamental surface features that are typically associated with landslide movements. Surface observations and geological mapping provide a means to establish a preliminary view on the landslide mechanism, which forms the basis for scoping of subsurface investigations and the engineering analysis that follows.

Part 4 discusses subsurface investigations and landslide monitoring techniques that supplement the surface observation and mapping techniques discussed in Part 3.

Appropriate scale and type of investigations depend upon landslide size, complexity, and the risks the landslide poses. The aim of investigations is to obtain material parameters, pore pressures or groundwater levels, understand internal variations, depth and nature of the slide surface/s and vertical and lateral limits of the landslide as part of development of the Engineering Geological Model, discussed in Part 5.

2 BACKGROUND

Part 3 includes a series of investigation techniques that can be applied in any landslide assessment, regardless of size. These are the 'must do`s' that form **Stage 1** of any landslide investigation, which help to develop the Conceptual Engineering Geological Model, discussed in Part 5. Geological mapping provides the basis for planning subsurface investigations and locating appropriate instrumentation.

Part 4 of the guidance covers those investigations and common monitoring techniques that enable the geoprofessional to obtain a sufficient understanding of the landslide to reduce remaining uncertainties. These investigations and monitoring techniques can also be used to define the landslide mechanism such that consequences, likelihood and risks can be adequately assessed along with appropriate mitigation options. This is **Stage 2** of the investigations and forms part of the development of the Observational Engineering Geological Model.

Ideally the geoprofessional should have an understanding of the landslide mechanisms typical in the project area before undertaking subsurface investigations. This is where local knowledge is important: having experience of landslide types through knowledge of local geology. Also, it is very important for the geoprofessional to recognise landslide types from the geomorphology they have observed during their desktop assessment, site walkover and geological mapping.

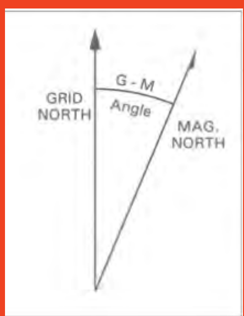
Following desktop assessment, site walkover and geological mapping in Stage 1, Stage 2 techniques should be considered depending upon the size, complexity and (un)certainty of the landslide. These are described in the following sections.

3 SURFACE INVESTIGATIONS

Landslide topography can provide indications of potential, and / or existing instability. Accurately evaluating the surface characteristics of a landslide helps to a large degree to understand the likely subsurface characteristics. This is typically done by a combination of Aerial Photographic Interpretation (API) following by ground survey, as outlined in the following sections.

MAGNETIC DECLINATION IN NEW ZEALAND

In NZ magnetic declination changes from around 19° east of grid, or true north, in Northland to 26° in eastern Southland. This needs to be accounted for on a regional basis when assessing structural data.



Magnetic declination can be determined for every town in NZ at the website: Magnetic Declination (magnetic-declination.com)

3.1 GEOMORPHOLOGICAL MAPPING

Surface features should be recorded on a geological and geomorphological map or an annotated aerial image to create a geomorphological model. Morphological, geological and landslide features will create a record of surface features that can be interpreted by the geoprofessional to indicate the presence and nature of existing landslides and associated (e.g. seepage or sinkholes) or controlling (e.g. structural defects) features. These maps can then be used to help design the subsurface investigations and to generate geological cross sections once subsurface investigations have been completed, as part of development of the Engineering Geological Model (discussed in Part 5). The level of detail to be included on a map should be considered depending upon its purpose but should include surface features such as changes in slope, scarps, cracks, outcrop areas, areas of seepage, drainage lines, local depressions, vegetated areas and displaced or rotated vegetation. While the primary objective is to delineate areas judged to be disturbed by slope movements, it is equally important to record ridge crests, roads, tracks, fences, powerlines and the like that lie within or cross the study area as these may also provide evidence of slope movement.

Although all pertinent data should be collected in the field and recorded in field mapping, sometimes the end product map to be included in the final report may be better simplified with only pertinent features shown (landslide extent, main cracks or springs, water seeps etc.).

3.2 GROUND SURVEY

THE IMPORTANCE OF MAPPING

Engineering geological maps and sections are a fundamental part of the EGM knowledge framework. From IAEG Commission 25:

"Engineering geological mapping – the preparation of a map depicting the distribution and surface boundaries of engineering geological units, geological structures, geomorphology and hydrogeological conditions that are of significance to the project using appropriate symbology."

Ground surveys serve two purposes:

1. to determine the areal extent of landslide activity and to define surface features such as cracks and bulges, springs and seeps, and
2. to establish the rate and pattern(s) of movement. This will require the establishment of an accurate 3D referencing system to establish a ground control, and a baseline survey to compare against future movements.

There are a variety of surveying methods available ranging from the basic methods of low cost, low accuracy and with variable reliability through to more detailed methods of higher cost, high accuracy and excellent reliability. Examples of commonly used methods are summarised in Table 4.1. Figures 4.2 and 4.3 show examples of geomorphological maps combining mapping and location survey of key features.

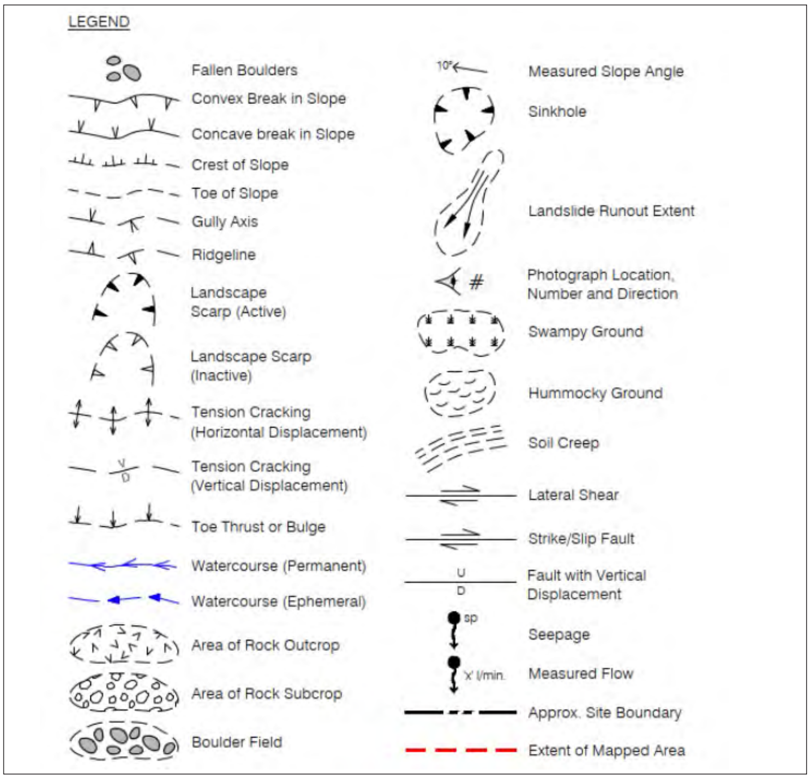


FIGURE 4.1. Example geological and geomorphological mapping symbols. Note that these are examples and geoprofessionals will likely have their own symbol variations to record the key infomation. A good example of mapping symbols is provided in Figure 2.1 and 2.2 of "Geomorphological Field Manual" by Dackombe and Gardiner (1983).

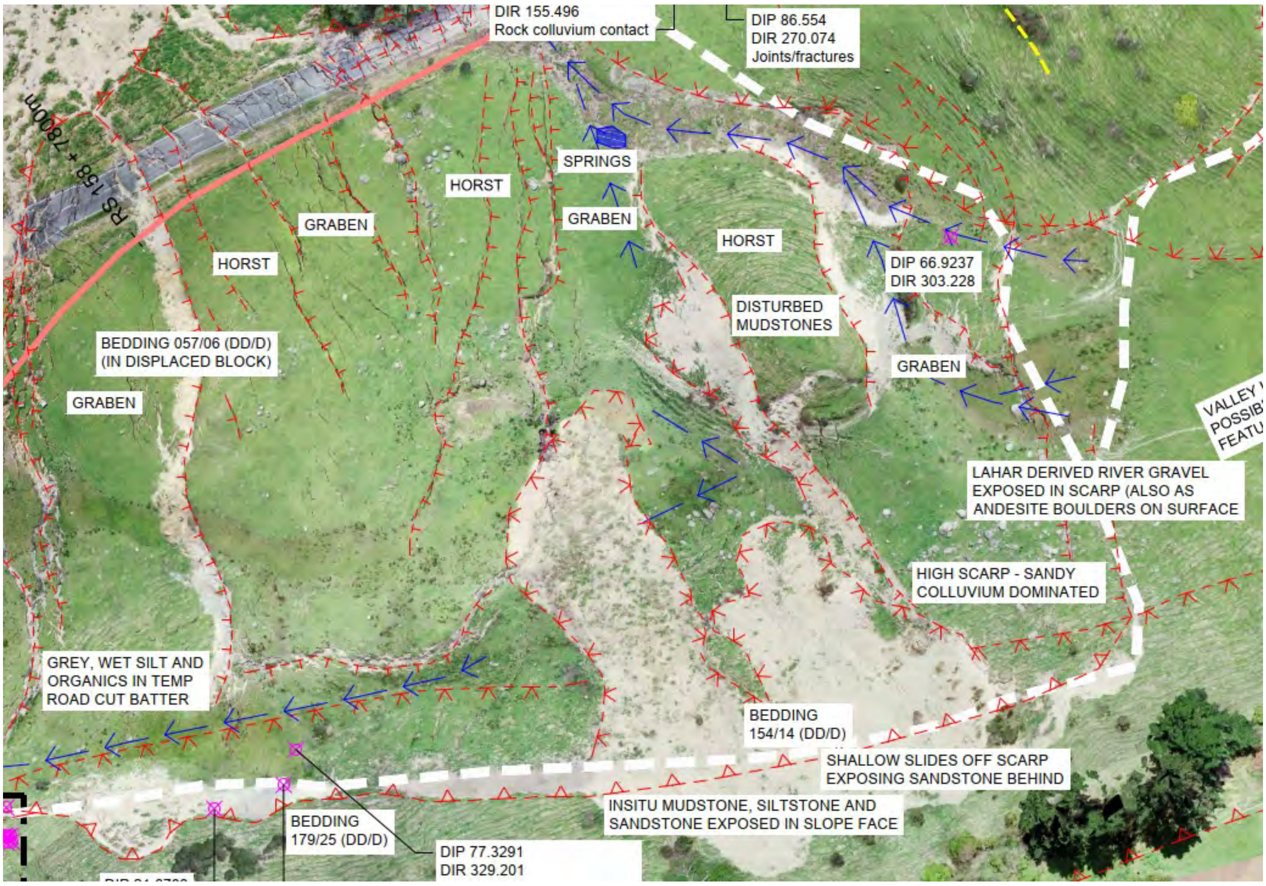


FIGURE 4.2. Part of an example preliminary geomorphological map of Te Oreore landslide, SH4 (reproduced with permission of Waka Kotahi)

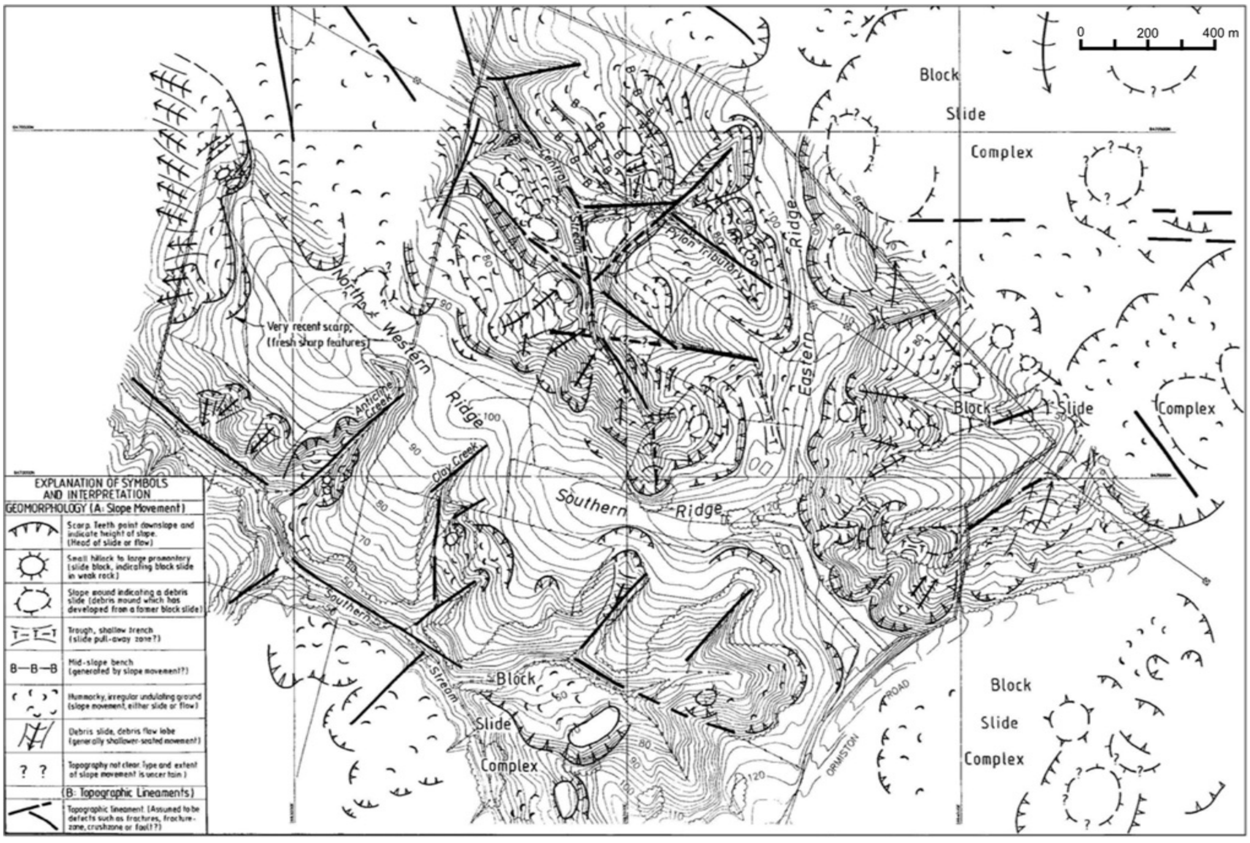


FIGURE 4.3. Example Geomorphological map for a block slide complex (Prebble & Williams, 2016)

TABLE 4.1 Comparison of common ground survey methods

METHOD	RELATIVE ACCURACY	ADVANTAGES	LIMITATIONS
Compass and tape	Low to Moderate. Provides approximate values.	Rapid and allows the production of a map or section.	Reasonable estimates on rough terrain. Reliability can vary depending upon experience and roughness of terrain
Electronic Distance Measurement (EDM)	Moderate	Precise, long range and rapid. Good reliability.	Accuracy is influenced by atmospheric conditions. Limited accuracy over short distances.
Total Station	Moderate to high	Precise and rapid and provides digital data. Good to excellent reliability.	Accuracy is influenced by atmospheric conditions.
GPS	High	Precise horizontal position and possible precise vertical position.	Possible interference
Drone	Very high	Precise. Rapid.	Rain and high wind prevent use

4 SUBSURFACE INVESTIGATIONS

WHY DO INVESTIGATIONS?

Investigations are undertaken because:

1. **The geoprofessional has a really good idea of what's going on and wants to prove their ground model and to obtain specific information for analysis purposes. This is the preferable scenario.**
2. **The geoprofessional thinks they know what's going on and wants to test their theory. This is an acceptable scenario but needs careful thought before proceeding – there may be other theories.**
3. **The geoprofessional has no idea what is going on and is trying to develop a ground model. This is not an advisable scenario. Before proceeding, re-review collated information, develop ground model and test with ground truthing observations (Refer Part 5 of the guidance).**

4.1 INTRODUCTION

Although geological and geotechnical information can be obtained from surface investigations, subsurface investigations are required to obtain more definitive information. Subsurface investigations should follow an iterative process that is constantly adjusted as new information is obtained and hypotheses and mitigation strategies tested.

Part 3 of Module 2 of the Earthquake Engineering Series contains significant information on a range of commonly used subsurface investigation techniques and should be referred to for further detail. A good example of the project activities associated with geotechnical investigation (and design) is given in Figure 1 of Auckland Council`s Code of Practice for Land Development and Subdivision", Chapter 2: Earthworks and Geotechnical.

Surface observations, engineering geological mapping, walkovers and site surveying should provide detailed information that can then be used to scope appropriate subsurface investigation methods.

The subsurface methods (along with monitoring data obtained from field instrumentation discussed in Section 5 below) should therefore be aimed at correlating with the surface mapping observations to improve understanding and knowledge of:

- distribution, modes and rates of slope movement
- position and geometry of slip surfaces
- position of groundwater profile(s) and piezometric surfaces
- relationship of rainfall to groundwater profile and movement
- possible locations for stabilising or mitigation structures

Subsurface investigations and monitoring results should be used to revise and update the surface mapping and ground model. This re-evaluation may identify a need for additional subsurface investigations.

Surface and subsurface investigations of landslides should always remain dynamic. All parties (the geoprofessional, client, and contractors) should be ready to accommodate changes to the investigation scope based on the findings of the investigations as they proceed. The need for additional information should be allowed for with a contingency for additional costs.

4.2 DEFINING APPROPRIATE INVESTIGATION METHODS

Selection of the appropriate investigation method is based on considerations such as:

- study objectives
- size of the landslide area

- geological conditions / complexity
- access, land ownership
- budget and time constraints (staging of investigations).

Information from the Stage 1 investigations should be used to support the selection of the investigation methods and the location(s) of the investigations. Decisions regarding the type and location of subsurface investigations are dependent upon the information needed to confirm the hypotheses on the landslide, but some general guidance for surface and subsurface investigation methods (excluding desktop / aerial imagery interpretation and mapping techniques discussed in Part 3) are described in the sections below.

4.3 PLANNING SUBSURFACE INVESTIGATIONS

A landslide investigation will need to consider information on site accessibility, the anticipated geological conditions, investigation locations, spacing, depths, samples required and sampling frequency. The investigation programme should be coordinated with laboratory testing and / or instrumentation to obtain the necessary parameters for analysis.
When planning a landslide investigation there are two primary considerations:

1. Safety of personnel – because landslides may be active, there is risk to workers and equipment from ongoing movement and so precautions, protection or movement warning devices may be required.
2. Oversight and guidance on appropriate landslide investigations should be undertaken by a suitably qualified geoprofessional (an engineering geologist or geotechnical engineer with experience of landslides and the geology in which they have occurred), or an engineering geologist or geotechnical engineer working alongside or under the guidance of a geoprofessional with experience, and who has been involved in developing the investigation programme.

4.3.1 Area of investigations

For well-defined landslides, the area of investigations may be confined to the landslide area and the area immediately surrounding the landslide if time and budget permits and the area is accessible. It may not be practical or feasible to do this in residential areas where existing development may surround the area under investigation or adjacent landowners do not permit investigations within their landholding. However, the risk of confined investigations increases the risk of not identifying a larger landslide hazard that might not be immediately obvious. Where possible the investigations should be considered outside the obvious landslide footprint to ensure a bigger landslide hazard is not present. If the geoprofessional suspects the instability extends beyond property boundaries, then they should consider communicating (& documenting) the risks with other stakeholders (e.g. Council) and landowners and provide recommendations for further investigations. Where the landslide is poorly defined or the study is of an area of potential instability, the area of investigations may initially need to be much larger. As a general guide, the area to be studied should be two to three times wider and longer than the area suspected. This greater extent should be initially assessed by mapping (including drone survey and DTM) to better define the actual landslide area and then determine whether subsurface investigations are justified and how they might be optimised. Again, access and land ownership constraints may exist to prevent a sufficiently wide investigation area and if so these constraints and the associated risks should be communicated and documented with the client, stakeholders and landowners.

4.3.2 Investigation locations

The layout, spacing and depth of investigations across a landslide will depend upon information obtained in Stage 1. It is recommended that there are investigations along the centre line near the top, middle and bottom of the landslide as a minimum. It adds value to the geoprofessional's understanding of the landslide if some investigations are also undertaken outside the landslide footprint to provide a comparison of conditions on the unstable and stable parts of the slope respectively.

THE NEW ZEALAND GEOTECHNICAL DATABASE

As outlined by Engineering NZ, *"The NZGD was established in 2013 by the Canterbury Earthquake Recovery Authority (CERA) and has a user base of more than 11,000 individuals. It has facilitated the upload of approximately 168,000 geotechnical reports.*

Efficient availability of information through the NZGD offers benefits to geotechnical engineers and their clients, and local communities. By providing data for high-level regional assessments, the platform helps reduce costs by preventing the duplication of sub-surface geotechnical investigations."

We strongly encourage all geoprofessionals to upload subsurface information and testing to the NZGD. It should be considered part of our duties under the concept of kaitiakanga.

4.3.3 Depth of investigation

Determining the depth of investigations in advance can sometimes be difficult unless there are surface features like grabens that help give an indication of depth. Otherwise, the geoprofessional is reliant on their experience of investigating similar landslides in similar geological terrain.

General guides that may be considered include:

1. The depth of movement at the centre of the landslide is rarely greater than the width of the zone of surface movement. Landslide depth can vary. Circular failures are much deeper than translational slides and their depth will be much closer to the width of a landslide than that of a translational slide. Translational slides of colluvium soils over rock can be significantly wider than they are deep.
2. Assuming vector displacement parallel to the failure surface allows an approximate estimation of the failure surface along either translational or circular slides.
3. The maximum depth of the failure surface for a shallow translational slide, rock avalanche or debris flow is often approximately equal to the distance from the break in slope on the original ground surface to the most uphill crack or scarp. See Figure 4.4.
4. For deeper rotational slides, the ratio of the maximum depth perpendicular to the slope/length of the landslide (Dr/Lr) ranges between 0.15 and 0.33 (Skempton & Hutchinson 1969). Smaller values of the Dr/Lr ratio are linked to shallow landslides. An indicative rule of thumb is that the maximum depth of a rotational slide is about 0.3 of its slope length.
5. For very large, deep seated complex landslides it may be that the depth of the basal failure surface can only be determined by drilling. The Clyde landslides around Lake Dunstan were good examples of this; several of them had stepped slide bases that could only be defined once subsurface information was available.

Cross-sections drawn through the landslide will help define investigation depths for different failure mode scenarios. If the subsurface geology is understood sufficiently, the geoprofessional might be able to anticipate if the landslide is more rotational or translational. For circular failures in soils, connecting the possible toe bulges with the uphill scarps will give an approximation of the failure surface depth (Figure 4.5)

It should be noted that landslides may not be purely translational or circular. Circular landslides can have a translational component and vice versa, so these general guides should be considered and applied carefully.

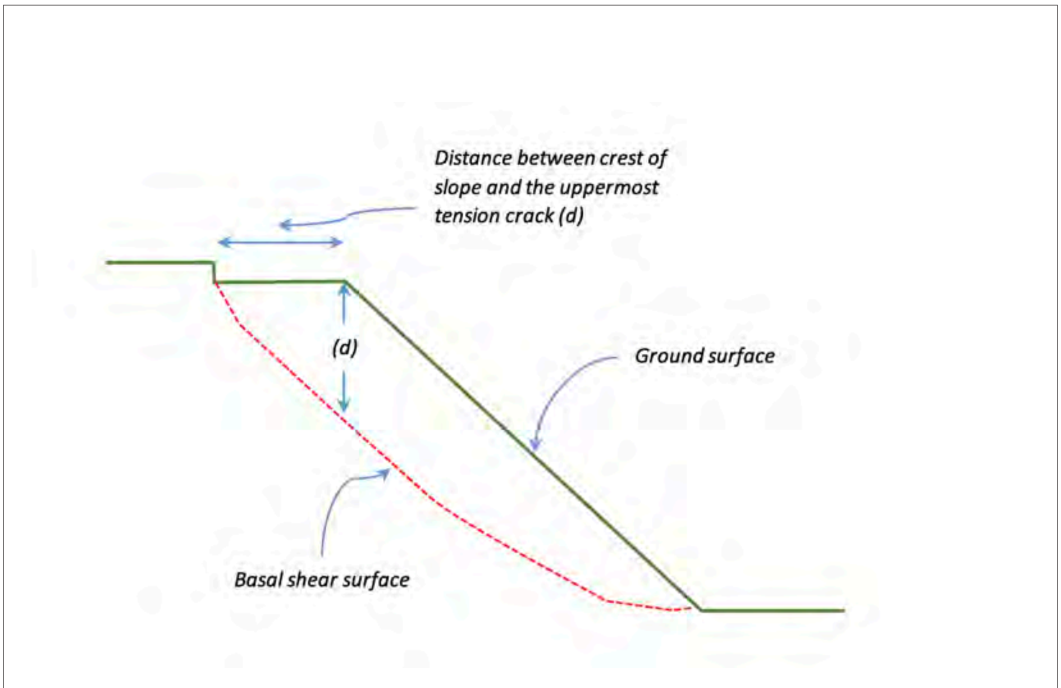


FIGURE 4.4. Inferred landslide depth in shallow translational landslide based on position of uphill cracking to break of slope (schematic diagram).

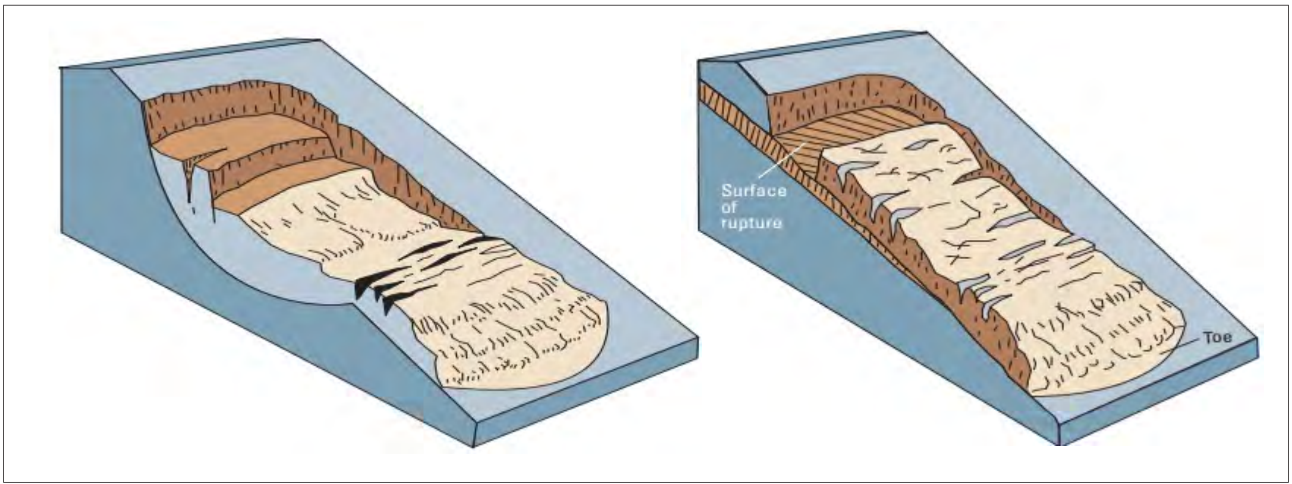


FIGURE 4.5. **Comparative shape of failure surfaces in rotational slides (left) and translational slides can help to assess their likely depth and the best place(s) for investigations.**

Boreholes or other boring methods should extend sufficiently below the inferred failure surface to identify material that has not been displaced. This is where engineering judgement based on a good understanding of the landslide model is key to ensuring investigations are sufficiently deep. The landslide model will indicate the inferred depth of the failure surface. The geoprofessional should then target their investigations to a minimum of say 5 m below the landslide failure surface or to a depth of around 25% of the landslide thickness below the shear surface (which can only be confirmed once the drillhole intercepts the slide surface). What is critical during the investigation is that the geoprofessional monitors the drilling as it proceeds. Use of SPTs within 5 m of the anticipated failure zone should be avoided. It is important to always keep in mind that borehole depths may need to be revised as the investigation programme proceeds. In some cases instrumentation installed may indicate that the landslide is deeper than originally anticipated and require supplementary investigation.

Where landslides heavily disturb the geology then identifying the landslide shear surface would appear to be relatively easier. In Figure 4.6 below for example, the landslide basal shear surface was inferred to be at approximately 26.5 m depth (indicated by the dashed red line).

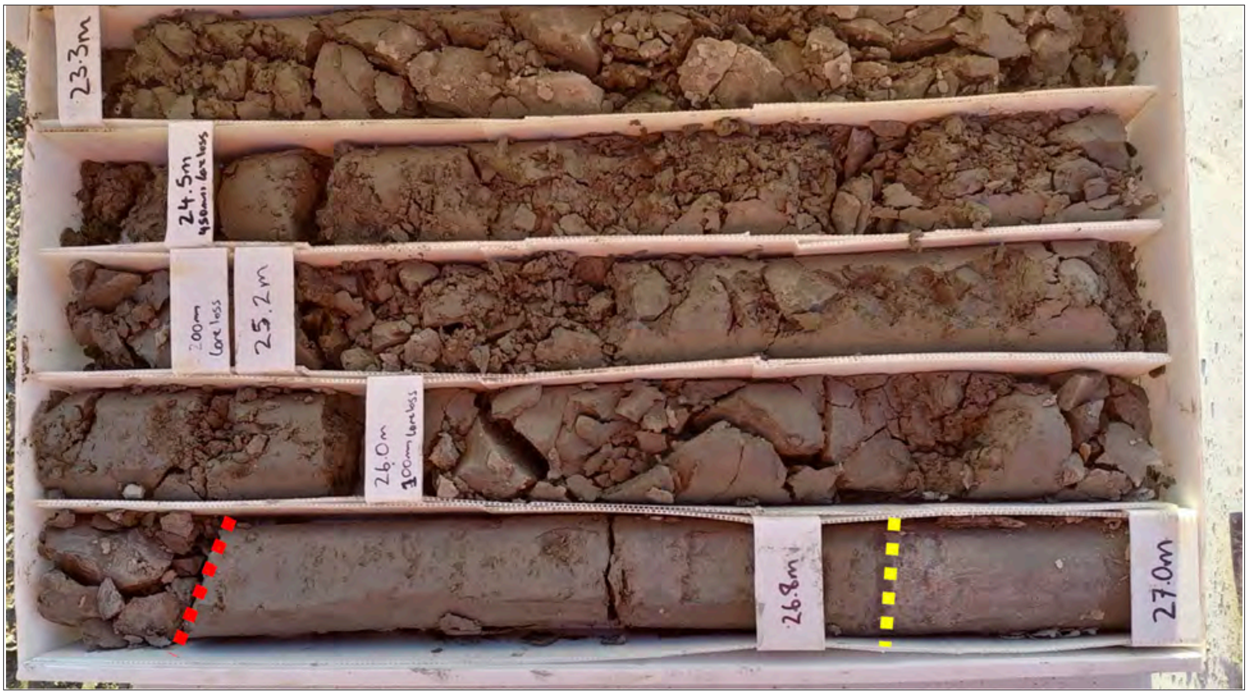


FIGURE 4.6. **Landslide deposits overlying intact Tertiary Mudstone. Red dashed line shows inferred basal shear surface; yellow dashed line indicates sheared surface shown in Figure 4.7.**

FIGURE 4.7. Landslide shear surface identified by polished surface and striations in core.

However, the basal shear surface was actually located at approximately 26.85 m depth (yellow dashed line in Figure 4.6) indicated by a polished and striated surface in the core (Figure 4.7). The geoprofessional should therefore take extreme care with inspecting and handling the core to identify the slide surface before the core becomes too disturbed by transport or cursory inspections.

4.4 ROLES OF THE GEOPROFESSIONAL DURING INVESTIGATIONS

The primary roles of the geoprofessional during a landslide investigation should be to:

a. Ensure the investigations are undertaken to the technical and contract specifications.
b. Maintain communication with the project team on findings.
c. Select and approve changes to the scope as new information is collected or unanticipated conditions are encountered (i.e. allow room in the contract for flexibility in the investigation depths or location etc if initial findings are not as anticipated).
d. Ensure that complete and reliable investigation information is collected.
e. Identify all geological conditions and the landslide model as accurately as possible.

The geoprofessional must remain vigilant to the information being collected during landslide investigations. They should never just "do what they said they were going to do" because that can result in lost or missing key information which could result in uncertainties in the landslide model and mechanism, along with design assumptions which result in greater project risk.

4.5 INVESTIGATION METHODS

Subsurface investigation methods can be classed as either direct methods or indirect methods.

Direct methods that allow direct examination of materials in the landslide include hand augers, window samplers, test pits and boreholes. These methods allow the geoprofessional to physically examine the materials within the landslide.

Indirect methods are those that measure material properties without physically examining the materials. For example by geophysics, dilatometers, Dynamic Cone (Scala) penetrometers or Cone Penetration Testing (CPTs). These methods are then used to estimate the material type.

In New Zealand any subsurface investigations should be undertaken in accordance with the New Zealand Ground Investigation Specification – Volume 1: Master Specification (current version April 2017) (NZGS, 2017).

The following sections discuss different types of investigation method.

4.5.1 Investigation methods

Table 4.2 provides a summary of common investigation techniques used in New Zealand. Other methods are available which the geoprofessional may wish to consider. All methods should be considered in terms of their applicability and limitations.

TABLE 4.2 Application and limitations of common investigation methods

INVESTIGATION	APPLICATION	LIMITATIONS
Dynamic Cone Penetration tests (Scalas)	• Fast and efficient. • Continuous penetration over shallow depth. • Shallow landslides comprising loose to dense soils	• No samples for inspection • Cannot penetrate strong / very dense soils or rock
Cone Penetration Tests (CPTs)	• Fast and efficient. • Continuous penetration.	• No samples for inspection • Cannot penetrate strong / very dense soils or rock. • Access / rough terrain may constrain locations.
Hand augers	• Provide continuous profile in granular soils above the water table and in firm or stronger clayey soils above and below the water table	• Samples are disturbed. • Cannot penetrate below groundwater table in granular soils. • Penetration in strong soils is difficult. • Limited to 4 – 5 m depth below ground surface
Test pits and trenches	• Provides a visual exposure of geology, groundwater, soil / rock interface, discontinuities and failure surface (if within 3 – 4 m of ground surface). • Allows samples to be collected.	• Limited depth with lightweight machinery. • Pit sides can be smeared. • Pit stability health and safety issues • Limited use below water table.
Boreholes	• Provides a continuous profile through the landslide and into underlying intact material • Allows for downhole testing and sampling to be undertaken at desired depths • Allows for installation of Optical or Acoustic Televiewers (OTV/ATV) for observing borehole sidewalls • Boreholes can be drilled to desired depth and are typically not limited	• Access to set up rig can be difficult meaning investigation locations may be limited. • More expensive than other methods

TABLE 4.3 Application and limitations of common geophysics investigation methods

GEOPHYSICAL METHODS:		
Seismic refraction profiling	• Used to determine characteristic seismic velocities.	• May be difficult to interpret
Direct seismic (downhole and cross hole) surveys	• Used to determine seismic velocities and rock-mass quality	• Data represent averages and may be affected by mass characteristics • May be incorrectly interpreted
Ground penetrating radar	• Provides a subsurface profile, located buried objects (i.e. services), boulders and soil-bedrock interface	• Limited depth and penetration in clay materials. • May be incorrectly interpreted
Electric and electromagnetic resistivity surveys	• Locates boundaries between clean granular and clay strata, groundwater table and soil-bedrock interface	• Can be difficult to interpret. • Does not provide engineering strength properties.

4.5.2 Geophysics methods

Table 4.3 provides a summary of common geophysics investigation techniques used in New Zealand. Again, other methods are available which the geoprofessional may wish to consider. All methods should be considered in terms of their applicability and limitations.

4.5.3 Boreholes

Boreholes form a critical component of landslide subsurface investigations. Their primary purpose is to:

1. Identify the subsurface distribution of materials within the landslide and the landslide depth
2. Allow for sampling or in situ testing to obtain information on landslide material parameters

3. Allow the acquisition of groundwater information through observations and monitoring.

The method of drilling is important to ensure that as much information as possible is obtained from the investigation and from the soil or rock below the landslide. Typically, rotary coring using PQ or HQ method is used to obtain information on the landslide materials and to minimise any changes in the nature of the strata being sampled and tested, such as can be caused by sonic drilling. Sonic drilling may be suitable in the main body of the landslide to allow recovery of a continuous sample, but should be switched to a cored drilling method at least 5 m before the failure surface is anticipated. Sonic drilling may be suitable for deep landslides (i.e. where the failure surface is greater than 10m deep, but should be avoided if the landslide is expected to be relatively shallow depth (i.e. less than 5-10 m).

Other drilling methods like percussion drilling, wash drilling or augering do not allow for the ability to obtain continuous material retrieval and therefore key information like identifying the landslide shear surface will be missed.

4.5.3.1 Field Testing

Field testing in the boreholes allows the geoprofessional to determine the physical strength properties of the materials that make up the slope. Field testing in the boreholes typically comprises penetration tests that are undertaken as the borehole is progressed. The most common borehole field test is the Standard Penetration Test (SPT). The SPT can be conducted at any desired depth within an advancing borehole although common spacings used in the industry are continuously or 1.5 m centres to suit the drill rod lengths. The values obtained from the SPT are empirically correlated with soil properties obtained from laboratory or field tests to make these a low cost but extremely valuable test to supplement other sampling or testing procedures.

Boreholes also allow samples to be taken for laboratory testing. Sampling may be directly from extracted cores or by push tubes or even SPT samples. In most cases a split-spoon can be used to allow recovery of sample while obtaining strength test data.

Care should be taken when undertaking SPTs in landslides. The use of SPTs within a few metres of the expected depth of the landslide failure surface should be avoided, as critical features are likely to be missed.

The cone penetration test (CPT), which has become internationally one of the most widely used and accepted test methods for determining geotechnical soil properties, gives a continuous profile of the subsoil layers, often called a 'trace'. CPT results are used to understand the soil properties and how the ground is likely to behave under different levels of earthquake shaking. CPTs were used prolifically in Christchurch following the 2010-2011 earthquakes and are used across New Zealand to assess hazards on liquefaction prone land. CPTs are less widely used in landslides, but if the geology is favourable (i.e. the landslide comprises loose to dense soils with little rock or strong boulder obstructions) then they can be used to obtain a useful soil profile and indication of groundwater level.

Other borehole field testing methods may include vane shear tests, pressuremeter or dilatometer tests but these are lesser used in New Zealand and have more limitations. A good discussion on the relative merits of the Dilatometer Testing (DMT) methods is given by Failmezger (2021), who indicates that DMT provides more accurate measurements of soil stiffness, compared to SPT and CPT testing.

4.5.4 Investigating groundwater conditions

Groundwater level / pressure is a contributor or a major trigger of many landslides so identifying groundwater conditions and understanding how they vary is a critical aspect of any subsurface investigation.

Because of the disturbance of soil and rock, groundwater conditions within a landslide can be complex. Landslide movement can affect and change groundwater level, and groundwater flow directions and volumes. Observations and geological mapping should identify surface expressions of groundwater such as springs, seeps, natural ponds and marshy ground as well as the relative levels of nearby streams, rivers, lakes or coastlines. These can be used to help determine the position and complexity of the water table(s) and the saturated zone(s) of the landslide.

Observations of seeps at the toe of the landslide may indicate that the landslide is draining but mostly should be interpreted to mean that there is potential for groundwater to recharge during rainfall events. A reduction or cessation of earlier flows at the toe may be another sign that there is a potential for further instability.

Specific issues to consider when designing investigations targeting groundwater conditions include:

- Groundwater conditions in small landslides might be straight-forward but groundwater conditions should always be considered as complex for larger landslides.
- Groundwater conditions within a landslide may change over time due to ongoing landslide movements, rainfall or seasonal changes.

- Disturbance of materials within the landslide mass (sheared soils and rocks, loosened debris, entrained plant material etc.) can lead to juxtaposed materials with quite different permeabilities and the development of perched groundwater levels and isolated zones of saturated permeable material containing groundwater under elevated pressures.
- Groundwater pressures may differ at different depths at the same location (see Figure 4.8) due to the presence of perched and / or confined groundwater
- Groundwater in a landslide may be directly or indirectly affected by rainfall (see Figure 4.8)
- Groundwater in a landslide may be fed by surface water discharge, typically poorly controlled surface water runoff – check whether this is a contributor
- Generally, increases in groundwater pressure with depth indicate an upward flow of groundwater whereas a reducing pressure with depth indicates a downward flow. However, these changes may also be indicators of multiple aquifers within the slope that are not interconnected.

Drilling in large, complex landslides may encounter a wide range of groundwater conditions due to the disturbance of the materials and the resulting presence of impersistent perched or confined aquifers within the landslide that are separate from the sub-basal water table but can influence the behaviour of the slide mass. Investigations in the very complex schist landslides in the Cromwell Gorge for the Clyde Power Project made use of the DVD (depth v depth) plot described in Dick (1976) which plots the depth of the drillhole against the recorded water levels at the beginning and end of each day (see Figure 4.9). The method was suitable in those slopes because the materials were of low to medium permeability, the holes were commonly deep (> 100 m) and drilling was slow, which allowed more data points to be obtained. On that project the DVD plot allowed the identification of possible internal aquifers and their depth range, determination of their nature (confined or not) and assisted with optimally locating piezometers within and beneath the landslides (Macfarlane, Pattle & Salt, 1992).

Although landslides are complex and generalisations should be considered carefully, there are a number of assumptions that can be used as a guide when undertaking investigations:

- Surface observations are essential.
- Seasonal changes in groundwater will not be detected unless monitoring is undertaken over a period of at least one year and preferably many years.
- Consideration should be given to measurement of temporary or transient groundwater pressures resulting from intense rainfall, e.g. percolating groundwater fronts, which may not connect with underlying long term groundwater tables.
- Landslide movement can result in the formation of open cracks in the ground that can facilitate access of water into the landslide mass.
- Ponding of water or the disruption of surface water channels anywhere on or adjacent to the landslide can increase surface water infiltration into the landslide.
- Landslide movements may block groundwater flow paths through more permeable areas of the landslide, resulting in increased water pressures and further instability.
- Low permeability soils in landslides respond more slowly to changing groundwater conditions that can only be observed and understood through long term monitoring.

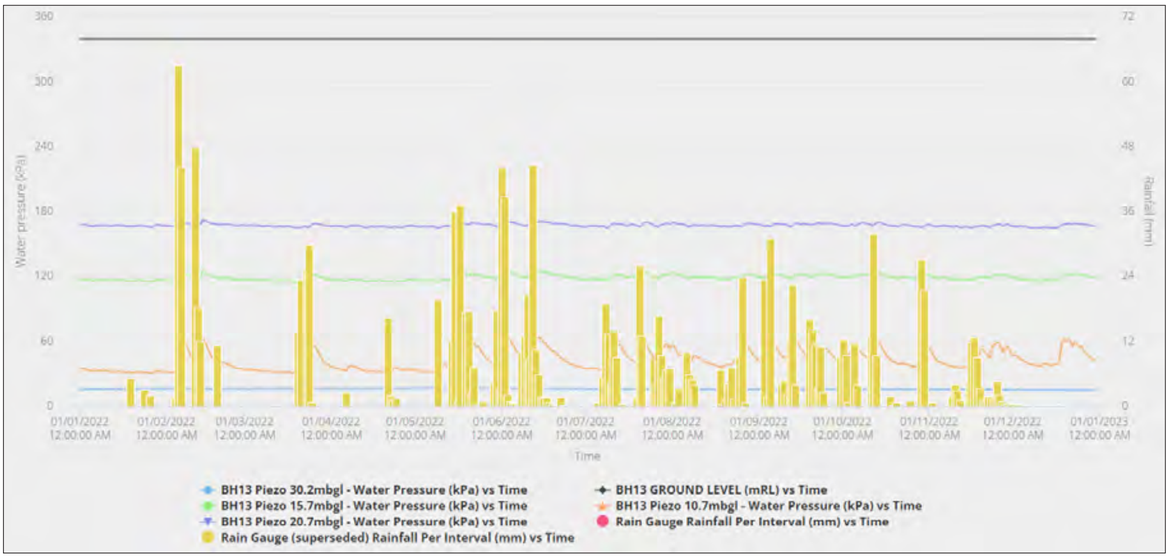


FIGURE 4.8. Different water pressures recorded by vibrating wire piezometers at different depths in a borehole at Te Oreore landslide versus rainfall. The piezometers in the landslide (10.7 m bgl, 15.7 m bgl, 20.7 m bgl) show increasing water pressure with depth within the landslide mass. The piezometer at 30.2 m bgl is below the landslide in an undisturbed sandstone bed indicating the sandstone is relatively free draining as the water pressures are low.

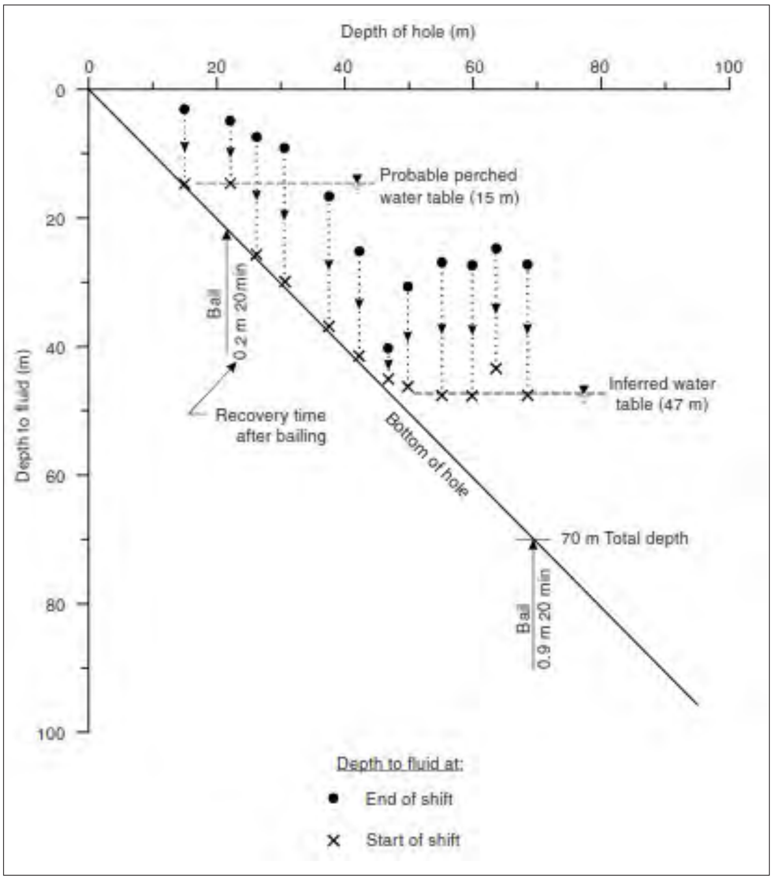


FIGURE 4.9. Example of a depth v depth plot identifying multiple aquifers within the drillhole. From Fell et al (2015, Figure 5.33)

Direct measurements of groundwater level are typically made by monitoring the groundwater level in a well or standpipe piezometer installed in a borehole. Groundwater pressures can be directly recorded using a vibrating wire piezometer.

The main types of piezometers used in New Zealand are standpipe piezometers and vibrating wire piezometers. Historically standpipe piezometers were read manually by dipping and readings were slow, infrequent, costly and not as accurate. In the present time, level loggers can be placed down the borehole for more accurate readings and loggers can be left for long periods with long term data sets being collected. In both standpipe piezometers and vibrating wire piezometers, they can also be monitored by way of telemetry. This allows real time results to be relayed to the geoprofessional for review by way of a portal (Figure 4.8). Telemetry has been used effectively on both the Te Oreore and Utiku landslides to monitor groundwater levels following rainfall, and in the case of Te Oreore, the effects on groundwater from ongoing groundwater pumping.

Very detailed information can be obtained using multi-port piezometers, such as the Westbay instruments installed in some of the Lake Dunstan (Cromwell Gorge) landslides, but these are very expensive and usually difficult to justify.

The installation of standpipe piezometers should be undertaken in accordance with Section 13 of the New Zealand Ground Investigation Specification (April 2017). Vibrating wire piezometer installation should be undertaken by specialist instrumentation companies.

5 FIELD INSTRUMENTATION FOR LANDSLIDE MONITORING

Field instrumentation can provide valuable information on movement of incipient and fully developed landslides. It is commonly installed either on the surface or in boreholes and it provides supplementary information to that collected from surface observations and other subsurface investigation techniques.

Instrumentation can be used in the following situations to obtain specific information including:

- Locating active failure surfaces within the landslide
- Determining the rate of landsliding
- Determining the depth and shape of the landslide to support the calculation of strength parameters at failure and the design of remedial measures

- Monitoring marginally stable natural or cut slopes and the effects of construction work or rainfall on the slopes
- Monitoring groundwater levels or porewater pressures so that effective stress analysis can be undertaken
- Allowing the installation of remote monitoring and alarm systems that can send alerts and alarms indicating potential or imminent movements
- Allowing the monitoring and evaluation of the performance and effectiveness of landslide control measures.

5.1 INSTRUMENT SELECTION

Instrumentation typically aims to measure crack opening, landslide movements and groundwater levels or porewater pressures. The types of instrumentation, instrument layout and monitoring requirements will be determined by the needs and stage of the project or structures / infrastructure impacted (or potentially impacted) and the available budget; instrumentation varies greatly in cost and degree of sophistication.

In the early stages of a landslide investigation, simple surface located survey marks and crack or tilt meters and a rain gauge might be installed to help understand the way the slope is behaving and its sensitivity to rainfall. This may help the development of the ground model. However, the location of instrumentation for long term monitoring needs to be based on a good understanding of the geological model and groundwater conditions so that meaningful and useful information can be obtained. For example, if movements are large or the shear surface is well defined then crude and simple movement instrumentation could be used, whereas if movements are small and the depth of movement is uncertain, then more precise instrumentation should be considered. Depending on the project and need for precise instrumentation, a well-defined landslide with ongoing movement could also warrant the need for precise telemetered instrumentation to be installed.

When selecting instrumentation for a landslide assessment that necessitates longer term monitoring, there are some basic requirements that need to be considered. In particular, instruments should be:

- reliable, rugged and capable of functioning for long periods without repair or replacement and,
- able to record changes in conditions accurately and rapidly.

5.2 SURFACE MEASUREMENT INSTRUMENTATION

Surface instrumentation ranges in cost and precision and common methods are summarised below. Other methods are available and may be considered once researched and the benefits and limitations understood for the landslide being assessed.

- **Surveying** – Refer Section 3.2 and Table 4.1. Surveys are used to determine areas and rates of surface movement. These methods can be crude and simple, such as installing movement marker pegs across cracks, or more precise and costly such as installing prisms as part of a total station survey network.
- **Surface extensometers and tiltmeters or laser tilt nodes.** Used to accurately measure movements and tilt on a landslide, these are best used across scarps and large tension cracks. They are lightweight, easy to interpret and of relatively low cost. Both extensometers and tiltmeters can be connected to data collection platforms to record and relay information in real time and provide alerts to phones.
- **GPS** uses a base station at a fixed location to measure changes between stations on the landslide surface. Accuracy will be affected in areas of thick vegetation or where weather is poor, so reliability may be questionable.

5.3 GROUND DISPLACEMENT MEASUREMENT INSTRUMENTATION

Ground displacement instrumentation is usually installed in boreholes and can range in precision and cost. The most simplistic way of measuring displacements in a borehole is to lower a steel rod down the borehole standpipe until it cannot be lowered any further. If it can no longer reach the base of the completed **borehole standpipe**, this indicates the point where the standpipe is deforming and the approximate position of the shear surface. Rods of varying lengths can be used (the smaller the rods, the more likely they will be able to descend past the deformation in the borehole) to get better accuracy of the shear surface position. If the landslide is still moving and the borehole is continuing to deform, then the rods that can pass the deformation in the pipe will get smaller and smaller until no rods can pass the deformation in the pipe.

Shape arrays and Inclinometers are used for determining the location of the failure surface(s) and to monitor the magnitude, rate, and direction of landslide movement. Inclinometers essentially comprise a machined casing that must be installed in vertical boreholes. Special devices are lowered down the borehole to measure horizontal displacements at nominated depth intervals and this data is used to provide a profile of horizontal deformation with depth. These are the most widely used instrument for measurement of small amounts of ground creep or shear zone movement.

In-place inclinometers that remain in the borehole (other than to allow for maintenance or adjustments) provide more accurate measurements and continuous readings which can be relayed remotely. Shape arrays are essentially a chain of rigid segments connected by flexible joints that are designed to resist twisting but allow the segments to tilt in any direction. They are typically inserted into a small diameter plastic pipe before being installed down the borehole. The borehole is then backfilled with grout. However, they are more expensive and complex than probe inclinometers and if the landslide is still moving there is a higher risk of borehole deformation and loss of the instruments. Boreholes drilled with the intent of installing inclinometers should be extended several metres below the landslide and into undisturbed ground so that the basal shear surface can be identified and the rate of movement along it measured, while attaining base fixity for the inclinometer (i.e. a stable zone against which deformations can be calibrated). The base fixity zone should be several meters long.

Other movement measurement methods that can be installed in boreholes could include extensometers or strain meters and these may be considered by the geoprofessional. However, they are less used in New Zealand than inclinometers or shape arrays.

5.4 GROUNDWATER MONITORING

Using methods described in Section 4.5.4, groundwater levels and pressures can be recorded over long periods to determine changes in groundwater levels in response to rainfall and in turn they can be used to observe at what critical level or pressure the landslide reactivates. Groundwater monitoring at the Utiku and Te Oreore landslides in the central North Island has been effectively used to help determine trigger levels for landslide movement and the effect of rainfall events and antecedent rainfall on landside stability.

5.5 DURATION OF MONITORING

Past the completion of the physical works, it is usually not practical to do anything other than repeat surveys or monitoring of downhole inclinometers and/or groundwater.

The duration of the monitoring period for the types of instruments described in Section 5 is usually dependent upon project size and complexity. Small landslides affecting residential development are less likely to be monitored over long time periods but large landslides affecting infrastructure or transport corridors may be monitored over a much longer period of time, or in some cases, indefinitely (the Lake Dunstan landslides for example). Because of the relatively low cost to install loggers into boreholes it is becoming more common that they can be left to collect groundwater readings over a longer period. Likewise, prisms can be installed on the ground surface to allow total station or remote monitoring to be undertaken at relatively low cost.

6 REMOTE SENSING

Internationally, remote sensing has been widely used for landslide detection and monitoring since the launch of the first Landsat satellite in 1972 and involves detecting and monitoring the physical features of an area by measuring reflected and emitted radiation from a distance (usually from a satellite or aircraft).

Remote sensing imaging instruments can be categorised as having either active or passive sensors, which can be optical, radar, LiDAR, or multi-spectral / hyperspectral. They can also be classified according to the platforms in which the sensors are installed: satellite-based, aircraft-based, unmanned aerial vehicle (UAV)-based, and ground-based (Auflic et al 2023).

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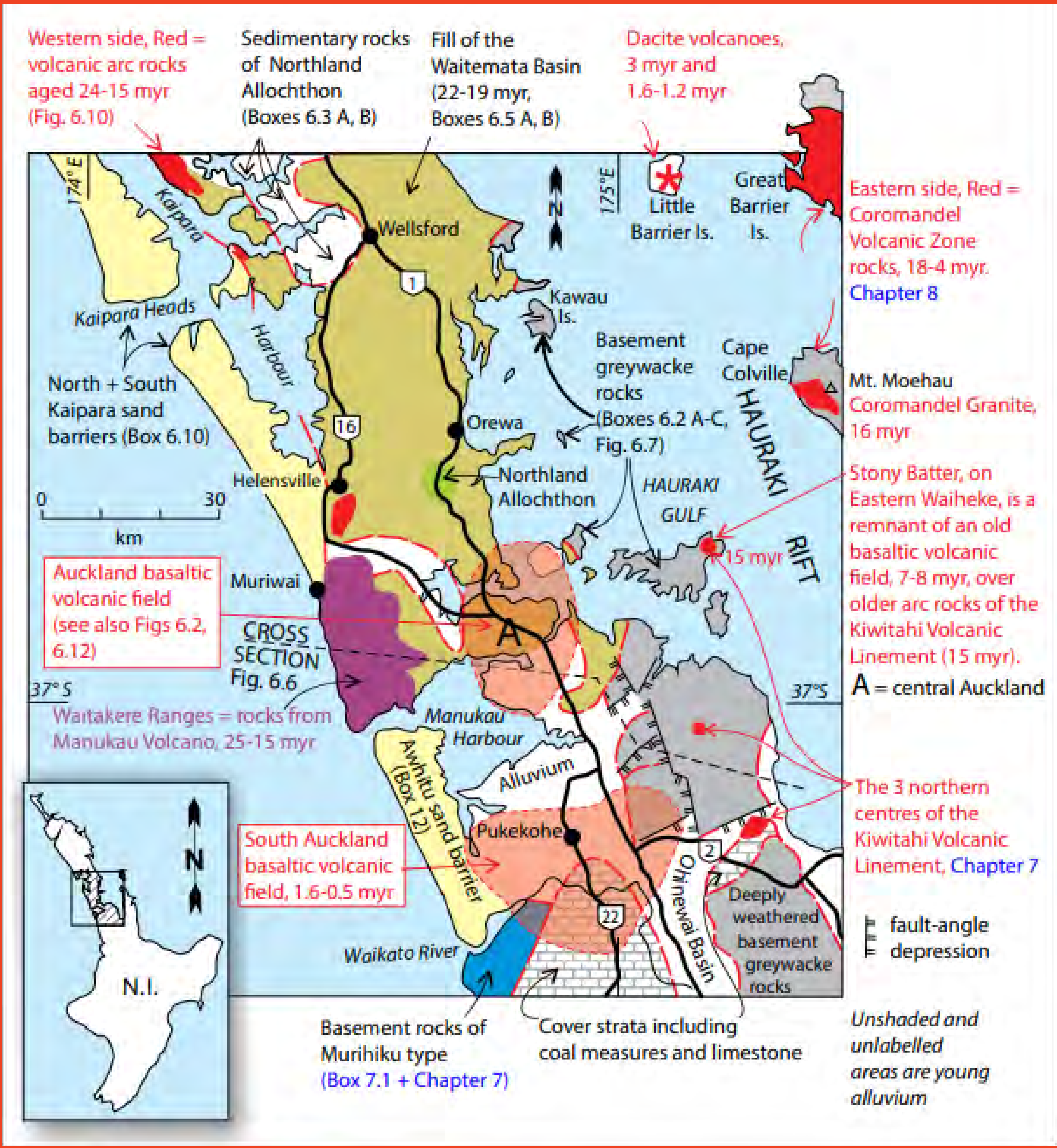
PART 5

ENGINEERING GEOLOGICAL MODELS



Simplified Geological Map of Auckland (Ballance, 2009). Understanding the geological setting is a key component of developing an Engineering Geological Model

PART 5 – ENGINEERING GEOLOGICAL MODELS

1 INTRODUCTION

For any geotechnical engineering project (including landslide assessments), the Engineering Geological Model (EGM) represents the best interpretation of the surface and subsurface conditions and forms the basis for hazard assessment and analytics (which would typically include slope stability modelling, for example, limit equilibrium or finite element analysis).

AN EGM SHOULD BE:

- **based on all available and relevant knowledge,**
- **logically constructed following established principles**
- **focused on the relevant geological conditions and engineering characteristics of significance to the project, and**
- **be clearly communicated.**

AN EGM IS UNLIKELY TO BE:

- **A table of interpreted subsurface conditions (these are only appropriate in very simple situations, which landslides typically are not)**

AN EGM IS NOT:

- **An output from some form of slope stability analysis**

IAEG Commission 25 defines an EGM as

A comprehensive knowledge framework that allows for the logical evaluation and interpretation of the geological, geomorphological and hydrogeological conditions that could impact a project and their engineering characteristics. The EGM comprises both conceptual and observational components and may consist of interrelated models and approaches. The Geological Model, the Geotechnical Model and a Geohazard Assessment are outputs from the EGM knowledge framework.

For landslides, depending on complexity (which may be driven by the geological conditions, or the engineering requirements as discussed in Section 4 following), the EGM maybe as simple as an interpreted cross section. It may be a series of cross sections in combination with a geomorphic map, or it may be perhaps a 3-D block diagram. The key requirement is that it conveys the detail necessary for understanding and illustrating the problem at hand.

Regardless of what the final product is, the model evolves over the length of time of the project – it starts as a hypothesis (the conceptual model) that is tested by some form of site investigation (refer Part 4 of the guidance). It is refined during the investigation based on site observations, and might be refined further during the construction phase, if there is one. *There are no recipes, no standards and no set procedures to follow – engineering geological model development is an art* (Mr David Bell, University of Canterbury).

The bulk of this section of the guidance is derived from IAEG Commission 25, which should be referred to for further detail.

2 DEVELOPMENT PROCESS

An engineering geological model captures all the knowledge of the (in this case) landslide in one place and presents this information in an understandable form. The development of the model starts long before the report (or whatever the deliverables are) is drafted.

As illustrated in Figure 5.1, the initial desktop review helps to build the initial framework of the **conceptual engineering geological model**. As the project progresses, this is developed by including progressively more detailed information, such as aerial photograph interpretation, field mapping, including geological and geomorphological observations, and information from the subsurface investigations, to form the **observational engineering geological model**. The observational model should continue to be developed (and challenged as necessary) in later project phases, if there are any.

Three key outputs from the EGM for a project are the Geological Model, the Geotechnical Model and a Geohazard Assessment. These are described briefly in Section 3 following.

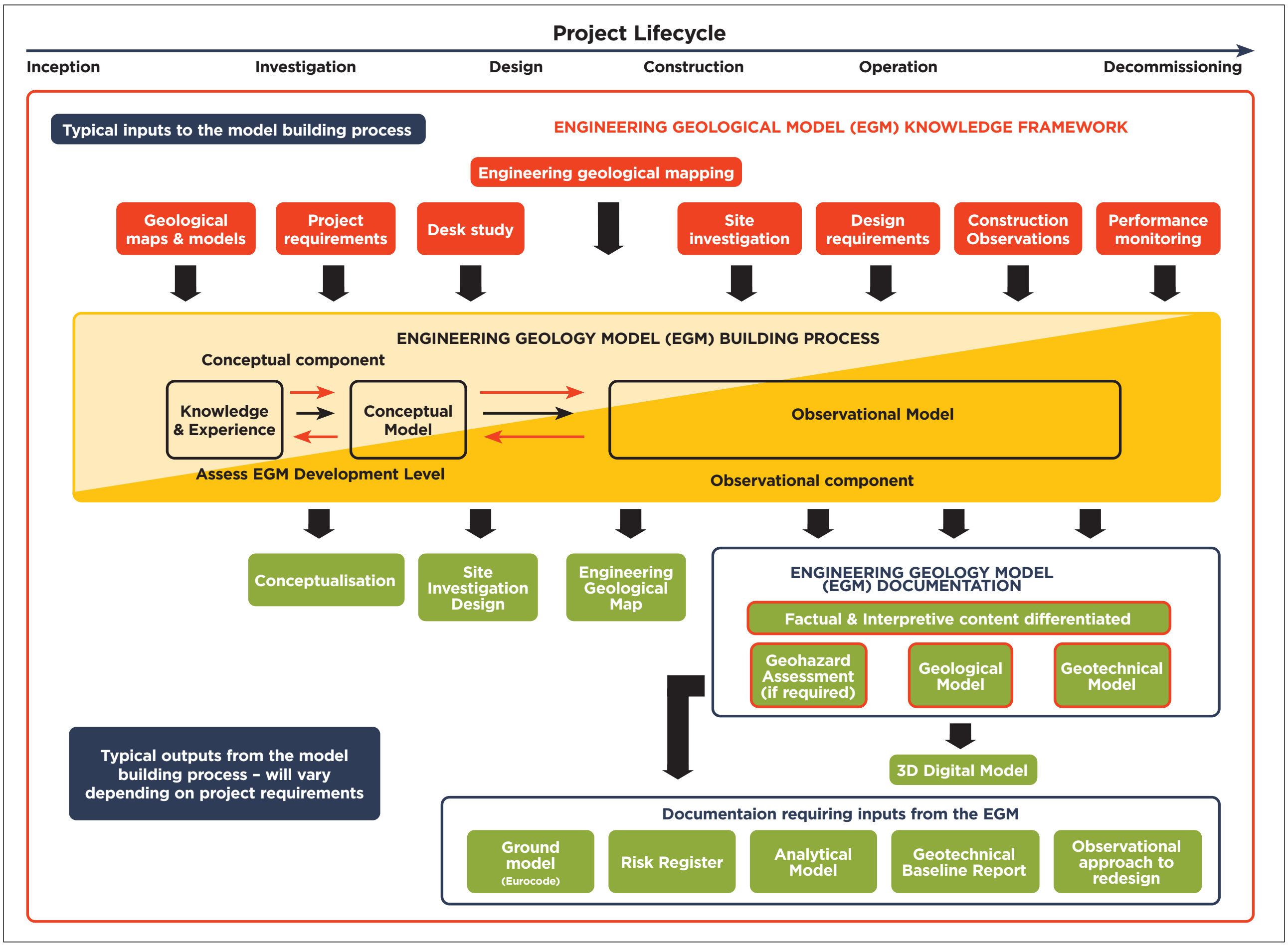


FIGURE 5.1. Engineering Geological Model Development Through the Project Lifecycle (IAEG Commission 25)

2.1 IT STARTS WITH THE GEOLOGY

The process for engineering geological model building must start by understanding the geology, before any attempts are made at geotechnical characterisation (Parry et al, 2013).

As with all things in geology, and especially when developing a geological model, it is important to start with the big picture (tectonic scale) and progressively concentrate down to site scale making sure that all the relevant features are captured. An excellent guide to this approach is presented in Fookes et al. (2001) *Total geological history: A model approach to understanding site conditions.*

The principle of the Total Geology approach is that the geotechnical characteristics of a particular site are the product of its geological conditions and history, its geomorphic conditions (including human influences) and climatic conditions (both past and present).

2.2 CONCEPTUAL MODEL

The preliminary, or conceptual engineering geological model is built up by looking at geological maps, ideally at a variety of scales, finding and reading relevant geological publications, incorporating local geological knowledge and adding general geological knowledge and experience of what might be anticipated in similar ground conditions.

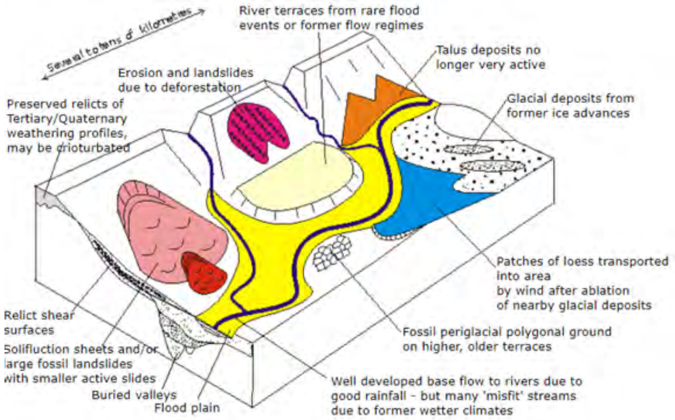
The conceptual model should initially be used to form the basis for planning the site investigations (type and location) which will be used to develop the observational model, discussed in the following section.

An example of development of a conceptual ground model for the Broken River Landslide is provided in Figure 5.2 (following page) and involves:

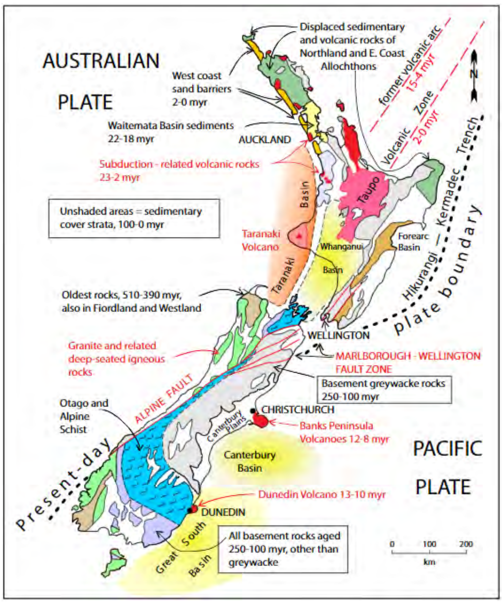
- A regional scale understanding of the tectonic setting, basement geology, historical seismicity / active faults and / or volcanic activity as appropriate :
- Understanding of the local scale geology to appreciate the distribution of faults and rock types that may be present in the area, and
- Consideration of the geomorphic history of the site.

Only once this thought process has been completed are subsurface investigations considered, scoped and completed, to resolve (or at least, reduce) uncertainties in the conceptual model.

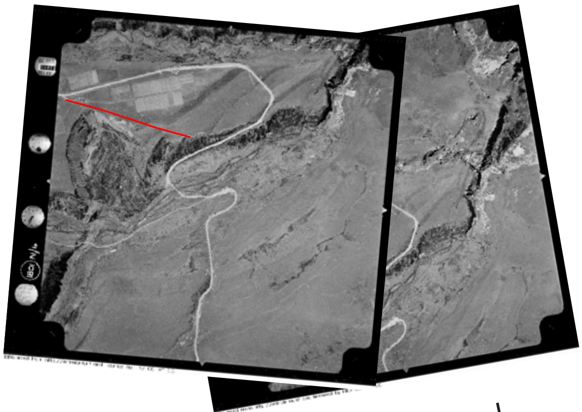
(A). In this example, from inland Canterbury, a large scale landslide (circled) is apparent close to SH73



(D). Conceptual Geomorphic history: Temperate and periglacial landforms that could be expected (from Fookes et al 2001)



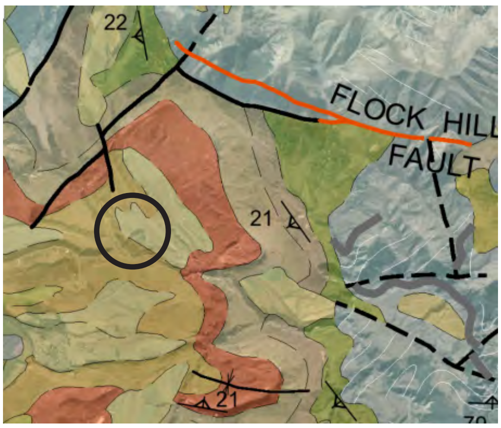
(B). The tectonic setting suggests the region is dominated by strike slip faulting, with basement Greywacke rock (figure from Balance, 2012)



E. Historical Aerial Photographs suggest a possible lineament on the north-eastern side of the landslide (shown in red)

Questions to be addressed in the development of the Observational Model, via field investigation (surface mapping and subsurface drilling)

1) **Is the possible lineament real, and if so**
2) **Is the lineament an extension to the cross-cutting fault?**
3) **To what extent does it control movement of the landslide (groundwater, low strength fault gouge)**
4) **Or is there low strength soil/rock material ('claystone') present below outwash gravel?**
5) **Or both?**



(C). 1:250,000 geological map suggests folded and faulted Tertiary rocks, with the landslide hosted within the White Rock Coal Measures, described as 'weak claystone, siltstone and sandstone', overlain by Pleistocene outwash gravels. An inactive fault system is located to the NW, with a cross-cutting fault extending broadly towards the landslide. Basement Greywacke is located towards the east to northeast of the landslide.

FIGURE 5.2. Development of a Conceptual EGM

2.3 OBSERVATIONAL MODEL

Once a conceptual EGM has been developed, work on the observational model can begin. This model builds on the conceptual model, but it is based on the observed and measured distribution of engineering geological units and processes, related to the surface and sub-surface observations (boreholes, test pits, face logs, geophysics etc).

Like all models, further refinement may be necessary if the engineering questions have not been satisfactorily answered. For example, further investigations into the depth to fresh rock may be required if the weathering profile proves to be deeper than expected, or perhaps sheared material is present in the borehole core which might point to the presence of some relict slope instability, which has not been observed (or might have been missed) in the initial assessment of the surface geomorphology.

3 MODEL OUTPUTS

3.1 GEOLOGICAL MODEL

The Geological Model provides the distribution in 3D space of engineering geological units and geological structures (for example, bedding orientation, faults, sink holes etc.) that are present at the site and are of relevance to the project. As illustrated in Figure 5.3, for landslides and slope stability assessment these could include:

- Material displaced in the landslide mass
- A number of *in situ* geological units that could include:
  - Relatively weak units that could contribute to the displaced mass by future failure, or
  - Relatively strong units that may control or influence the extent of failure
- The location of the basal shear surface of the landslide
- The location of the groundwater table(s)

3.2 GEOTECHNICAL MODEL

As described in IAEG Commission 25, the Geotechnical Model presents the engineering characteristics and geotechnical parameters of every relevant aspect of the Geological Model. For every engineering geological unit identified, an engineering description and geotechnical parameters should be provided. The range of material properties should also be provided.

The process of assigning material parameters commences during development of the conceptual engineering geological model. However, parameters assigned at this stage will likely have considerable uncertainty. Site investigations involving *in situ* and laboratory testing should refine the relevant geotechnical parameters that are then taken forward into any slope stability analysis.

ENGINEERING PARAMETERS

The geotechnical model should *always* include the interpreted engineering parameters for the materials involved which are relevant to the engineering problem at hand. These might include γ, c', ϕ', GSI and Q for rock masses, etc.

The adopted geotechnical parameters should always be agreed in consultation between the engineering geologist and the geotechnical engineer.

3.3 GEOHAZARD ASSESSMENT

The geohazard assessment output from the EGM considers the geological and geomorphological processes or phenomena that can adversely impact a project. For landslides the geohazard assessment could include:

- Size (or magnitude) and frequency of failure
- Expected triggering events for failure (earthquake, rainfall, construction activities etc.)
- Expected extent of movement (if this is possible, although this may be based on later analysis)
- Displacement velocity and runout distance for rapid landslides
- Location and construction details for structures or infrastructure that might be impacted by the landslide/s.

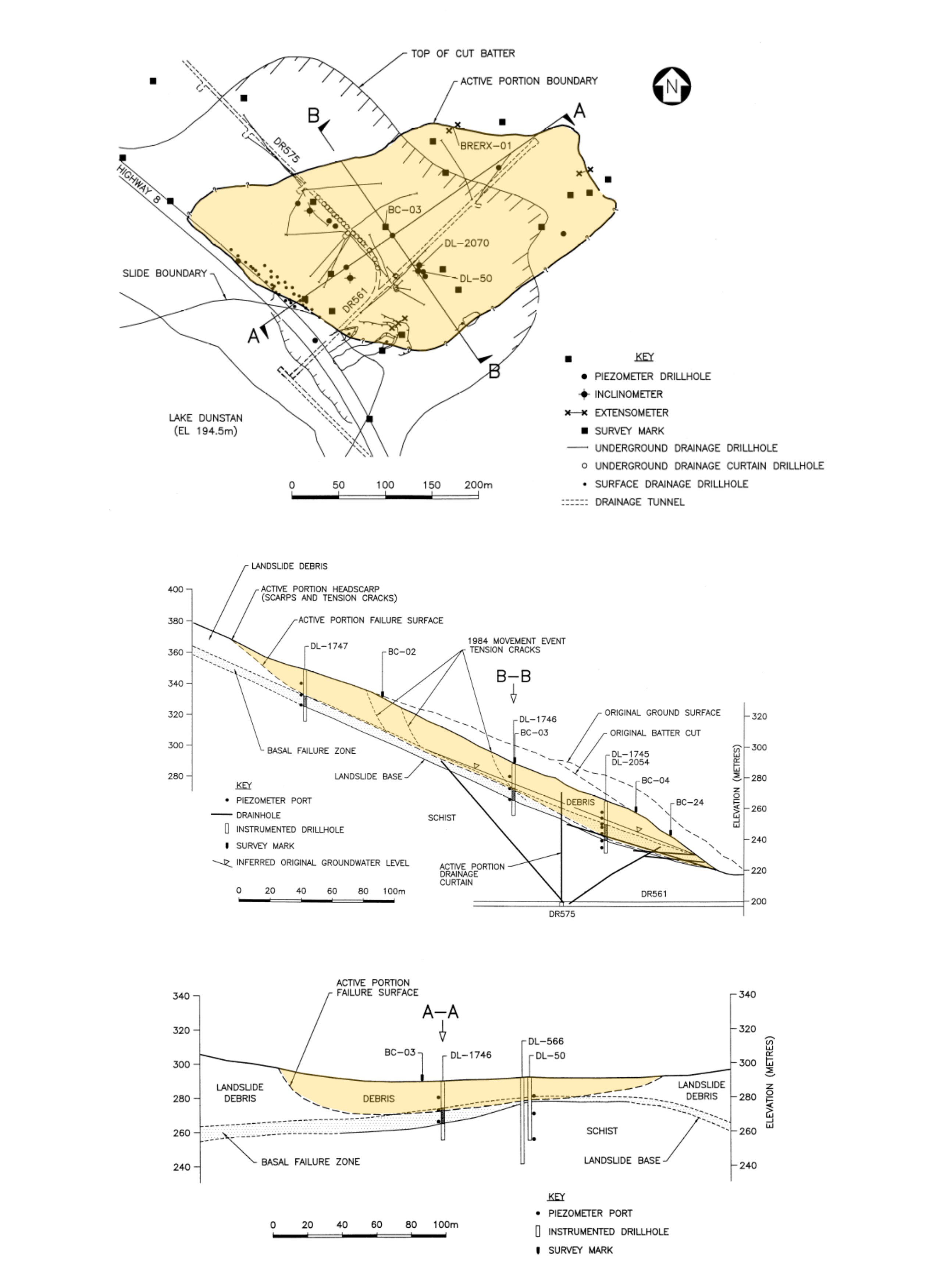


FIGURE 5.3. Example Geological Model outputs as a plan and interpreted sections (modified from Macfarlane et al, 1999)

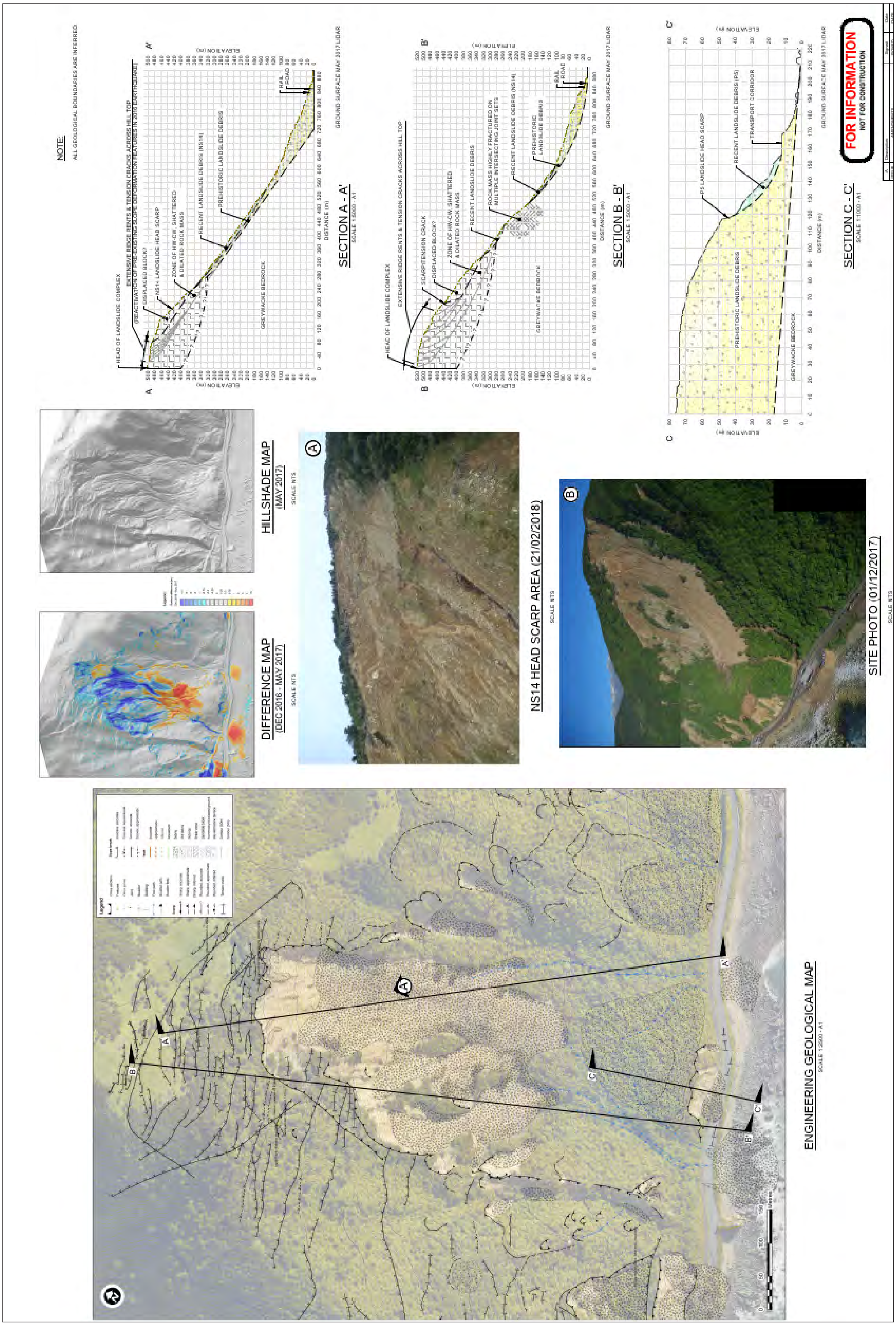


FIGURE 5.4 Example Geohazard Model for a Debris Slide/Avalanche Complex (NCTIR)

4 LEVEL OF DEVELOPMENT

The level of development of the EGM is a function of the geotechnical complexity of the site and what IAEG Commission 25 calls the 'project' complexity – which in some cases may be existing infrastructure (a residential house or a highway, for example). Guidance on the level of development of the EGM is provided in Figures 5.5 and 5.6.

	Geotechnical complexity of the ground that could influence the project – as indicated by the conceptual model developed in accordance with these Guidelines		
Project Complexity ##	**SIMPLE/UNIFORM:** Gently dipping or horizontal strata, uniform soils, no geohazards, few geotechnical constraints	**MODERATE/ VARIABLE:** Variable folding and/ or faulting, variable soils, unconformities, few geohazards, some potential geotechnical constraints	**COMPLEX/HAZARDOUS:** Highly variable folding and/or faulting, deep irregular soils, unconformities, considerable geotechnical complexity, significant geohazards such as major landslide complexes, active faults, karst or the potential for geohazards magnitude and/or frequency to be increased by the project
Minor engineering development, small footprint, low consequence of failure	Level 1	Level 1	Level 2
Medium sized engineering development, with medium consequence of failure	Level 2	Level 2	Level 3
Major infrastructure, large linear projects, regional studies, high consequences of failure	Level 3	Level 3	Level 3

FIGURE 5.5. EGM development levels related to project and geotechnical complexity. From IAEG Commission 25.

	Level 1	**Level 2**	**Level 3**
Specialist studies	None	None	Commision separate geohazard studies (where applicable) Possibility of specialised geological studies Possibility of ground/structure interaction studies
Mapping	Minimum of site visit, reconnaissance mapping, engineering geological sketch map/cross section of site	Engineering geological mapping including cross sections of site and surrounds	Engineering geological mapping including cross sections of project site and surrounds, at a variety of scales
Subsurface investigations	Single stage minor subsurface investigations for example, trial pits, boreholes as appropriate	Subsurface investigations as appropriate using boreholes, cone penetrometer tests, geophysics etc. Instrumentation	Multistage subsurface investigations using methods such as boreholes, *in situ* testing, geophysics etc., instrumentation and long-term monitoring as appropriate. Base line data collection
Laboratory testing	Limited or no laboratory testing	Laboratory testing as appropriate	Extensive and possibly specialised laboratory testing as appropriate
Documentation	Documentation of the EGM in a simple combined factual and interpretive report	Documentaion of the EGM in factual and interpretive reports	Documentation of the EGM in factual and interpretive reports. Consideration of 3D digital visualisation
Team	Possibly a single individual responsible for the works	Small team of engineering geologists and geotechnical engineers responsible for the works	Large multi-disciplinary group responsible for the works
Review	Internal Review ++	Internal/External Peer Review ++	External Review/Panel of Experts ++

FIGURE 5.6. Guidance on scope requirements for EGM development levels. From IAEG Commission 25

In most cases, landslides will have ‘Moderate/Variable’ or ‘Complex/Hazardous’ geotechnical complexity. As described by IAEG Commission 25, *Project complexity is subjective; low and medium consequence of failure would typically be limited to financial impacts; high consequence of failure would typically be associated with loss of life.* Failure is when the project does not perform in accordance with the design/specified performance.

It is important to note that Level 2 and 3 EGMS both require detailed mapping, essential for understanding landslide mechanism and risk, as part of the development process. Geomorphic, or engineering geological mapping techniques are described in Part 4.

5 CONVEYING UNCERTAINTY

UNCERTAINTY ARISES FROM:

Conceptual Uncertainty. ***We don’t know what we don’t know.***

Is the conceptual model right and does it reflect the geotechnical conditions? (geological knowledge, API, mapping, previous experience)

Observational Uncertainty. ***We don’t know what we don’t see.***

Variability in ground conditions, density and locations of investigations, measurement accuracy?

Engineering geological models (EGMs) comprise both conceptual ideas and observational data. Any EGM will always remain a simplification of reality and therefore has some degree of uncertainty associated with it. Uncertainty arises from:

1) conceptual uncertainty - which can be reduced only if more knowledge is incorporated into the model; and
2) observational uncertainty - which can be reduced by acquiring more observations.

It is important that this uncertainty is well conveyed. For landslides this might involve:

- Uncertainty of extent of the landslide mass
- Limited data (surface and subsurface)
- Limitations in the knowledge of the location of the basal shear surface
- Uncertainties, including natural variability in material parameters
- Uncertainties around the depth and role of groundwater
- Uncertainties in terms of bounce height and energy for falling boulders, or friction of downslope terrain for debris flows
- Uncertainties around prior movements and their triggers

For simple projects, the use of appropriate symbology on the geological cross section (or sections) with some description of the uncertainties in the accompanying text may be sufficient. More complex projects will need greater effort – heat maps, use of risk registers etc.

6 REFERENCES

Baynes, F. J and Parry, S (2022). *Guidelines for the Development and Application of Engineering Geological Models on Projects.* IAEG Commission 25 Publication No. 1.

Fookes et al. (2001). *Total geological history: A model approach to understanding site conditions.*

Macfarlane, D.F., Crampton, N.A., Watts, C.R. (1999). Movement History and Sensitivity to Rainfall of part of Brewery Creek Slide, New Zealand. 8th Australia New Zealand Conference on Geomechanics, Hobart.

Parry, S, Baynes, F. J., Culshaw, G., Eggers, M., Keaton., J.F, Lentfer, K, Novotny, J and Paul. D. (2013). Engineering geological models: an introduction: IAEG commission 25. Bull Eng Geol Environ DOI 10.1007/s10064-014-0576-x

PART 6

RISK ANALYSIS

Locomotive buried in Debris Flow material, Kaikoura Coast following Cyclone Allison (1975)

PART 6 – RISK ANALYSIS

THE INTERSECTION OF HAZARD, EXPOSURE, AND VULNERABILITY EQUATES TO RISK (STROUTH ET AL 2023)

1 INTRODUCTION

Risk assessment is the key process in determining how likely a hazard (in this case, landslide activity) is to damage infrastructure, buildings, or result in injury or death of people. 'Risk' can be defined (de Vilder et al, 2020) as:

A measure of the probability and severity of an adverse effect to life, health, property or the environment. Quantitatively, Risk = Hazard × Potential Worth of Loss. This can be also expressed as 'Probability of an adverse event times the consequences, if the event occurs'

The assessment of risk provides Stakeholders[1] with a basis for a systematic understanding of whether the potential for landslide movement to affect their objectives is tolerable or whether action to lower the risk is required. Done in a consistent and rigorous manner, risk assessment serves to identify preferred courses of action for risk management. It can also serve as a tool to communicate the level of risk to other Stakeholders. Communicating the concept of risk of a specific landslide hazard to non-technical stakeholders is critical to maximising the understanding and informing choices by key decision makers.

Most current practices in landslide risk assessment are based on the *Australian Geomechanics Society Guidelines for Landslide Risk Management* (2007a - e). These well-known guidelines for landslide risk management, developed following the Thredbo disaster in 1997, provide a comprehensive body of work which should be referred to for further detail. Risk concepts are also clearly explained in Section 4 of the Landslide Planning Guidance (de Vilder et al, 2023) and are not addressed to the same level of detail in this document.

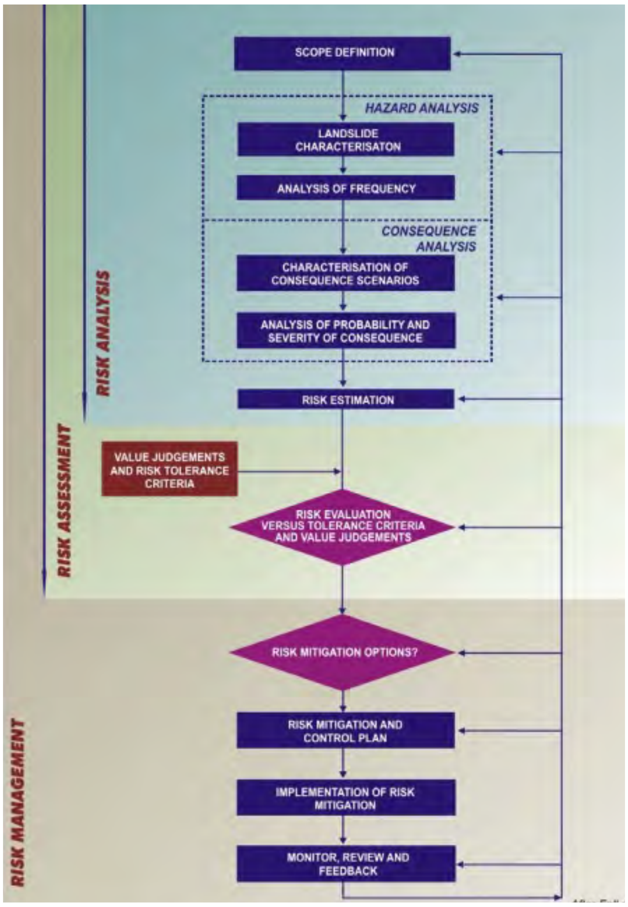


FIGURE 6.1. Framework for Landslide Risk Management (AGS, 2007a)

[1] Stakeholders can include landowners, territorial authorities, asset owners, etc involved in or impacted by the decisions that are made.

Figure 6.1 illustrates the essential elements of a risk assessment process. Landslide risk assessment typically involves two main steps: risk analysis and risk evaluation, as outlined on the following pages.

Risk Analysis involves two discrete elements:

- **Hazard Analysis,** which involves identifying the potential for slope instability to occur (steep slopes, unstable soils) and the potential triggers for movement to occur (heavy rainfall, ground acceleration under seismic events, removal of toe of slope for example); and
- **Consequence Analysis,** which involves identifying the elements at risk, such as buildings, infrastructure, and natural resources, and evaluating their potential for damage from landslide impact. As an example, a house situated below a slope that could fail rapidly will be more vulnerable to catastrophic damage than one sitting on an existing landslide that moves slowly under residual strength conditions (Figure 6.2).
- **Risk Estimation** involves combining the hazard and consequence assessments to evaluate (or quantify) the risk posed by landslides in the study area.

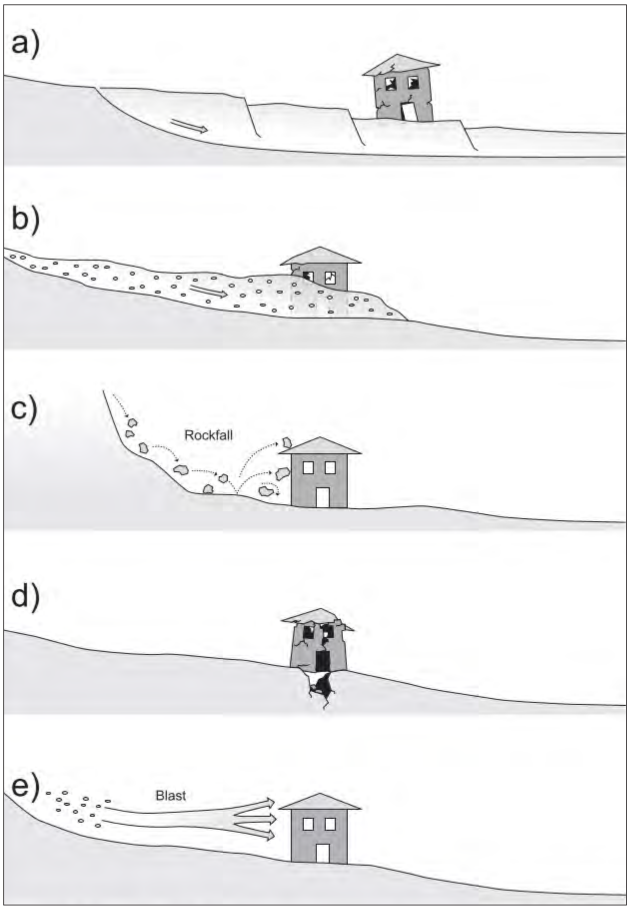


FIGURE 6.2. Risks posed by landslides depend on the style and velocity of movement. (a) shallow translational or rotational movement (b) debris flow (c) rock fall (d) subsidence and (e) rock avalanche (Papathoma-Kohle and Glade, 2013)

Risk Assessment involves assessing the level of risk against tolerability criteria, normally over a particular timeframe. Some examples of tolerability criteria are provided below, and more detail is provided in Part 7 of Unit 1.

- A required level of resilience for a highway against blockage (above road) or evacuative failure (below road), e.g. loss of a lane once in ten years.
- A minimum design life of a house (50 years per the Building Act 2004, unless considered under Section 113 of the Act).
- The tolerable likelihood of a death (for a number of agencies, 1 life lost every 10,000 years (or 10^{-4}) for a specific individual is often adopted; see for example Taig et al, 2012).
- Other client or regulator defined timeframes applicable to their specific objectives.

2 QUALITATIVE VERSUS QUANTITATIVE RISK ASSESSMENT

Qualitative Risk Assessment is a process of evaluating the potential risks associated with landslides based on expert judgment and qualitative observations ('Low' 'Medium', 'High' for example). It is a useful tool when quantitative data is simply not available or when there is a lack of knowledge or resources for conducting a more rigorous analysis. It can also be used as preliminary screening for deciding whether more comprehensive analysis is required.

Semi-Quantitative assessments are essentially a qualitative assessment in which the likelihood is linked to an indicative probability and / or the consequence is linked to a measurable quantity (for example, the cost of damage).

Quantitative Risk Assessments are based on more detailed[1] investigations of the landslide characteristics to derive mathematical values of triggering probability, the runout path of the landslide that leads to exposure of people or assets and their vulnerability in terms of loss of life or economic damage if structures such as houses, or infrastructure assets are struck. Risk is expressed as a quantity, for example, an annualised probability of a fatality of 1.5×10^{-4}.

The risk assessment method chosen should take into account the purpose, element/s at risk, as well as the stage of the assessment:

- As an initial stage, a qualitative risk method could be considered where absolute risk levels are not yet required, or there is insufficient data to support more detailed assessment.

[1] While it is preferable that investigation data is used to support a quantitative risk assessment it may be helpful to get a feel for the level of risk by undertaking a sensitivity analysis for the various input parameters at an early stage in the assessment.

- Where there is sufficient data, risks are likely (or have been qualitatively assessed) to be high, or life risk is being assessed, more sophisticated, quantitative assessment, should be considered.

2.1 QUALITATIVE AND SEMI QUANTITATIVE RISK ASSESSMENTS

Both these forms of risk assessments can provide valuable insights into the potential risks associated with landslides and help decision-makers to prioritise their resources and take appropriate actions to mitigate the potential impacts of landslides. However, they may be subject to biases and uncertainties inherent in expert judgments and subjective observations. In addition, it is difficult, or not possible, to capture uncertainties associated with the hazard and / or its consequences.

Qualitative risk assessment tends to be subjective. It focuses on identifying hazards to measure both the likelihood of a specific hazard event occurring and the impact it will have should it occur. The goal is to determine the severity of the consequences. Results are typically displayed in a colour-coded risk assessment matrix. By assigning a score, the matrix becomes a semi-quantitative 'relative risk rating'. This type of rating is used where it is not possible to assign all elements of the risk analysis a meaningful numerical value. See Figure 6.3 for an example.

As indicated in AGS 2007c, a semi-quantitative analysis or a qualitative analysis may be appropriate:

- As an initial screening process to identify hazards and risks which require more detailed consideration and analysis.
- When the level of risk does not justify the time and effort required for more detailed analysis.
- Where the possibility of obtaining numerical data is limited such that a quantitative analysis is unlikely to be meaningful or may be misleading.

2.1.1 Qualitative and Semi Quantitative Risk Assessment Systems used in New Zealand

Several risk assessment systems or frameworks have been developed and are widely used within New Zealand. They all follow the basic methodology outlined in the sections above of assigning a likelihood of event occurrence and the potential consequences of that event if it occurs. The following sections provide a summary overview of the systems commonly used in slope assessment practice.

Australian Geomechanics Society (2007)

The AGS qualitative risk system has become widely used in New Zealand as an indicator of property risk. While it remains a very useful framework, care needs to be taken in its application, as the measures of recurrence interval consider very long-time intervals, such that it is not well aligned with Building Code requirements. For example, it is entirely possible to generate a 'Very High' Risk for an IL2 structure that can suffer very significant damage under a 1:500-year return period ULS earthquake. 'Very High' risks are described in the AGS as being unacceptable without treatment, which is not compatible with the Building Code: the IL2 structure in this example would be code compliant.

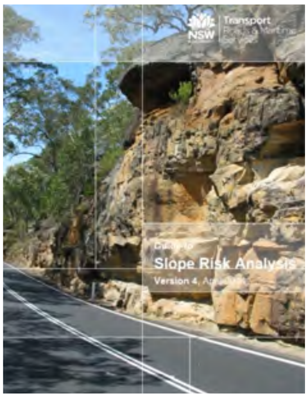
If the AGS framework is used, it is recommended that measures of likelihood are assigned using the descriptions provided and not the annual probability of failure or recurrence interval.

		CONSEQUENCE				
LIKELIHOOD		Insignificant 1	Minor 2	Moderate 3	Major 4	Extreme 5
	Almost Certain 5	Medium 5	High 10	Very high 15	Extremely high 20	Extremely high 25
	Likely 4	Low 4	Medium 8	High 12	Very high 16	Extremely high 20
	Moderate 3	Low 3	Medium 6	Medium 9	High 12	Very high 15
	Unlikely 2	Very low 2	Low 4	Medium 6	Medium 8	High 10
	Rare 1	Very low 1	Very low 2	Low 3	Low 4	Medium 5

FIGURE 6.3. Example of a colour-coded qualitative/semi-quantitative risk assessment matrix. (Auckland Code of Practice for Land Development and Subdivision, Auckland Council (2023))

Department of Conservation (2020)
DOC's *Guidelines for Natural Hazard Risk Analysis on Public Conservation Lands and Waters* (de Vilder et al, 2020) were developed by GNS for DOC and are largely focussed on landslide life-safety risk for users of huts, tracks and other DOC assets. Three levels of assessment are defined – Preliminary, using qualitative methods, Basic and Advanced, both using quantitative methods. Advanced level assessments are restricted to high risk sites such as high altitude alpine huts.

The Preliminary risk assessment is used to determine the need for, and appropriate level of, more detailed analysis.



New South Wales Roads and Maritime Services (NSW RMS)
NZ Transport Agency, Waka Kotahi (WK), has introduced a semi-quantitative system for analysing existing slopes along its highways based on a system for Slope Risk Analysis developed by the NSW Government Transport, Roads and Maritime Services. This guide was developed primarily for assessing slope risks to existing highway infrastructure but can be used for new roads. It is largely based on information gathered from visual assessments of slopes by geoprofessionals specifically trained in the use of the system.

The semi-quantitative risk assessment process uses rating scales to assign alphanumeric values to:
- the likelihood of a slope failure "interacting" with road users and
- the consequences of that interaction.

Consequences are expressed in terms of the loss of life and do not consider property damage in the WK modified Guide, however property could be considered in applications outside of WK requirements.

The components of likelihood of a road user being struck by falling debris or (for example) running into an area of subsidence produced by embankment failure are split into the probability of the event triggering (termed the probability of detachment as an annual probability of occurrence, or Pd) and the travel probability (Pt); examples of which are the probability of the motorist either being struck by rockfall or striking an obstruction in their travel lane such as a stationary rock, or the probability of an embankment failure encroaching onto the carriageway (often termed an 'underslip'). These Pd and Pt components are split into coarse order of magnitude steps (e.g. 0.1, 0.01, 0.001 etc.) which reflect the level of accuracy of a visual assessment method.

The consequence assessment then considers the temporal probability (T) of a road user being present at the time of the event and their vulnerability (V) with both components being assigned a rating value. When combined to derive a consequence, the process then uses the event likelihood and consequence to derive the "Assessed Risk Level" (ARL), which ranges between 1 (highest) to 5 (lowest).

This value is compared against the target ARL set by Waka Kotahi. A target level value of ARL 3 is typically specified. Values below this (ARL 1 and 2) indicate a need to consider mitigation measures.

The advantages of this risk assessment system are:
1. It explicitly acknowledges the difficulty of deriving better than order of magnitude annual probability for a slope event occurrence.
2. It enables relatively rapid assessment of many slope situations along highways (or other assets) to identify those that should be prioritised for mitigation funding.
3. It can be used to assess property risk as well as lives risk.
4. It has been developed to allow a wider usage other than roading networks.

2.1.2 Qualitative Risk Assessment Frameworks used by Territorial Authorities

Several Qualitative or Semi-Quantitative risk assessment frameworks have been developed or adopted by Territorial Authorities around New Zealand. Typically, these are based on the AGS (2007) guidelines. While they provide relatively straight forward methodologies, there are some significant differences in the descriptors of likelihood and consequence so they should be used with care to ensure that the outputs are consistent with wider established criteria. Examples include:
- Auckland City Council's Code of Practice for Land Development and Subdivision
- Whangarei District Council's Flow Chart for Slope Stabilisation. The WDC system initially utilises the AGS semi-quantitative framework, but then considers target factors of safety to be achieved for the project slope area through the use of slope stabilisation measures, e.g. drainage measures or buttressing.
- BOP Regional Council (Appendix L of the BOP Policy Statement): The measures of consequence under this framework are more suitable for area-wide assessments, and not individual properties. Even when this is considered, measures of consequence to lives result in risks which are not consistent with normally tolerable criteria. Use of this particular part of the framework is not recommended.

- Wellington City and Porirua City Councils Quantitative Risk Assessment Framework (Justice et al, 2006) – this framework is specifically applicable to roading networks but does provide a description of consequences to private property (houses). It does not consider a loss of life consequence.

2.1.3 Life Risk Assessment using Qualitative Frameworks

Life risk typically requires quantitative analysis and is a requirement under AGS (2007) guidelines. While there are qualitative schemes that also consider life loss, these frameworks should be used with caution such that the risk outputs are consistent with societally acceptable criteria (see Taig et al, 2012 for further detail).

2.2 QUANTITATIVE RISK ASSESSMENTS

Quantitative risk analysis (QRA) can be systematic and objective and enables the risk from multiple hazards at the same location to be compared and combined. The quantitative risk metric provides a consistent basis from which to determine risk tolerance and acceptability levels and provides stakeholders, planners and policy makers with a sound methodology on which to base decisions.

Quantitative assessments are typically used to assess risk to life, but can be used to assess economic loss, although this is much less common in geotechnical practice. From de Vilder and Massey (2020), the annual risk of loss of life to an individual can be calculated as:

$$P_{(LOL)} = P_{(L)}\ x\ P_{(T:L)}\ x\ P_{(S:T)}\ x\ V_{(D:T)}$$

Where:

$P_{(LOL)}$ Is the annual probability that a person will be killed.

$P_{(L)}$ Is the annual probability of the landslide occurring.

$P_{(T:L)}$ Is the probability of the landslide reaching the element at risk.

$P_{(S:T)}$ Is the spatio-temporal probability of the person at risk (the proportion of the year that the person is in the path of the landslide when it reaches or passes the element at risk).

$V_{(D:T)}$ Is the vulnerability of the person to the landslide event (the probability that the person will be killed if impacted by the landslide).

Quantitative risk assessments provide a greater ability to consider uncertainty than do qualitative assessments. However, like many other analyses the quality of the output is dependent on the quality of data inputs, which may be difficult to assess (in particular, probability of failure) without significant effort and therefore greater cost.

Much greater detail on quantitative risk assessment is provided in, for example, de Vilder et al (2020), and Massey et al (2012), which should be referred to for further information.

2.3 SOCIETAL RISK

Societal risk is scenario-based and is essentially the risk of multiple fatalities triggered by a single event such as an earthquake or rainfall event with widespread or large-scale consequence - for example, a large landslide, or several smaller landslides.

Two broad quantitative risk metrics are considered for societal risk (de Vilder et al, 2023):

1. fN pairs or curves, which are calculated by linking some specific scenarios that relate the number of people who might be in a group with the likelihood of them being killed if a hazard of a given magnitude were to occur (N) and the probability of the hazard occurring (f).
2. Annual Probable Lives Lost (APLL): The product of probability (f) and number of fatalities (N) yield probable life loss (PLL). APLL describes the expected number of deaths over a year.

2.4 RISK ACCEPTABILITY

For qualitative and semi-quantitative risk assessments, the level of risk that is acceptable varies depending on the tolerance of the stakeholders. However, in most frameworks, ‘moderate’ or ‘medium’ risks are typically considered as broadly tolerable, often with the requirement for risk maintenance measures to be implemented.

In quantitative assessments of lives risk, societal acceptance criteria need to be considered – society has a lower tolerance for hazard events that result in multiple fatalities. Risk-based performance criteria are discussed in Section 3 of Part 10.

3 ESTIMATING LANDSLIDE HAZARD

Estimating the likelihood / probability of landslide movement is a key objective of a detailed landslide risk assessment. This requires investigating the *hazard* the landslide presents with activities including field investigations, examination of past failures to define magnitudes and probability relationships and estimating the runout extent of different size events. It often requires the bulk of the effort in estimating landslide risk, particularly for events with a low probability/high consequence, where estimating the likelihood of a future event lacks sufficient record from which to estimate the likelihood of future events.

Probability estimates need to be based on best estimates of parameters (shear strength, groundwater pressures and failure surface shapes), omitting the use of partial factors on strength and load factors on loads.

Estimation of the landslide hazard must consider both:
1. The probability of failure or probability of hazard occurrence (P_H), and
2. The probability of interaction with the element at risk (e.g. road or house). This is considered as the probability of travel (P_T) in the NSW RMS system, or $P_{(T:L)}$ in de Vilder and Massey (2020).

3.1 PROBABILITY OF FAILURE

The probability of failure, or probability of hazard occurrence (P_H), can be considered as movement that poses a consequence to infrastructure, the project, or people. In some cases, a degree of movement may be acceptable, but might not be in others.

Estimates of the probability of failure are a fundamental part of the geohazard assessment, which is an output from the engineering geological model, discussed in Part 5.

3.1.1 Frequency – Magnitude Assessment

The risk assessment process considers the Likelihood and Consequence of a range of slope movement event scenarios. The geoprofessional should understand and evaluate the full range of plausible potential slope failure scenarios, including their size, speed and run-out distance, and frequency. While these won't necessarily be known accurately, they will be informed by precedent at the site or similar sites, as described in the previous sections.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. The probability or likelihood of a particular event magnitude can be expressed either qualitatively as a single number representing the annual probability of an event (e.g. 0.05 events per year), a cumulative probability of an event of equal to or greater than a particular magnitude (e.g. 1 in 200 years) or qualitatively based using a likelihood descriptor on an accepted scale (e.g. "*unlikely*" or "*certain*") to which a value is assigned.

Magnitude – defines the volume of material or energy released during a particular event. Larger magnitude events will more likely run out further and have a higher consequence to the element at risk, compared to smaller or less energetic events.

Frequency-magnitude estimation relates the volumes of mass movements to specific return periods (or annual frequencies) of their occurrence. In general, larger events will occur much less frequently compared to smaller events (Figure 6.3).

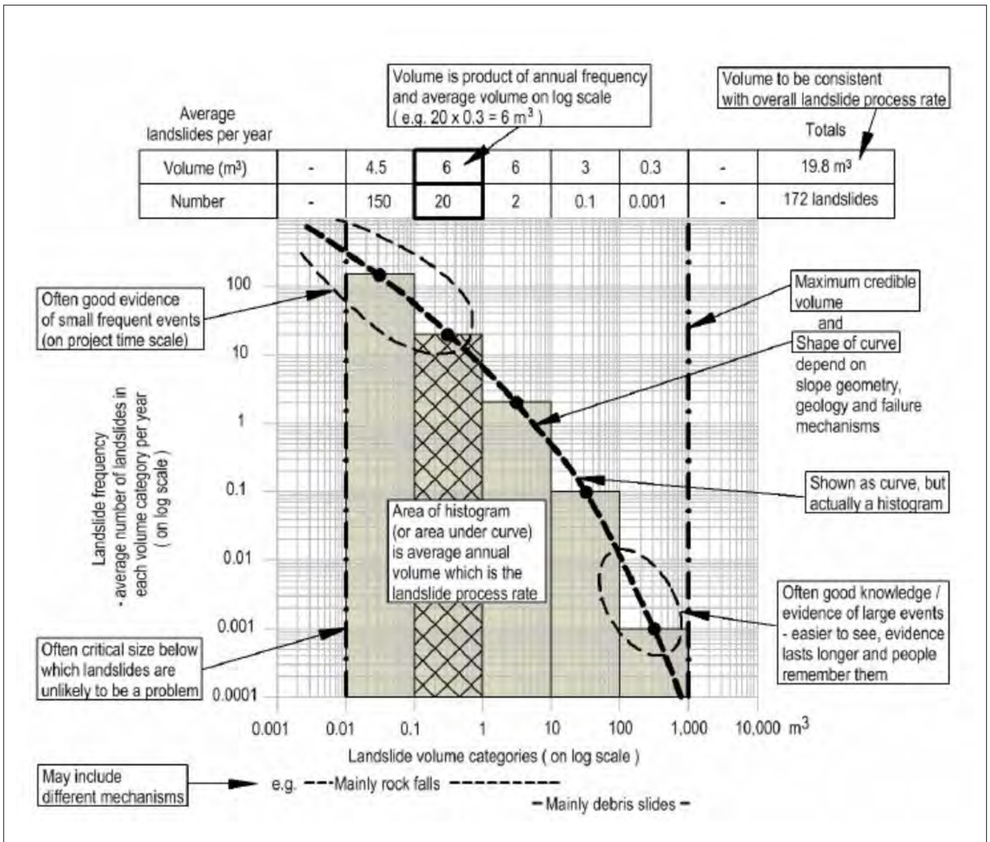


FIGURE 6.3 Generic Frequency/Magnitude relationship model (Baynes and Parry, 2022). In this example the maximum failure volume is 1000 m³ but the principle is applicable to larger landslides too.

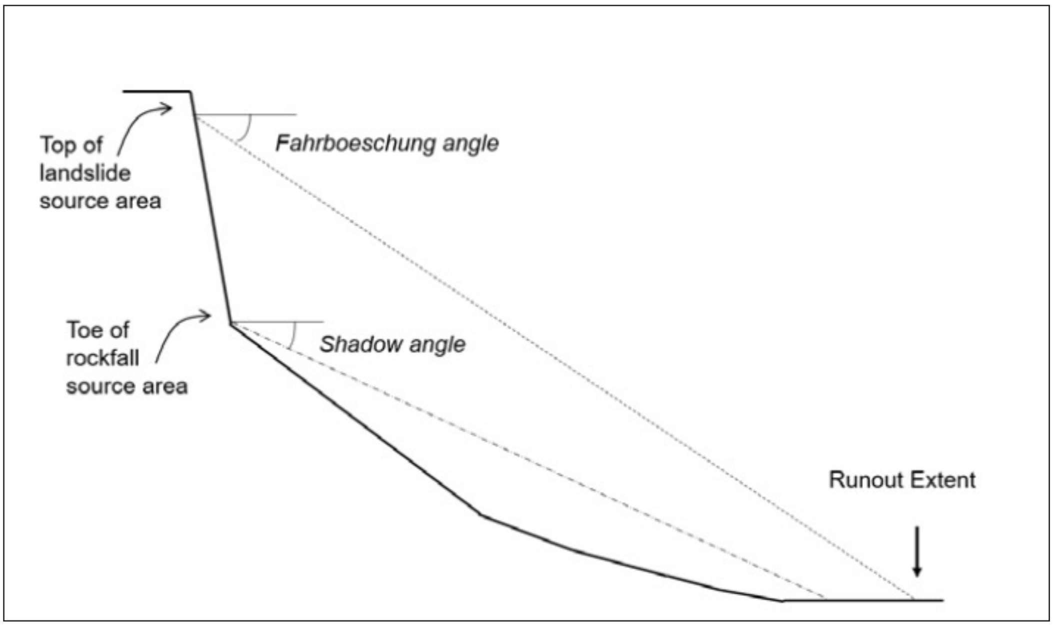


FIGURE 6.4. Conceptual cross-section showing Shadow angle and F-angle (de Vilder and Massey, 2020)

TABLE 6.1. F-angles for channelised flows and open-slope avalanches for different landslide volumes (Brideau et al, 2021)

Landslide Volume (m^3)	10%-Passing Fahrböschung Angle		
	Channelised Flow	Dry Avalanche	Wet Avalanche
10	22	38	29
100	18	36	25
1000	14	34	21
10,000	11	32	18
100,000	9	30	16
1,000,000	7	28	NA*

Establishing the Frequency / Magnitude relationship for the particular landslide hazard is a core element of any hazard assessment but is subject to many uncertainties and is difficult to assess as detailed records of landslide events and their magnitude are rarely available.

The frequency of smaller events may be able to be assessed via historical records, assessment of landslide stratigraphy combined with dating processes, vegetation growth etc. Where information is required from more than around 50 – 100 years ago, as will be the case most, if not all, of the time, a number of judgement-based assumptions will be required to develop the Frequency / Magnitude model. Care should be taken where empirical and statistical predictive equations are used as a number of geological, geomorphological, hydrological and land use factors may affect landslide initiation and magnitude which these equations might not take into account.

3.2 PROBABILITY OF TRAVEL

The probability of travel (Pt or $P_{T:L}$) is the probability that the landslide travels sufficiently far that it impacts an element at risk. An element (infrastructure or structure of some sort) already sitting on a landslide is likely to have a high degree of exposure if movement triggers, whereas the same element sitting on a debris flow fan may have its exposure level dictated by debris flow run out extent and its proximity to the flow path. The travel distance depends on the type of landslide; slow moving, deep seated landslides may displace by a small amount (a few metres perhaps) whereas rapid landslides (Rockfalls, Debris Avalanches and Debris Flows) may travel hundreds of metres or even kilometres.

For rapid landslides (debris flows, avalanches and rockfalls, a of travel distance may be estimated using the Fahrboeschung (F) angle or the Shadow (S) angle (Figure 6.4).

Runout distances for rockfall and debris flow slope instabilities are discussed further in Part 8 and 9, respectively, and these should be referred to for more detail. Table 6.1 shows 10% F-angles for channelised flows and open slope avalanches for different landslide volumes. As the values provided in Table 6.1 are 10% passing, there is a 90% probability that the landslide debris travels less distance. This can be used as an initial 'look up' table to determine likely runout distances.[2]

4 ESTIMATING CONSEQUENCE

The consequences of landslide movement are subdivided into the following elements:

Exposure is the likelihood of an element at risk (infrastructure, building or person) being impacted if landslide movement occurs. For permanent structures, this is 100%, but for persons in most cases this will be less than 100%, and for sparsely occupied sites may be significantly less than 100%.

Vulnerability refers to the likelihood of damage / economic loss / injury or loss of life if the landslide event reaches the element at risk.

The degree of vulnerability depends very much on landslide size (volume) and speed of movement, and is well covered in Massey et al (2018). Amongst other things (including the type of building and how it is impacted) vulnerability depends on the style of landslide:

- Debris flows and avalanches where the height and velocity of the moving mass are critical. Here, elements at risk may be people, or buildings.
- Translational or deep-seated failures, which might also need to consider speed, but particularly need to consider likely displacement over the design life of the element at risk. As people are less likely to be harmed, the focus for these types of landslide is more on risk to property and infrastructure than on life safety.

[2] We note that this is for channelised debris flow and open slope avalanche; in other environments topography and vegetation would likely influence these values.

Vulnerability' can be expressed either

- in quantitative terms describing the probability of damage for various hazard intensities (e.g. for a given hazard intensity there is an 80% chance the dwelling will be destroyed: Massey et al, 2018), or
- as a Damage State or Damage Ratio, as described below.

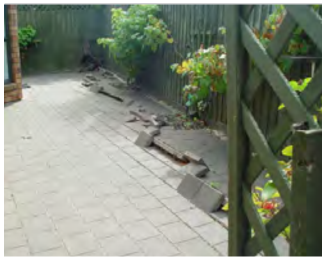
Damage State

The degree of physical damage to property and infrastructure can be considered in terms of a 'damage state', which describes the amount of damage in relation to the ability of the building or infrastructure to function normally. Damage states can be described quantitively and qualitatively and are usually expressed on a scale from no damage (0) to destroyed (1). Generally, the higher the hazard intensity (e.g. landslide debris height, velocity) a building is subjected to, the higher the damage state (Massey et al, 2018). Damage States with New Zealand timber framed house examples are provided in Figure 6.5. Note that there are a number of classifications of building damage state that could be used, depending on building type. Kappos and Papanikolaou (2016) discuss four damage states ranging from 'DS1 negligible structural damage' to 'DS4 Collapse' for unreinforced masonry structures, while Burland (2012) considers six damage states (0-5) based on visible damage to brickwork or masonry walls (0 – Negligible to 5 – Severe). These systems could be considered as alternatives to the information provided in Figure 6.5 depending on the type of building construction. They are widely applied to infrastructure assessments in New Zealand (even though they were developed for building structures).

Damage Ratio

Damage ratio describes economic loss. It is calculated by dividing the cost to repair a damaged asset by the cost of replacing the asset.

FIGURE 6.5 Examples of Damage States for Rapid and Slow Moving Landslides

Damage States	Description (Massey et al, 2018)	Rapid Landslides (Debris Flow and Rockfall)	Intensity Index (modified from Jakob et al 2012)	Slow Moving Landslides	Cumulative differential horizontal or vertical displacement (typical timber frame buildings) (mm) *indicative values only*
0	None: No damage	Debris Flow/Avalanche stops short of building		Building beyond toe of landslide, or landslide remains dormant	NA
<0.1	Insignificant: Minor non-structural damage		Less than 1		<20
0.1 – 0.25	Light: Non-structural damage only		1 -2		20 – 50
0.25 – 0.6	Moderate: Reparable structural damage.		2-10		50 – 200mm
0.6 – 1.0	Severe: Irreparable structural damage.		10 – 100		0.2 – 1.0m
1.0	Critical: Structural integrity fails		>100		>1 metre

Photograph References: Column 3 top – bottom: (1) Roxburgh, 2017. (Dellow et al, 2018); Second and third photos, (2 &3) Nyhane Drive Debris Flow, 2011. (Page et al, 2012) (4) Rosy Morn Debris Flow, Kaikoura Coast (February 2018). Photo courtesy NCTIR. (5) Fatal landslide at Kaiteriteri, 2013 (Massey et al, 2018).
Column 5 top – bottom (1) Toe Slump following 2011 Christchurch Earthquake (Dellow et al, 2011); (2) Toe Slump following 2011 Christchurch, photo courtesy ENGEO; (3) Christchurch 2011 (Dellow et al, 2011); (4) Tahunanui Landslide, Nelson, 1962. (Denton and Johnson, 1996); (5) Abbotsford Landslide, Dunedin, 1979 (Stock photo)

FIGURE 6.8. Example of incipient failure requiring risk assessment to determine immediate, short- and long-term risks. Assessment to consider risk to road users and house resident and maybe other stakeholders (e.g. utility providers). Photo provided by Ross Roberts, Chief Geotechnical Engineer, Auckland Council, 2023.

4.1 SLOW MOVING LANDSLIDES

Slow moving landslides (less than 13 m / month – see Figure 2.10 in Part 2 of Unit 1) rarely physically affect people as they are normally able to escape. Therefore, elements at risk from slower moving landslides are typically buildings and infrastructure (Figure 6.8), where vulnerability should consider the degree of displacement (Figure 6.6). For slow-moving landslides, consideration should be given to the nature and degree of movement over the life span of the structure at risk. For aseismic conditions this will require consideration of historical records and geomorphic assessment. However, it is unlikely that the future deformation will be able to be accurately determined in most situations.

Displacement under seismic conditions can be assessed using Newmark block methods, as described in Part 8.

4.2 RAPID LANDSLIDES

Determining the vulnerability of an element at risk from rapid landslides (debris flows and avalanches) includes assessment of the following parameters:

- **Flow or Avalanche velocity.** The velocity of the moving debris is critical as it governs impact forces on buildings within the runout distance of the landslide.
- **Maximum flow or avalanche depth.** The depth of an impacting flow relates broadly to the width of the flow / avalanche (large flow depths would normally be associated with wider flows) and needs to be considered to assess the vulnerability of structures to damage.
- **Landslide material.** Debris flows or avalanches transporting large boulders and other entrained debris such as trees are expected to have a greater impact on buildings compared to flows of earth or mud for the same height and velocity. While this is a relatively smaller influence compared to velocity and flow depth, particle size will need to be considered as part of remedial works.

Further information on the assessment of the assessment of debris flow hazard is provided in Part 10 of Unit 1.

4.3 ROCKFALL

Vulnerability of buildings and infrastructure to rockfall is usually considered in relation to kinetic energy: the higher the kinetic energy, the more damaging the rockfall is likely to be. Further information on the assessment of rockfall is provided in Part 9 of Unit 1, and in the MBIE (2016) guide, “Rockfall: Design considerations for passive protection structures”.

5 SOURCES OF UNCERTAINTY IN LANDSLIDE RISK ASSESSMENT

The assessment of landslide risk involves the evaluation of a number of factors that contribute to the potential occurrence and consequences of landslides. However, it is important to recognise that there are several uncertainties which include:

- **Data Uncertainty:** One of the primary challenges in landslide risk assessment is the availability and quality of data. Obtaining accurate and up-to-date information on factors such as topography, geology, rainfall patterns, and land cover can be challenging. Incomplete or unreliable data can introduce uncertainties into the assessment process.
- **Prediction Uncertainty:** Predicting the occurrence and timing of landslides is inherently challenging. Despite advances in technology and modelling techniques, there are still many uncertainties associated with the accuracy of landslide prediction models. Factors such as the complexity of geological processes, spatial and temporal scales, and the influence of triggering events (e.g., rainfall) can all introduce uncertainties into landslide predictions.
- **Modelling Uncertainty:** Mathematical or empirical models to simulate and predict landslide behaviour rely on various assumptions, simplifications, and parameterizations, which inevitably introduce uncertainties. The accuracy of the model outputs depends on the quality of input data, model calibration, and the validity of the underlying assumptions.
- **Climate Change Uncertainty:** Climate change may have significant impacts on landslide risk, particularly through changes in precipitation patterns and intensity. However, the exact magnitude and spatial distribution of these changes are uncertain. Climate change projections involve uncertainties, and incorporating these uncertainties into landslide risk assessment can be challenging.
- **Human Factors:** Assessing landslide risk often involves considering human factors such as population density, infrastructure development, and land-use changes. These factors introduce uncertainties due to their dynamic nature and the challenges in predicting human behaviour and decision-making processes.
- **Uncertain Consequences:** Estimating the consequences of landslides, such as property damage, casualties, and economic losses, involves uncertainties. The accuracy of such estimations can vary in time due to variations in population exposure, vulnerability, and the accuracy of damage assessment methods.

Addressing these uncertainties requires ongoing research, data collection, and improvement of models and methodologies. It is important to communicate the uncertainties associated with landslide risk assessments to ensure decision-makers and stakeholders have a clear understanding of the limitations and potential variability in the results.

5.1 SENSITIVITY ANALYSIS

Sensitivity analysis is a central component of geotechnical risk assessment. It involves evaluation of how changes in input parameters or assumptions affect the range of values of risks generated in the analysis. This helps in understanding the uncertainty associated with the risk analyses and increases the confidence in the design or risk management approach. For example, if sensitivity analysis shows that a slight variation in a particular parameter leads to significant changes in risk, it may prompt stakeholders and geoprofessionals to adjust their mitigation strategies accordingly.

Sensitivity analysis also allows critical parameters to be identified: that is, which parameters or factors have the most significant influence on the outcomes. This helps to focus resources and efforts on understanding and managing the most critical risk factors. By understanding which parameters have the greatest impact, geoprofessionals can prioritise their efforts in data collection, testing, and monitoring.

6 FURTHER READING

There is significant documentation on landslide risk assessment available in the literature, much more than can be covered in this part of the guidance. This section is deliberately brief and is intended as an overview for practitioners. More detailed explanation can be found in the referenced documents (among many others).

THE LOW PROBABILITY TIGER – A PARABLE OF RISK

A king offered a challenge to three young men. Each young man would be put in a room with two doors. The young man could open either door he pleased. If he opened the one, there came out of it a hungry tiger, the fiercest and most cruel that could be procured, which would immediately tear him to pieces. But if he opened the other door, there came forth from it a lady; the most suitable to his years and station that His Majesty could select among his fair subjects. So I leave it to you, which door to open?

The first man refused to take the chance. He lived safe and died chaste.

The second man hired risk assessment consultants. He collected all the available data on lady and tiger populations. He brought in sophisticated technology to listen for growling and detect the faintest whiff of perfume. He completed checklists. He developed a utility function and assessed his risk averseness. Finally, sensing that in a few more years he would be in no condition to enjoy the ladies' company anyway, he opened the optimal door. And was eaten by a low probability tiger.

The third man took a course in tiger taming. He opened a door at random and was eaten by the lady.

From WC Clark, in "Societal Risk Assessment", edited by R C Schwing and W A Albers, Plenum Press, London & New York, 1980, p 302.

Like the first man, we can choose to retreat – this eliminates the risk

Like the second man, we can try to work out the risks and be as safe as possible — but we can never be completely safe. There is no such thing as 'No risk'.

Like the third man, we could change the work situation, to choose designs or methods of working which isolate or reduce the hazard. But we can only do this for hazards we know and understand.

THE LOW PROBABILITY TIGER – A CHRISTCHURCH EXAMPLE

Up until the February 2011 Christchurch Earthquake, most geotechnical professionals were unaware of the full range of slope failure scenarios that could occur in Canterbury. Risk assessments of cliffs in the Christchurch Port Hills area would typically consider discrete rockfall as the main hazard. The February 2011 earthquake reminded us that the Cliff Collapse is a plausible failure scenario. The frequency of such events is very low, however the consequence when they do occur can be catastrophic. Two of the five rockfall fatalities occurred in this area.

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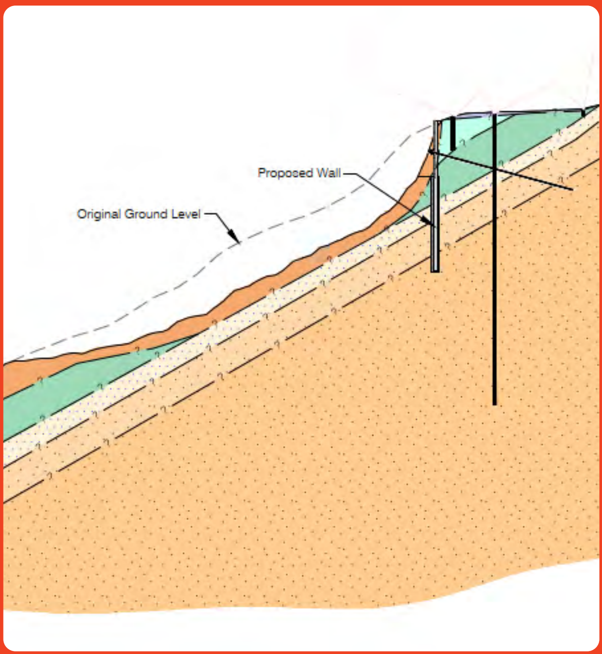
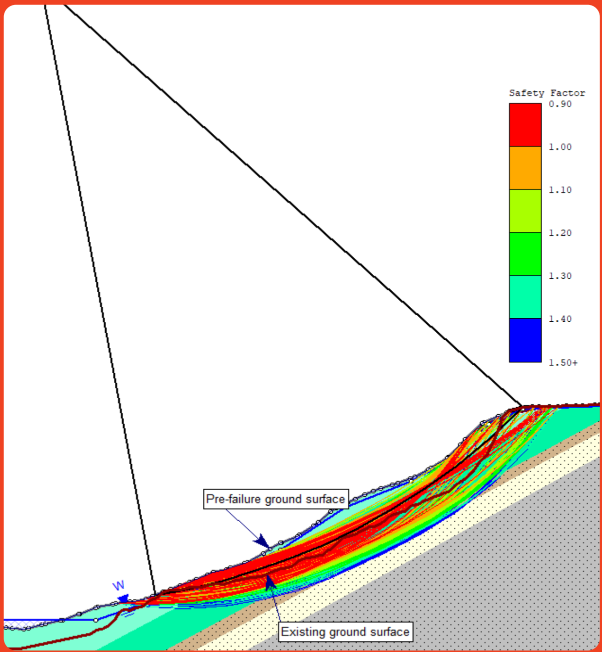
PART 7

SLOPE STABILITY MODELLING

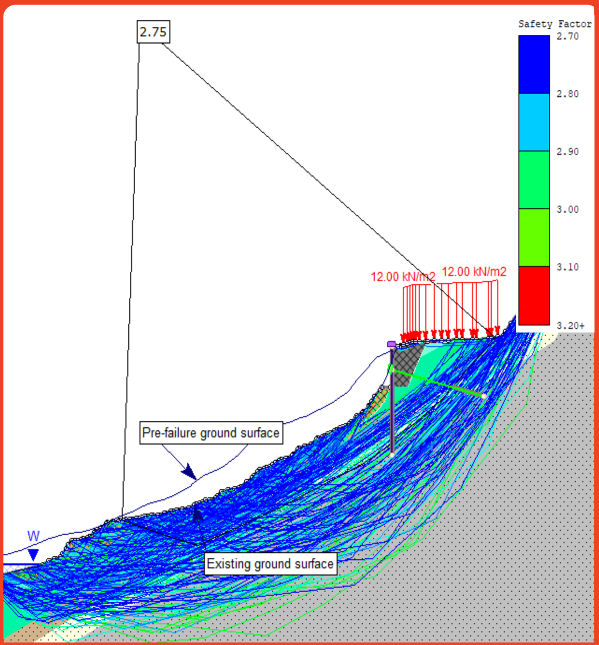
Site Investigation



Engineering Geological Model



Back-Analysis



Analysis of Mitigation

PART 7 – SLOPE STABILITY MODELLING

1 MODELLING AND ANALYSING SLOPE STABILITY

Slope stability analysis is a complex task with multiple facets that need to be considered to arrive at a realistic understanding of the current slope stability and how it may change during a project's life cycle. The content in this Part of Unit 1 introduces the aspects of slope stability modelling that the geoprofessional should consider but is not exhaustive. Additional detail can be found in the Appendix.
Analysing the stability of a slope begins with the site investigations (described in Part 4) undertaken to develop the Engineering Geological Model (EGM, described in Part 5) of the slope or landslide. On completion, the EGM should contain information on the following aspects:

- The geometry of the slope.
- The geological materials comprising the slope and their distribution within it.
- Estimates of the shear strength and unit weight of the materials within the slope.
- Estimates of the distribution of the groundwater pressures acting within the slope and below the failure surface, and how they vary with time (e.g. seasonally, or extreme events).
- Identification of evidence of previous instability, including clay seams, carbonaceous material or debris in the subsurface record, or toe bulging, ground cracking, and debris fans in the geomorphology. If a landslide has occurred, the geotechnical model should include an estimate of the pre-landslide ground surface to allow back-analysis of slope failure. Where failure has occurred in a cut slope, the model should include the original ground profile.

This part of Unit 1 focusses on modelling of soil slopes, but much of the advice on analysis is also relevant to heavily jointed rock slopes. Rock strength and stability will be discussed in Unit 4. Additional information on rock slope stability can be found in many texts including Wylie (2017) and Turner and Schuster (1996).

2 SOIL MECHANICS PRINCIPLES

Soil mechanics principles underpin our understanding of soil shear strength and behaviour. These concepts are only briefly introduced here to provide a basis for further discussion on soil stability analysis. Further discussion on these concepts is provided in several texts including Walker & Fell (1987), Terzaghi et al (1996), Duncan et al (2014), and Das et al (2018).

2.1 DEFINITIONS

Undrained conditions occur when load changes happen faster than water can flow into or out of the soil, meaning that the water pressure in the space between soil particles (pore pressure) can change in response to a change in loading.

Drained conditions occur when load changes are slow enough to allow water to flow into or out of the soil without a corresponding change in pore pressure, or when the load remains for long enough to allow soil to drain any excess pore pressure.

Total stress refers to the total force exerted (that transmitted through particle contact and that transmitted through pore pressure) divided by total area. Total stress does not change from the drained to the undrained condition because it does not depend on whether the force on the soil is carried by interparticle contacts or pore pressure.

Effective stress represents the force transmitted through interparticle contacts only, divided by area. Effective stress is equal to the total stress minus the pore pressure.

$$\sigma' = \sigma - u$$

Where:

σ' = effective stress
σ = total stress
u = pore pressure

Drained shear strength is the strength of a soil loaded to failure under drained conditions, that is with no change in pore pressures due to the applied load and therefore no changes in effective stress.

The drained shear strength of the soil should be evaluated using the effective stress strength envelope.

Undrained shear strength is the strength of soil when loaded to failure, where load is applied faster than the soil can drain. The tendency to change volume under loading results in a change in the pore pressure and therefore effective stress. This change can be positive or negative depending on the soil.

The undrained shear strength can be defined using effective or total stress strength envelopes, but it is common to express it in terms of the total stress strength envelope.

2.2 TOTAL AND EFFECTIVE STRESS ANALYSIS

In effective stress analysis, effective normal stress on the failure plane is used to calculate the soil shear strength. This requires a determination of the pore pressures along a failure surface. For the drained condition, pore pressures are relatively easy to estimate from the hydrostatic or steady-state seepage conditions, and hence effective stress analysis can be and should be used for drained conditions.

For the undrained condition, excess pore pressures are induced but cannot be estimated accurately. Instead, under undrained loading, it is possible to relate shear strength to the total stress, which does not require the estimation of pore pressures.

Using total stress procedures for analysis of undrained conditions is more straightforward and reliable than trying to predict undrained excess pore pressures for use in effective stress analysis of undrained conditions. (Turner and Schuster, 1996).

Where slopes consist of soils with a range of permeabilities, for a given load condition, some soil layers may be drained, and others undrained. It is therefore logical to treat the drained soils in terms of effective stress and the undrained soils in terms of total stresses in the same analysis (Turner and Schuster, 1996).

Typical conditions for short- and long-term analyses are illustrated in Figure 7.1. A discussion of common loading conditions for analysis is included in Section 3.

2.3 DETERMINING DRAINAGE CONDITIONS

Determining the drainage conditions of soils in response to loading is a critical aspect of determining the shear strength of the soil.

Two variables need to be considered in determining whether undrained or drained conditions govern for a particular soil:

a) The rate of loading.
b) The speed at which the soil can drain the excess porewater pressures generated by the loading.

The time required for drainage is governed by the soil permeability and the length of drainage paths. Where loading occurs over several weeks or months, a soil with a permeability of greater than 10-6 m/s can typically be assumed to be drained and soils with permeability less than 10-9m/s will typically be undrained (Fig 7.1). Silts with permeabilities 10-6 to 10-9 m/s are likely to be partially drained. When it is uncertain if a soil will be drained or undrained, or if it is likely that soil will be undrained initially then drained, both conditions should be analysed to cover the range of possibilities (Turner and Schuster, 1996).

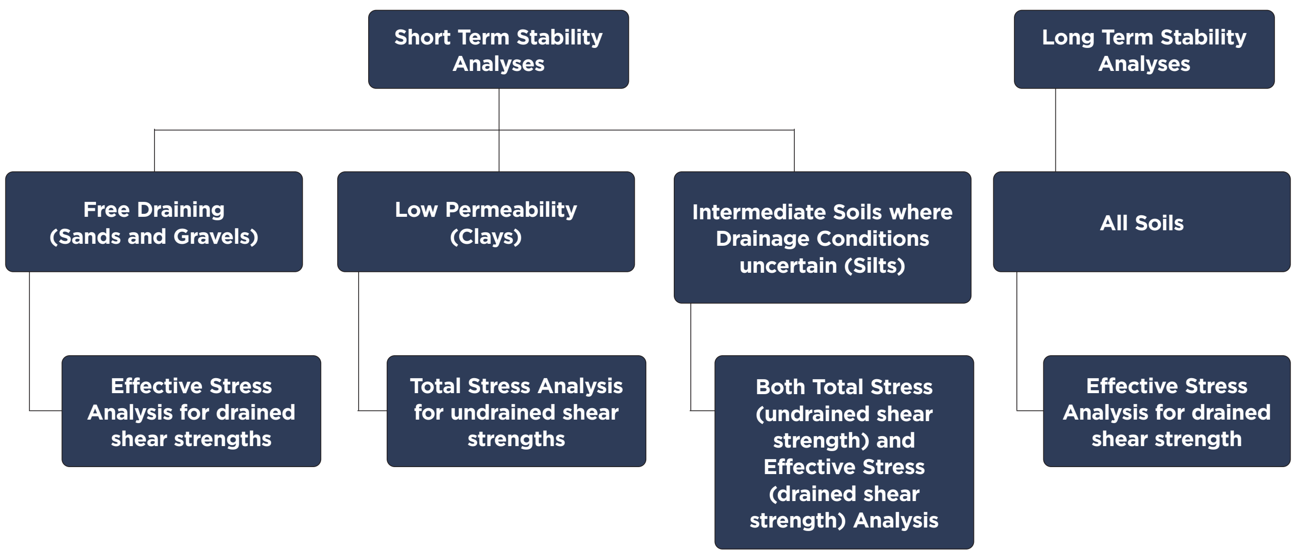


FIGURE 7.1: Soil Strengths for Typical Loading Conditions

PROGRESSIVE FAILURE

Progressive failure occurs when peak strengths cannot be mobilised at all points in the failure surface at the same time due to varying amounts of deformation within the slope. Shear strengths along the failure surface peak then reduce as displacement increases. The slope can reach a point where displacements rise rapidly, and the slope fails. Limit equilibrium analyses assume that the soil's shear strength is mobilised at all points along the failure surface simultaneously. This assumption is reasonable for soils with shear strength consistent over a wide range of deformations (i.e., ductile behaviour). However, in sensitive or brittle soils that experience a significant reduction in strength with increasing strain, the assumption of peak strength along the entire failure surface may be unconservative.

Soils most prone to progressive failure are overconsolidated clays, particularly stiff fissured clays. In these soils, softened strengths should be assumed in limit equilibrium analysis. Where shear zones have developed, residual strengths should be used.

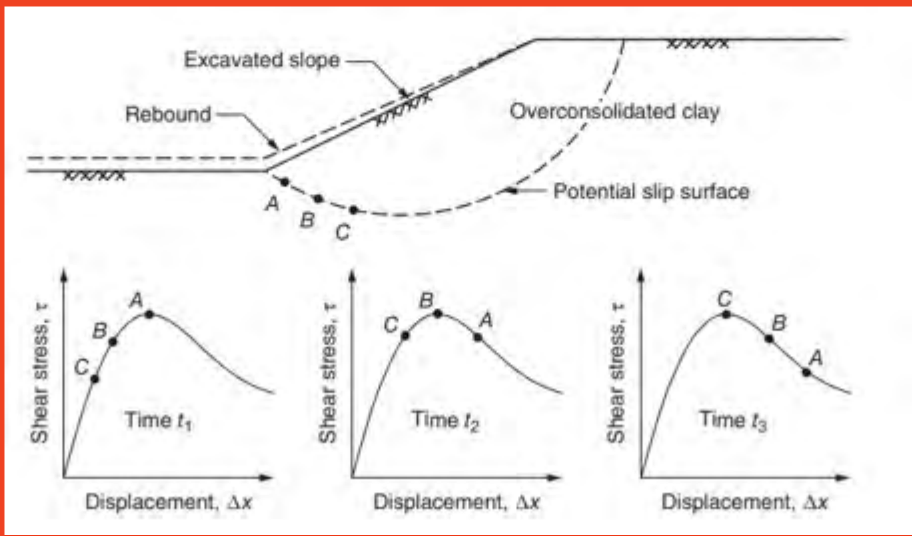


FIGURE 7.2– Mechanisms of progressive failure on an excavated slope in overconsolidated clay. From Duncan et al 2014, Figure 3.9.

2.4 PARTIAL SATURATION

It is common practice in New Zealand to assume fully saturated conditions for low permeability soils. However, some soils (like loess, e.g. on the Port Hills near Christchurch) are partially saturated in their natural state and have strength properties that are highly dependent on water content. Careful consideration of the shear strength characteristics of partially saturated soils and the likely changes in moisture conditions are required for slopes where these soils are present.

Discussion of partially saturated soil mechanics is largely beyond the scope of this guidance and further discussion can be found in Duncan et al, 2014.

3 CONDITIONS FOR ANALYSIS

Slopes are subject to changes in loads and changes in shear strengths over their lifetime. To capture these changing situations, it is necessary to analyse multiple load cases. Typical conditions for analysis are outlined in Table 7.1. This is not an exhaustive list. Additional or alternative loading cases may require analysis depending on the slope (e.g., dams).

Target Factors of Safety (FoS) commonly used in practice for each scenario are shown in Table 7.1, along with references for these values. These FoS values are shown only for context; the inclusion of the target FoS values in Table 7.1 **does not imply suitability for a specific use**. In some cases, the appropriate target FoS will be prescribed by the Crown entity, stakeholder, local authority (e.g. NZSOLD, 2015 for dams, NZTA / Waka Kotahi Bridge Manual, 2022 for roading cut and fill slopes), or a lesser FoS accepted in agreement with a client for practical reasons.

As discussed in Part 10, the appropriate target FoS should take into consideration the uncertainty in the ground model, and the consequences of failure. Further discussion on developing appropriate target FoS will be included in Unit 4.

TABLE 7.1: Typical Loading Conditions for Analysis

LOADING CONDITION	SOIL SHEAR STRENGTH	PORE PRESSURES	TYPICAL MINIMUM FOS	REFERENCES FOR MINIMUM FOS[1]
Long-term static	Drained shear strengths related to effective stresses for both free-draining and low permeability soils.	No Flow: hydrostatic pressures With Flow: steady state seepage analysis	1.5[2]	• Waka Kotahi/ NZTA Bridge Manual 2022. • NZ Dam Safety Guidelines, • U.S. Army Corps of Engineers' Slope Stability Guidelines. • MBIE Module 6: Earthquake Resistant Retaining Wall Design.
Short-term Loading (non-seismic)	Free-Draining Soils – drained shear strengths related to effective stresses.	No Flow: hydrostatic pressures. With Flow: steady state seepage analysis.	1.2 to 1.3	• Waka Kotahi/ NZTA Bridge Manual 2022. • NZ Dam Safety Guidelines. • U.S. Army Corps of Engineers' Slope Stability Guidelines.
	Low-permeability soils – undrained strengths related to total stresses	Total stresses with no pore pressure in computations		
Partial Consolidation and Staged Construction	Free-Draining Soils – drained shear strengths related to effective stresses.	No Flow: hydrostatic pressures With Flow: steady state seepage analysis.		
	Low permeability soils - Consolidation analyses used to estimate increases in effective stress due to fill. Effective stresses used to estimate undrained shear strength in total stress analysis or directly in effective stress analysis.	Total stress approach - no pore pressures in computation of strength. Effective stress approach – use consolidation analyses to estimate the excess pore pressure related the anticipated amount of dissipation at a particular time in construction.		
Earthquake Loading (effective stress or total stress soil strengths)	Free-Draining Soils – drained shear strengths related to effective stresses.	No Flow: hydrostatic pressures With Flow: steady state seepage analysis. Where pore pressure buildup is expected, excess pore pressures related to FoS against liquefaction should be included (Appendix Section A3.2)	1.0 to 1.3 or for FoS<1 deformation/ performance-based limits.3	• Waka Kotahi/ NZTA Bridge Manual 2022. • NZ Dam Safety Guidelines. • MBIE Module 6: Earthquake Resistant Retaining Wall Design-
	Low-permeability soils – undrained dynamic shear strengths related to total stresses. Use strain compatible dynamic strength.	Total stress with no pore pressure in computations		

TABLE 7.1: Typical Loading Conditions for Analysis (continued)

LOADING CONDITION	SOIL SHEAR STRENGTH	PORE PRESSURES	TYPICAL MINIMUM FOS	REFERENCES FOR MINIMUM FOS[1]
Post-Earthquake (residual liquefied soil strengths)	Non-liquefied Free-Draining Soils – drained shear strengths related to effective stresses.	No Flow: hydrostatic pressures With Flow: steady state seepage analysis. Where pore pressure buildup is expected, excess pore pressures related to FoS against liquefaction should be included (Appendix Section A3.2)	1.1 – 1.3	• Waka Kotahi/ NZTA Bridge Manual 2022. • NZ Dam Safety Guidelines (2015).
	Low-permeability soils – undrained dynamic shear strengths related to total stresses. Use strain compatible dynamic strength.	Total stress with no pore pressure in computations		
	Liquefied Soils – residual liquefied undrained shear strengths related to total stresses.	Total stresses with no pore pressure in computations		
High Ground Water Table (GWT), Flooding, Perched GWT due to high intensity rainfall[4]	Where instability may be caused by increase of pore pressure within the slope, drained shear strengths related to effective stress apply.	No flow: hydrostatic pressures related to high GWT condition. With Flow: steady state seepage analysis associated with high GWT.	1.2 – 1.3	• Waka Kotahi/ NZTA Bridge Manual 2022. • MBIE Module 6: Earthquake Resistant Retaining Wall Design • NZ Dam Safety Guidelines. • U.S. Army Corps of Engineers' Slope Stability Guidelines.

[1]Module 6 of the MBIE Earthquake Engineering Series specifies that "moderately conservative" soil parameters (i.e., lower quartile) are used. However, in a survey of New Zealand geo-practitioners, Councils and Territorial Authorities in 1994, most respondents indicated a preference to use "average" strength values and average groundwater table depth when targeting an FoS of 1.5 (NZ Geomechanics Society, 1994), but this should be based on a significant database of testing.

[2]It is common that slopes relying on drainage measures to achieve a FoS >= 1.5 for long term stability should also have a FoS >= 1.2 under long term loading conditions should drainage measures fail or not be maintained.

[3]Acceptable levels of deformation of a slope in a seismic event are dependent on several factors such as the nature of the particular amenity or structure being affected, its ability to tolerate deformation, and the required performance of the structure following a seismic event. The adoption of deformation limits should be done in close collaboration with the project team (particularly the structural engineer) and stakeholders. NZTA/Waka Kotahi Bridge Manual (2022) provides guidance on acceptable deformations related to roading infrastructure.

[4]Particular discussion on rapid drawdown is generally outside the scope of this document. Discussion on the rapid drawdown load case and its analysis can be found in Duncan et al (2014).

4 ESTIMATING SHEAR STRENGTH OF SOILS

A brief discussion on estimating soil shear strength is provided. Further discussion and strength parameter correlations for both granular and cohesive soils can be found in the following texts:

- Bowles (1996). Foundation Analysis and Design.
- Duncan et al (2014). Soil Strength and Slope Stability
- Look (2017). Handbook of Geotechnical Investigation and Design Tables 2nd edition.

The geoprofessional should use multiple methods of measuring and estimating shear strength parameters and multiple correlations to capture the variability in the test and correlation methods. The appropriate level of investigation and testing to develop design strength parameters depends on the complexity of the ground conditions and the complexity / consequence of failure of the development. Levels of investigation are discussed in Part 5, Section 4 of this Unit.

Additional detail on estimating soil shear strength can be found in Section A1 of the Appendix.

Granular Soils – Sand and Gravel: Granular soils are usually fully drained and cohesionless except during liquefaction. The value of effective friction angle in a granular soil is affected by the relative density of the soil, particle shape, and particle size distribution. Friction angle increases with relative density, particle angularity, and is higher for well graded soils than for uniform soils, all other things being equal.

Silt: The behaviour of silts varies widely and is not as well understood as that of granular soil or clay. Silts can be broadly categorized into non-plastic silts, which exhibit behaviour similar to fine sands, and plastic silts, which behave more like clays.

Clay: The presence of clay contributes to many slope instabilities, and the strength of clay changes with time. Part 10 Table 10.1 presents a list of the sources of instability, and many of these are related to the complex interaction of clay and water. Further discussion on evaluating the (drained and undrained) strength of clays is contained in Section A1 of the Appendix.

"BASED ON EXPERIENCE" SOIL STRENGTHS

In the authors' experience it is relatively common for soil strengths to be selected for analysis using a "based on experience with similar soils in this geology" approach without laboratory or insitu testing. This approach is generally not adequate for slope stability analysis, except for low-consequence-of-failure developments in simple/uniform ground conditions (i.e. low risk projects), or where the soil unit is particularly well studied and understood. If this approach is adopted, the geoprofessional should provide justification for the selected soil parameters answering the questions:

- **Where are the input parameters coming from?**
- **Why are they applicable here? and**
- **What are the uncertainties associated with these parameters? Are these considered 'best estimate' values or conservative estimates?**

The use of soil strength parameters "based on experience" without further justification is not appropriate in most circumstances.

5 PORE WATER PRESSURES

The field investigations should provide sufficient information to estimate the groundwater pressures acting within the soil. The groundwater profiles ultimately adopted for analysis should be generated from and consistent with the Engineering Geological Model (EGM). There are multiple ways of representing pore water pressures as listed below and outlined in more detail in Section A2 of the Appendix.

- Approximation based on the phreatic or piezometric surface (water table).
- Numerical solutions – most groundwater flow and seepage analyses today are performed using finite element modelling and these solutions are integrated into commercial slope stability software packages.
- Performing simple hand drawn flow nets is useful for validating complex computer models.
- For slopes with complex pore water pressure distributions within the slope, slope stability software packages (SLIDE, SLOPE/W used in partnership with SEEP/W) allow specification of pore water pressures in a variety of ways beyond a simple piezometric line.

Estimating pore pressures based on the piezometric surface provides a good approximation when groundwater flow is predominantly horizontal. Where flow is not predominately horizontal this method can result in non-conservative errors. In this case it is better to use finite element seepage analysis (Duncan et al 2014).

Seasonal fluctuations and extreme groundwater levels should be checked in the analysis to understand the sensitivity of stability to reasonable changes in groundwater levels. High sensitivity may identify the need to consider drainage measures as a means of preventing future uncontrolled increases in water pressures with consequential stability reductions. The Engineering Geological Model, together with field test data, should form the basis for determining the pore water pressure distributions for high groundwater and flooding load cases.

6 METHODS OF ANALYSIS

Once the slope geometry, soil strength, and pore pressures are known, stability analyses can be carried out. Analyses generally utilise either limit equilibrium methods (LEM) or numerical methods. Table 7.2 provides a brief summary of commonly used LEM and numerical methods. This document will focus on limit equilibrium (procedure of slices) methods due to their prevalence of use in New Zealand and their applicability over a wide range of conditions.

TABLE 7.2: Summary of Analysis Methods

ANALYSIS APPROACHES	METHOD DESCRIPTION	ADVANTAGES	DISADVANTAGES
Limit Equilibrium Methods (LEM) Limit equilibrium methods use the equations of static equilibrium to compare forces driving and resisting failure to determine a factor of safety (FoS). The static equilibrium conditions are equilibrium of (1) forces in the vertical direction, (2) forces in the horizontal direction, and (3) equilibrium of moments about a point. Equations are solved for a single failure surface so a critical failure surface needs to either be assumed or determined through trial-and-error calculations of multiple surfaces.	**Infinite Slope Analyses** This method assumes the slope is infinite in extent, the failure surface occurs along a single plane parallel to the slope surface, and that the soil properties are homogeneous and isotropic. These assumptions allow for the use of simple equations to calculate the factor of safety. Infinite slope equations for a few typical conditions can be found in Duncan et al (2014) and Chapter 13 of Turner and Schuster (1996).	• Quick to implement therefore useful for preliminary checks or to verify more complex methods. • The assumptions of this method are often reasonable for natural slopes within New Zealand, particularly where there is weaker surficial soil layer overlying stiffer, stronger material.	• The method is only applicable where the simplifying assumptions provide reasonable approximation of the site conditions. It is often not appropriate where soil stratigraphy, pore pressures, and/or slope topography is complex.
	Stability Charts Charts have been developed by several authors covering a variety of soil and slope conditions (Taylor 1937, Morgenstern 1963, Janbu 1968, Leshchinsky 1985, Duncan et al, 1987) to simplify the use of some LEM procedures. These charts provide a means of rapidly assessing the stability of simple slope geometries with defined groundwater pressure profiles and material distributions. A selection of these charts and discussion regarding their use is provided in Appendix A of Duncan et al (2014) and Chapter 13 of Turner and Schuster (1996).	• No estimate or iteration to determine the critical failure surface is required. • Quick to implement therefore useful for preliminary checks, comparing design alternatives, or to verify more complex methods.	• The method is only applicable where the simplifying assumptions provide reasonable approximation of the site conditions. It is often not appropriate where soil stratigraphy, pore pressures, and/or slope topography is complex.
	Procedures of Slices In these methods the potential failure mass is divided into several (typically) vertical slices and equilibrium equations are solved for each slice[1]. Today these methods are most often implemented in computer software packages. Procedures of slices are widely used within New Zealand and the focus of this Section.	• They have been incorporated into powerful computer software capable of fully analysing a wide variety of slope failure shapes, searching for critical failure surfaces, back analysis of shear strength and with provision to incorporate seepage analyses. • They are quick to perform and are applicable over a wide range of slope geometries, soil strengths and stratigraphy, pore pressure conditions, external loading, and internal reinforcement. • Sensitivity studies are easy to perform.	• Not as quick and easy to implement as stability charts or infinite slope methods. • Compared to the more sophisticated numerical analyses they don't produce an estimate of slope displacements • Their ability to model the impact of earthquake loading on slope behaviour is limited.

[1]Procedures that assume a circular slip surface consider equilibrium of moments about the centre of the entire potential slide mass comprised of all slices.

TABLE 7.2: Summary of Analysis Methods (continued)

ANALYSIS APPROACHES	METHOD DESCRIPTION	ADVANTAGES	DISADVANTAGES
Numerical Methods They use mathematical models to simulate the behaviour of soil or rock and to predict the failure mechanism and the factor of safety. They predict the likely displacements of the slope and can be used to examine areas of slope displacement of key importance. The most common numerical methods are the Finite Element (FEM) and Finite Difference (FDM) methods. It is sound practice to check outputs using LEM. For straightforward analyses (such as static loading) the factor of safety outputs reached by both methods should be similar.	**Finite Element Method (FEM)** the slope is discretised into small elements, and each element is analysed separately. The behaviour of each element, in terms of stress and deformation, is described by a set of mathematical equations, which are then solved using iterative techniques to determine the overall slope behaviour. Compared to FDM, FEM can handle more complex geometries and boundary conditions but requires more computational resources and is more difficult to implement. **Finite Difference Method (FDM)** In FDM the slope is divided into a grid of small cells with nodes. The behaviour of each cell is described by a series of partial differential equations. The equations are solved using iterative techniques to determine the factor of safety and potential failure modes. By solving the equations iteratively FDM calculates the distribution of stresses and displacements within the slope. FDM is more suited to analysing slopes with regular geometries such as embankments and cut slopes.	• Compared to LEM, numerical methods can model highly complex ground and groundwater conditions (e.g. variable, non-linear soil properties). • Can consider groundwater pressure changes and the response of the slope to earthquake shaking. • Allow for the prediction of failure mechanisms and deformations.	• Requires more computational power. • Requires a greater understanding of the soil behaviour. • The user must be highly skilled in slope stability modelling and the use of the software to obtain reasonable assessments of slope performance. • The accuracy of the output depends heavily on the quality of the input data. Errors made in modelling the slope inputs will lead to poor results.

PROCEDURES OF SLICES – WHAT METHODS TO USE

Numerous procedures of slices have been developed. Procedures that satisfy all three conditions of equilibrium, and allow for both circular and non-circular failure surfaces, include:

1. **Spencer's Method – assumes interslice forces are parallel.**
2. **Morgenstern and Price's Method - assumes a pattern of side forces that can be defined by the user.**
3. **Chen and Morgenstern's Procedure– a refinement of the Morgenstern Price method to better account for stresses at the ends of a slip surface.**
4. **Sarma's Procedure – considers the seismic coefficient to be unknown and the FoS to be known. Useful when calculating seismic yield coefficient.**
5. **Janbu's Generalized Procedure of Slices – there is some debate as to whether this procedure satisfies all conditions of equilibrium, but the procedure generally results in a factor of safety that is nearly identical to those that satisfy all conditions of equilibrium.**

No procedure that satisfies all conditions of equilibrium is more accurate than another (Duncan et al 2014). Other limit equilibrium methods which do not satisfy all the conditions of equilibrium such as the Ordinary Method of Slices, Bishop's Method, and Janbu's Simplified method are discouraged from general use as they have been shown to give inaccurate estimates of stability when compared to the more rigorous methods (Turner and Schuster, 1996).

A thorough discussion of these methods is included in Duncan et al (2014) and Turner and Schuster (1996).

7 CONSIDERATIONS IN STABILITY ANALYSES

7.1 LOCATING THE CRITICAL FAILURE SURFACE FOR ANALYSIS

The critical failure surface can either be determined from analysis as the surface with a minimum factor of safety, or from the field observations and investigations (e.g., recording of tension cracks, geological profile, deformation of inclinometers) that determine where slope movement is occurring.

Slope stability analysis software packages employ schemes to locate surfaces that produce the minimum factor of safety. For new slopes that have not previously failed (or don't yet exist) the critical surface identified by the software will form the basis for determining the factor of safety of the slope and the need for stability improvement measures. Some considerations for locating the critical failure surface include:

- Non-circular failure surfaces will often have lower FoS than circular surfaces.
- A good place to start searching for critical non-circular surfaces is at the location of the critical circular surface or by searching for surfaces that follow weak layers.
- There may be multiple local minimum failure surfaces. Selecting several starting points and searching for surfaces over a range of depths can help identify these.
- The surface with the absolute minimum FoS may not be the surface of greatest interest. Deeper surfaces with higher FoS but larger consequences of failure may be critical. Insignificant surfaces include those that are too shallow to be consequential, or those that are deemed to be unlikely to affect the structure / infrastructure of interest. To prevent these surfaces from being shown in outputs, computer programs allow the user to define minimum depths, minimum weight, or a range of points that the surface must pass through. Significant surfaces are those that are consequential and whose failure should be considered. There may be multiple significant surfaces that need to be considered.
- The critical surface is the kinematically admissible, significant surface with the lowest FoS.
- The critical failure surface can form the basis for determining, using back analysis, the average shear strength value(s) needed to bring the slope to a point of limit equilibrium (i.e., a factor of safety = 1.0). Back analysis is discussed in Section 9 following.

7.2 THREE DIMENSIONAL EFFECTS AND STRUCTURAL FEATURES

Most stability analysis of slopes in standard engineering practice is two dimensional. The 2D assumption in slope stability analysis implies that the slope is treated as a two-dimensional plane surface, which means that the analysis is limited to a single cross-section of the slope. This assumption is based on the observation that most slopes can be approximated as planar surfaces, and that the difference in behaviour of the slope in the direction perpendicular to the cross-section being analysed is negligible.

This assumption is violated by any structural weakness such as a fault running up one boundary of the failure area or a dominant defect in a rock slope that governs the failure surface mechanism. In these cases, other methods of stability analysis such as wedge analysis of rock slopes or 3D modelling may need to be used.

Structural features impeding the use of 2D analysis sections tend to occur in rock slopes. In soil slopes the 2D assumption is often reasonable and provided the critical cross-section is chosen for 2D analysis, 2D FoS are typically lower than 3D FoS (i.e., conservative). However, there may be some situations where soil geometry is such that there is significant benefit (cost savings) in accounting for 3D effects these include (1) where soil strengths are back-calculated from 3D failure and these biases are not compensated for and (2) where the slope is curved, or short and not well represented by a plane strain model.

7.3 TENSION AT CREST OF SLOPE

Analyses can calculate tension between some of the slices where there are cohesive soils at the top of the slope. Most soils do not have significant tensile capacity or may be subject to shrinkage during drier seasons. These tensile forces can cause problems in the FoS calculation, so they may need to be eliminated. This can be done by introducing a tension crack at the top of the slope. More details can be found in Duncan et al (2014).

7.4 VERIFICATION OF RESULTS

Slope stability analysis results should be independently checked to ensure defensible results. In all cases, slope stability results should be viewed within the context of the area's geology and observed slope performance. Some methods of verifying results from computer analyses include:

a) Use experience of the past performance of the slope and the past performance of other, similar slopes in similar geology. Check that the selected soil parameters predict performance commensurate with observed performance. This is the most important and useful means of verifying stability results. The geoprofessional should always be asking "Do these results reflect reality?" Where results are not consistent with observed performance, the geoprofessional should re-evaluate the ground model which may include additional site mapping and/or investigation.
b) Use more than one full equilibrium algorithm to check that the predicted stability results are not unduly influenced by the chosen methodology (e.g. use Spencer's method as well as Morgenstern Price).
c) Compare results against those calculated using another software package, or simplified calculations (slope stability charts, infinite slope).
d) Perform sensitivity studies to ensure that changes in input parameters cause reasonable changes in results. Typically, the key parameters will be the piezometric pressures, the critical failure surface location and shape, and the shear strength parameters.

8 SEISMIC SLOPE STABILITY

New Zealand has a high earthquake hazard and therefore earthquake considerations are integral to the design of the built environment and extremely important to geoprofessionals assessing slope stability hazards. Hancox et al (1997) report that in New Zealand, next to earthquake magnitude and intensity, landslide occurrence is most strongly controlled by topography, rock and soil types, with failures mostly on moderate to very steep slopes (20°-50°). The most common landslides during earthquakes are rock and soil falls on very steep cliffs, escarpments, gorges, gravel banks, and high unsupported man-made cuts. A large earthquake may be followed by a long period with heightened seismicity that may affect destabilised slopes.

This part of the guidance presents information on straight-forward procedures that can be employed to evaluate the performance of slopes in and following earthquakes for routine projects. More sophisticated analysis procedures involving dynamic effective stress numerical modelling incorporating non-linear stress strain properties of soils, are typically employed for large slopes or embankments where the consequences of failure are high. These comprehensive analyses are briefly discussed in the Appendix but are generally outside the scope of this document.

Fundamental issues that should be addressed when assessing seismic stability of soil slopes are:

1. Are there soils within or beneath the slope that could liquefy? This is an important consideration for man-made slopes in some geological environments as the presence of liquefiable soils can control the seismic performance of the slope and lead to large deformations or flows. If there are liquefiable soils, the residual liquefied shear strength should be estimated, and a post-seismic factor of safety calculated (i.e. flow failure check). If the FoS is near to or less than one, flow failure could occur, and mitigation or further detailed investigation and analysis is required. Evaluating soil liquefaction is discussed in Module 3 of the MBIE Earthquake Engineering Series.
2. If there are no soils that will undergo significant strength loss due to ground shaking, then seismic deformations of the slope should be estimated or a pseudo-static screening analysis can be undertaken to determine if adequate stability exists).

The general process for simplified seismic slope stability assessment is summarised in Figure 7.3. Details of the various elements of this process are discussed in Section A3 of the Appendix. Further detail on seismic slope stability assessment including worked examples will be included in Unit 4.

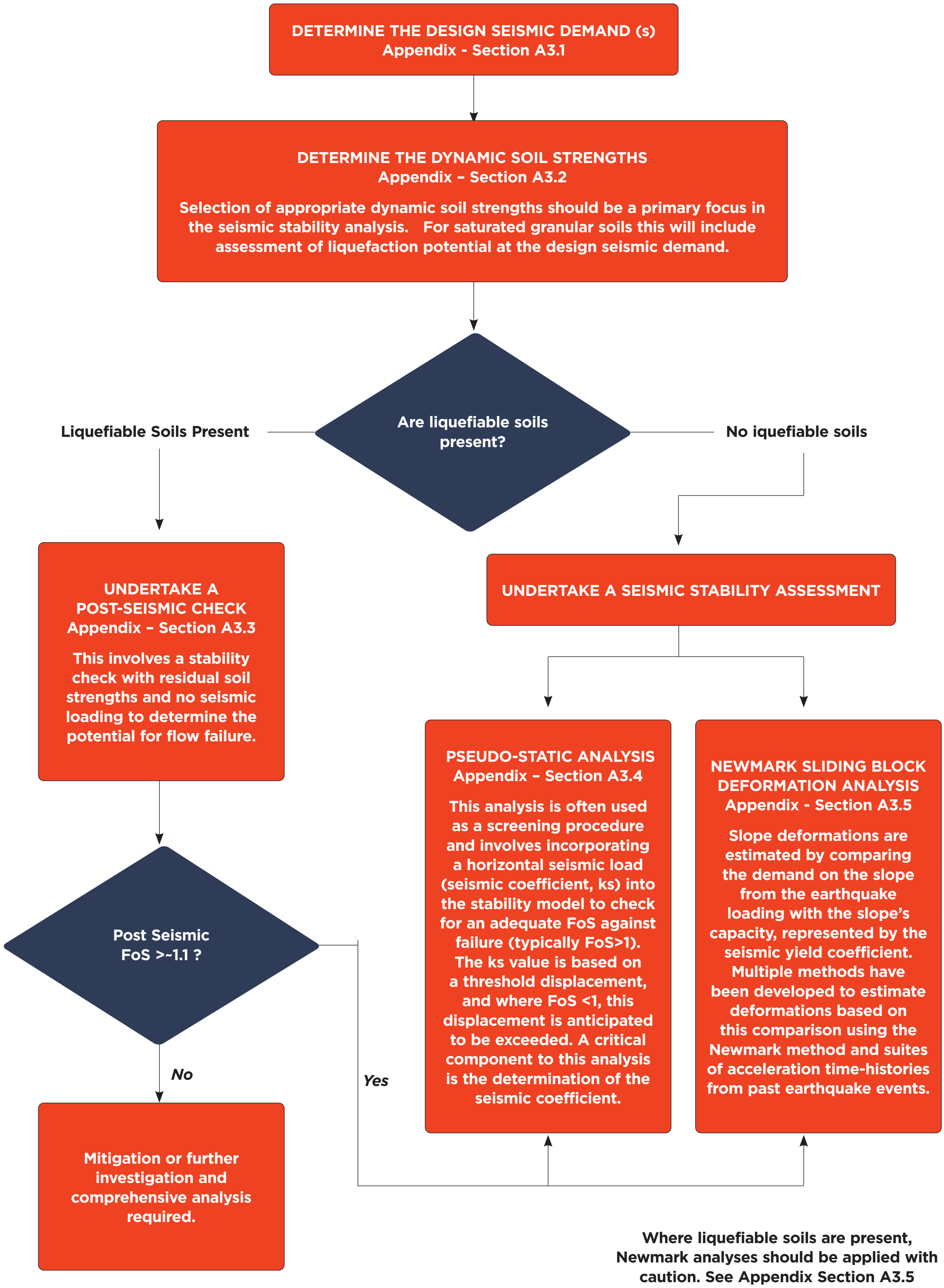


FIGURE 7.3: Seismic Slope Stability Assessment Process

9 BACK-ANALYSIS

Back-analysis of a slope failure involves retroactively analysing the failure by conducting slope stability analysis and adjusting input parameters (typically strength parameters) until a FoS = 1 is achieved. This is essentially a full-scale test model and can provide useful insights into the mechanisms of failure, strength of soils, and pore water pressure distribution at the time of failure.

Back-analysis can also be used to estimate soil strength parameters for slopes that have not failed. In this case the soil strengths must be at least high enough to achieve a FoS of 1. If the slope has remained stable for many years without failure, then the FoS should be at least 1 over the range of conditions it has been subject to in that time (i.e. high groundwater, earthquakes, surcharge loading etc.).

Soil strengths determined through back analysis of a failure provide a sound basis for analysing mitigation options. Mitigation options are discussed in Part 10 of this Guidance. Analysis of mitigation options will be addressed in more detail in Unit 3.

Considerations for back-analysis and mitigation assessment are included in Section A4 of the Appendix.

10 UNCERTAINTY AND PROBABILISTIC ANALYSES

Slope stability analysis often involves a high level of uncertainty related to the inherent variability in soil stratigraphy and strength, and uncertainty in environmental factors such as earthquake loading and rainfall events.

Historically, these uncertainties were not explicitly addressed, but managed in deterministic analysis through minimum FoS requirements and conservative soil strength assumptions. However, advancements in technology now make probabilistic slope stability analysis more accessible. Modern commercial software includes integrated tools for sensitivity and probabilistic analyses.

In New Zealand, the explicit consideration of uncertainty through routine sensitivity studies and/or probabilistic analyses is encouraged as common geotechnical practice. This approach, evaluating performance across a range of possible subsurface and environmental conditions for key parameters, provides a more robust foundation for decision-making.

Approaches for treatment of uncertainty include:

- Sensitivity / Parametric Studies - The following approach is outlined in Section 10.3 of Module 3 of the Earthquake Engineering Series and steps include:
  - Identify critical uncertainties in the analysis (e.g., soil strength, groundwater depth, earthquake acceleration).
  - Determine a reasonable range of values for each critical uncertainty.
  - Undertake sensitivity analyses using lower-bound, upper bound and best estimate values to assess the range of slope performance.
- Probabilistic Analyses – these methods estimate the probability of slope failure by directly accounting for uncertainty and variability in input parameters. Steps for carrying out these analyses include:
  - Identify critical uncertainties in the analysis (e.g., soil strength, groundwater depth, earthquake acceleration).
  - Represent the uncertain parameters using probability distributions, such as normal, log-normal, or uniform distributions, based on available data or expert judgment. Additional information on probability distributions of soil parameters can be found in Duncan et al (2014) and Look (2017).
  - Estimate the probability of failure. Duncan et al (2014) provides a procedure to evaluate the probability of failure. Probability of failure can also be determined using Monte Carlo simulations.
  - Once the probability of failure is understood, and the consequence of the failure is understood, slope stability can be assessed within a risk framework.
- Bray and Macedo (2023) outline a performance-based approach for seismic slope stability.

Further discussion on this topic will be provided in Unit 4.

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PART 8

ROCKFALL

Rockfall at Redcliffs, following the Christchurch Earthquake (February 2011)

PART 8 – ROCKFALL

1 INTRODUCTION

For the purposes of this guidance, the following definition of 'Rockfall' has been adopted (Hungr et al, 2014):

Detachment, fall, rolling, and bouncing of rock fragments. May occur singly or in clusters, but there is little dynamic interaction between the most mobile moving fragments, which interact mainly with the substrate (path). Fragment deformation is unimportant, although fragments can break during impacts. Usually of limited volume.

The term 'Rockfall' therefore relates to the isolated fall of a single rock block or blocks. In contrast, rock avalanches involve many blocks and have a much greater degree of particle collision during travel, which means that the overall behaviour is different due to their internal mechanics.

This part of Unit 1 is intended to be complementary to the Passive Design Guidance released by MBIE in 2018, MBIE (2018). It provides a description of the site assessment that should be undertaken and aspects of rockfall modelling, to allow:

- Assessment of rockfall hazard
- Appropriate parameters (bounce height and energy) to be determined as part of the design process discussed in the MBIE (2016) guidance.

As with other parts of Unit 1, it is not the intent of this section to prescribe steps that should be followed in an assessment of rockfall hazard. Rather, the intent of this part is to outline elements that the geoprofessional might need to be consider.

2 SITE ASSESSMENT

The usefulness of the rockfall hazard assessment is dependent on the level of understanding of the site conditions.

There are three important factors that need to be considered to assess the rockfall hazard and develop a rockfall model that best represents the site conditions:

- Characteristics of the source area (Table 8.1).
- Characteristics of the slope in the runout zone (Table 8.2).
- Evidence of previous rockfall (Table 8.3).

Table 8.1. Rock Source Characteristics

Element to be Addressed	How	Why is it important?
Location and extent	• Geological maps • Site observations • Historical Aerial photographs • Drone footage	• Identification of a credible source area; what length of the hillslope might generate rocks? • How many properties/ what length of corridor could be affected?
Rock Mass at Source	• Regional geological maps • NZGD • Published papers • Site observations, mapping of local geology with particular attention to variability and defects	• Provides understanding of transportation of the rock block. Will it break up on impact with rock mass below and/or hillslope?
Block size and Shape	• Site observations: – Measurement of defect spacing, lining and openness in source area – Record block sizes along a transect down the talus slope.	• Understanding mode of transportation of the rock block/s and energy of the rock block/s. • Understanding of likely run-out of the rock block/s Can affect run-out distance and block rotational energies generated (bouncing, cartwheeling, sliding).
Groundwater and surface water	• Site observations: – Staining of defects – Seepage from bedding or other defects	Important as water pressure may play a role in triggering rockfall.
Release Frequency	• Observation of areas of exposed fresh rock faces within the source area • Extent of lichen and vegetation growth • Aerial photographs of the slopes flown at different times	May provide an indication of the likelihood and frequency of initiation

Table 8.2 Slope Characteristics

Element to be Addressed	How	Why is it important?
Geomorphic Features	• Published topographic maps; • LiDAR; • Preparation of geomorphic maps from aerial photographs • Geomorphic mapping on site • Supplemented with drone footage	Critical for understanding bounce height, runout distances and preferred travel directions/paths. For example, gullies tend to 'topographically focus' rock block travel
Slope Materials	• Geological maps • Geomorphic maps • Site records	Understanding of damping effects and boulder transport and entrainment
Vegetation	• Aerial photograph interpretation • Site observations - Type of vegetation; how well established is it; presence of impact scars, proximity to assets.	Gives indication on frequency of occurrence of rockfall events; effect on rock fall trajectory and run-out distance.

Table 8.3 Evidence of Previous Rockfall

Element to be Addressed	How	Why is it important?
Recent historic events	• Site observations: – Is there lichen or weathered surface on In- situ rock or fallen rocks? – Are there tree scars (loss of foliage due to impact or flow) – Are there fresh scars or fresh rock blocks (without weathered faces) on the slope? • Historical Aerial photographs. • News articles and records.	Gives an indication of frequency of rockfall. Dependent on the information available can also give an indication of rock sizes, volumes and run-out distances.
Block Shape	• Observations of the talus slope: • Are rocks embedded in or resting on the slope? • Measurement of defect spacing in source area clues may be given by orientation of fresh faces	Affects both the run-out distance and block rotational energies. Allows appropriate block shapes to be considered in rockfall modelling.
Block Size Distribution	At a minimum, assess the average boulder size and the size of the 95th percentile block to gauge size distribution. A good way to do this is by placing a tape down the slope and recording the dimensions of each rock block intersected by the tape. This also allows estimation of block mass and plot of a grading curve.	Important to understand block sizes and mass for hazard analysis and design of mitigation works.
Trajectory	As a first approximation to determine preferential travel paths, assume rocks will fall perpendicular to the slope contour. For more complex sites, three dimensional rockfall modelling should be considered.	For the calculation of runout distances and travel directions. Should be calibrated to observations where possible.
Volume	• high-level assessment can be made by estimating the length, width and depth of observed rock fall; • Compare pre- and post-failure surfaces if LiDAR or other topographic survey available	For the understanding of impact, travel distance and mitigation
Runout Distance	Site Observations: • Assessment of the Fahrböeschung, or F-angle (see Section 3.1 below) or the minimum shadow angle (M) of a talus slope[1]. • Record locations of fallen rocks – LiDAR or aerial photographs may assist	• To determine the extent of rock fall hazard • Provides the ability to sense check rock fall modelling and calibrate as required.

1 The **Shadow angle** is defined from the apex of the talus slope to the outer margin of the rockfall shadow. The **Fahrboeschung angle** is measured from the top of the source area to the outer margin of the rockfall shadow. The rockfall shadow is the area downslope of the talus slope, which is covered discontinuously by scattered boulders and rock fragments that have rolled or bounced beyond the base of the talus.

3 ESTIMATING ROCKFALL HAZARD

As outlined in Part 6, the annual risk of loss of life ($P_{(LOL)}$) can be calculated as

$$P_{(LOL)} = P_{(L)}\ x\ P_{(T:L)}\ x\ P_{(S:T)}\ x\ V_{(D:T)}$$

For a rock block to potentially create a risk to people (or infrastructure) within a given period of time it must be released from its source area (the Probability of initiation or $P_{(L)}$) and propagate, or travel, from the source to the element at risk $P_{(T:L)}$. For rockfall hazards:

- Estimating the probability of initiation (or rock block detachment) ($P_{(L)}$) should be based on an assessment of the source and run-out areas as described in Tables 8.1 to 8.3.
- Assessment of the probability of travel ($P_{(T:L)}$) can be initially estimated by considering runout distance (see Section 3.1 below) and calibrated by site observations.
- If the outcome of this initial assessment suggests that there is a significant probability of rocks reaching the element at risk being considered, then more sophisticated assessment is likely to be required to assess vulnerabilities (refer Sections 4 and 5 following).

Some background on the elements that should be considered for assessing hazard and subsequent vulnerabilities is given in Part 6.

3.1 RUNOUT DISTANCE

3.1.1 Influencing Factors

Various factors influence rock fall trajectory and subsequent distance travelled during a rockfall event. These include:

- **Mass and Size:** The mass and size of the rock block play a significant role in determining how far it can travel. Larger and more massive rocks tend to have greater momentum and can travel farther than smaller ones.
- **Rock Strength:** Softer rocks (for example, Tertiary Mudstone) will tend to break up to a greater degree than more indurated rocks (Port Hills Basalt, for example), thereby losing energy and reducing run-out distance.
- **Shape:** The shape of a rock block affects its rolling and bounce behaviour. Equant blocks (roughly equidimensional, Figure 8.1) tend to have relatively even roll and bounce characteristics, whereas tabular blocks tend to either not travel, to slide on other tabular rocks, or travel large distances if they start 'cart-wheeling' down the slope.
- **Initial Angle and Speed:** The initial angle and speed at which a rock block detaches from its original position also impacts its travel distance. A rock launched at a steeper angle with greater initial velocity is likely to cover a longer distance before coming to a stop. If rock block movement is triggered by earthquake shaking, peak ground acceleration will likely also contribute to distance travelled (e.g., the Canterbury earthquake sequence was characterised by high vertical and horizontal ground accelerations.

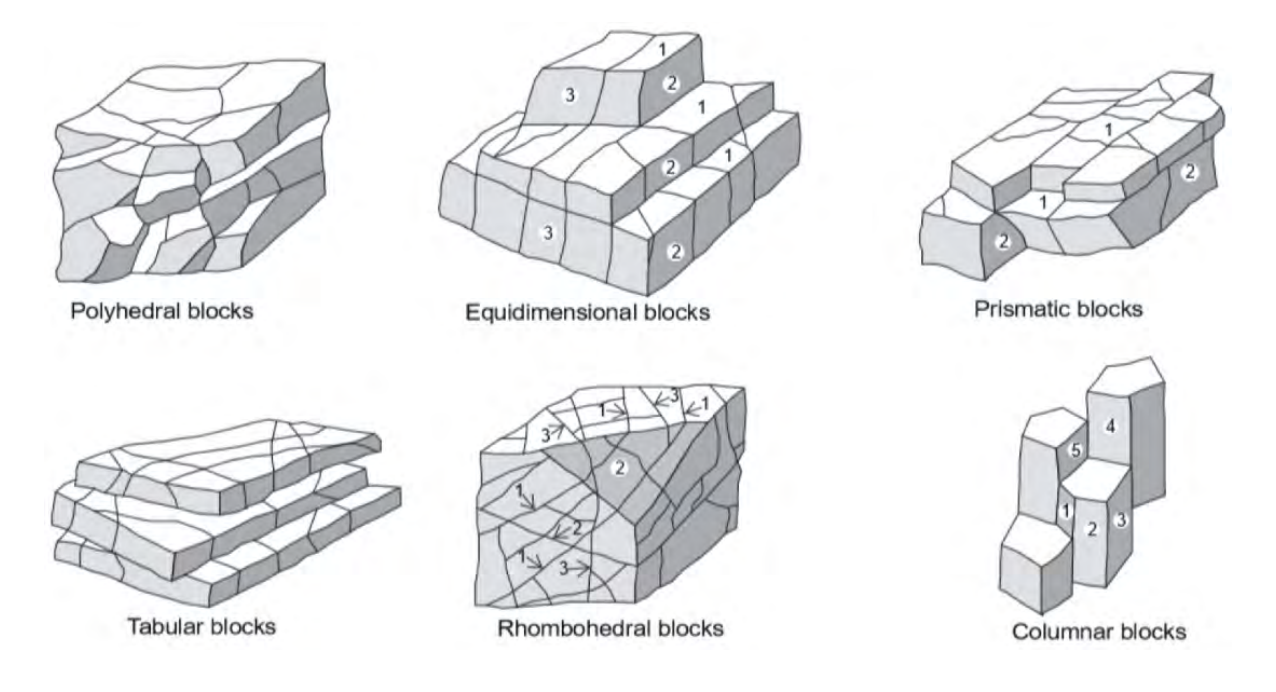


FIGURE 8.1 Block Shapes (Dearman, 2013). The same terminology is used in the NZGS field description for Soil and Rock

- **Surface conditions:** For soil slopes, there may be a significant change in material properties with seasonal moisture variation. Boulders released during winter months may travel less distance, due to softer surface soils meaning that there is a greater degree of rock ploughing into the surface. On the other hand, rocks released during dry summer months may travel further as the surface soils may be much harder.
- **Obstacles and Slope:** The presence of obstacles, such as vegetation, rocks, or irregular terrain, can obstruct or deflect the path of a falling boulder, reducing its travel distance. Additionally, the slope of the terrain can either aid (by allowing the rock block to 'ski-jump') or hinder the block's movement (on a relatively flat area for example), influencing how far it can travel. Similarly, a topographic low perpendicular to the slope might catch falling rock blocks and arrest their fall, but a topographic gully might focus rock fall and cause blocks to travel further.

The interaction between these factors can be complex and means that the precise prediction of rock fall travel distance is challenging – there are elements of "randomness" that influence rock fall.

3.1.2 F-angle

Rock block runout distance can initially be estimated using a Shadow or F-angle approach as described in Part 6. While this provides a reasonable first estimate, these relationships should be calibrated to the rock mass at the location of interest. Massey et al (2012) indicates the following relationship for rocks released during the Christchurch Earthquakes (Table 8.1). This dataset is based on relatively equant boulders, not subject to fragmentation, released during summer months onto dry soil, so is expected to be relatively conservative.

The F-angle relationship does not work well where the slope profile is initially steep and then becomes flat, as is the case for cut slopes. Here, alternative assessment (e.g., Pierson et al, (2001); see example in Figure 8.2) should be considered.

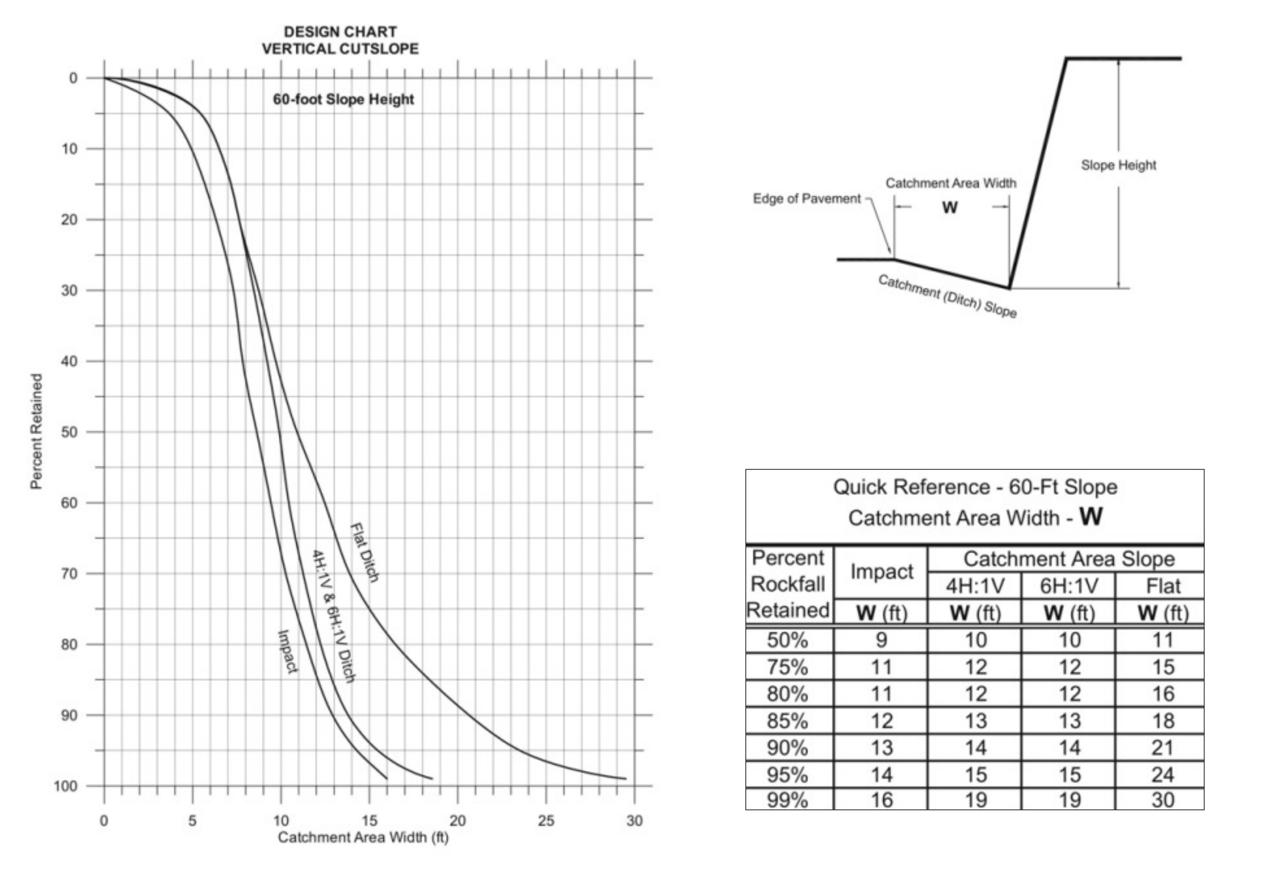
3.2 EFFECT OF ROCK BLOCK FRAGMENTATION

Rock block fragmentation can have a significant impact on rockfalls. When a large rock block breaks apart or fragments during a rockfall event, several factors come into play:

TABLE 8.4 – Estimated uncertainties of percentage of rock blocks passing a given Shadow angle for rockfalls triggered in the 22 February 2011 Christchurch Earthquake (Massey et al, 2012)

Shadow angle (°)	Mean % of boulders reaching or passing (all suburbs)	Standard deviation of mean (% passing) (all suburbs)	90% Limit (% passing) (all suburbs)
31	60.3%	6.3%	10.4%
29	42.4%	7.2%	11.9%
27	23.5%	6.1%	10.0%
25	12.8%	4.3%	7.0%
24	8.5%	3.3%	5.4%
23	3.0%	1.2%	2.0%
22	0.8%	0.6%	1.1%
21	0.1%	0.1%	0.1%

Notes:
1) the 90% Limit (% passing) column in Table 8.1 is intended to be added to the mean. By way of example, there is a 33.5% probability that a rock will reach or pass a shadow angle of 27°, at a confidence interval of 90%.
2) The data in Table 8.1 is site and event specific for Christchurch and may not apply to other areas or rock types.



Quick Reference - 60-Ft Slope
Catchment Area Width - **W**

Percent Rockfall Retained	Impact	Catchment Area Slope		
		4H:1V	6H:1V	Flat
	W (ft)	**W** (ft)	**W** (ft)	**W** (ft)
50%	9	10	10	11
75%	11	12	12	15
80%	11	12	12	16
85%	12	13	13	18
90%	13	14	14	21
95%	14	15	15	24
99%	16	19	19	30

FIGURE 8.2 **Example Design Chart for 60ft (18.3m) high vertical slope of Cumulative Percent Rockfall Retained vs Catchment Width (Pierson et al, 2001). The series of charts covers slopes from 12.2m to 24.4m high slopes cut at angles between 1V:1H and vertical.**

- **Fragmented Size:** The size of the resulting fragments affects their mobility and travel distance. Smaller fragments, such as rocks or debris, as outlined in Section 3.1.1.
- **Trajectory and Dispersion:** Fragmentation can cause the rockfall to disperse in multiple directions. As larger rocks break into smaller pieces, the trajectory and dispersal pattern of the fragments become more complex. This dispersion can result in a wider area of impact and potentially affect a larger zone, increasing the reach and potential hazards of the rockfall event.
- **Energy Redistribution:** Fragmentation redistributes the initial energy of the rockfall. As a rock block breaks apart, the energy that was initially concentrated in a single mass is redistributed among the resulting fragments. This redistribution can affect the overall dynamics of the rockfall, potentially altering the trajectory, velocity, and travel distance of the fragments.
- **Interactions with the Terrain:** Fragmentation can also impact how the fragments interact with the terrain. Smaller fragments may be more easily influenced by the roughness and irregularities of the ground, leading to changes in their movement and travel distance. Additionally, the interaction between fragmented rocks and the terrain can result in secondary processes such as bouncing, rolling, or sliding, further influencing the rockfall behaviour.

Effects of rock block fragmentation should be considered, according to the geotechnical environment. A suggested approach is as follows:

- In general, effects of fragmentation are ignored for initial, high-level assessment purposes.
- Where more detailed assessment is required, reference to papers such as Corominas et al (2017), Fell et al (2008) or Hantz et al (2021), can be made for further detail of the assessment of fragmentation potential.

4 ESTIMATING VULNERABILITY TO ROCKFALL

Vulnerability of buildings and infrastructure to rockfall is usually considered in relation to kinetic energy (see MBIE, 2018). In terms of building vulnerability, Massey et al (2018) indicate that rock blocks with impact kinetic energies of < 2 kJ are unlikely to penetrate more than

0.1 m into a dwelling, which is similar to the typical thickness of the walls of light timber-frame wooden buildings studied. Massey et al (2018) indicates, in research undertaken by Grant et al. (2017), that a threshold may exist at ~10 kJ, below which very little damage to the dwelling occurs, but above which, significant damage is possible.

This means that even low intensity rockfalls can potentially result in high damage ratios and states, as shown on Figure 8.3. For a relatively slight increase in the kinetic energy of the boulder impacting the dwelling, there is a rapid increase in both the damage ratio and state.

From a life risk perspective, these results would suggest that for New Zealand dwellings, boulders with impact kinetic energies ≥10 kJ would likely penetrate the dwelling putting their occupants at some level of elevated risk.

5 ROCKFALL ANALYSIS AND MODELLING

Rockfall modelling is used to simulate falling rock trajectories and is commonly used to estimate the velocity (and hence energy) and bounce height of the rock block along the slope profile. In almost all cases, computer-based rockfall modelling is used to assess three aspects:

- Distribution of end points of travel: to assess the probability of travel
- Rockfall energy and bounce height: either, or both:
  - At the point of impact with an element at risk - is it possible for a rock to penetrate into a structure for example?
  - At the location of any proposed rockfall protection structure. This helps inform the design of any mitigation measure.

It is important to note that the choice of rockfall modelling software depends on the specific project, the complexity of the terrain, available data, and the level of expertise of the user. Although some software packages (RAMMS::Rockfall or Rockyfor3D for example) can be used to model detailed outcomes, they rely on input parameters that may be difficult to determine for many projects. Packages such as Rocfall2 or Rocfall3 are more widely used, but have some limitations, particularly in regard to the assessment of rock shape.

Regardless of the modelling software used, it is very easy to produce results that are overly conservative, and it is important to critically evaluate the model at every step. Modelling outputs must fundamentally make sense compared to the distribution of travel distance and indications of rock bounce heights that are observed on site; if they do not, the model needs to be adjusted! In some cases, it might be justified to run model roll trials to calibrate models (e.g., Kaikoura earthquakes, rock cuts in infrastructure projects during construction – Colgan & Ewe 2019).

Design of passive protection measures and associated modelling is covered in Appendix 2 of the MBIE Guidance (MBIE 2016) which should be referred to for further detail.

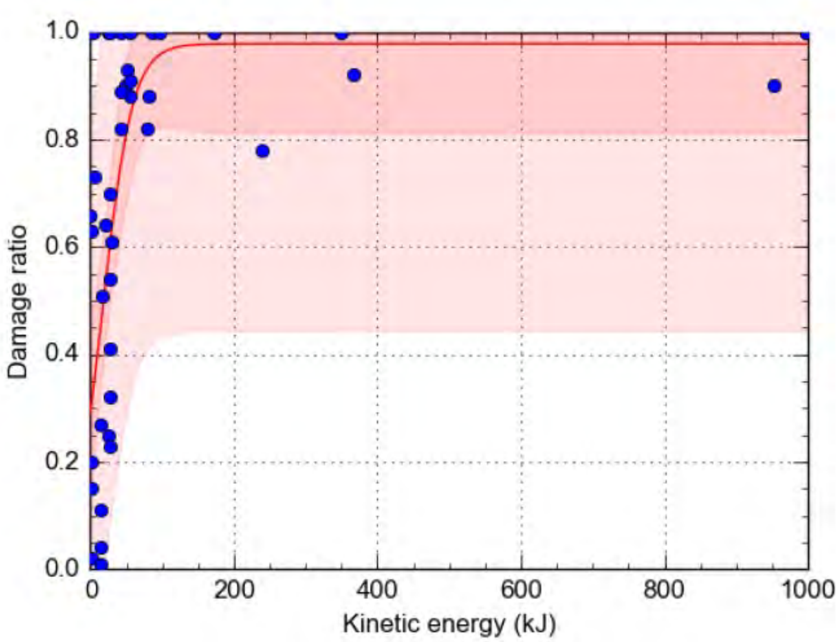


FIGURE 8.3 **Residential Building Damage Ratio v Rockfall Kinetic Energy (Massey et al, 2018). The darker red and lighter red shaded areas represent the 1st standard deviation and 95th percentile confidence range respectively.**

5.1 MODELLING ROCK SIZE DISTRIBUTION, ENERGY AND BOUNCE HEIGHT

Typical software packages assess the falling boulders with a statistical distribution by specifying a standard deviation, relative minimum and maximum:
3 x the standard deviation should cover almost all the distribution of the data set.

Fallen rock blocks typically follow a log-normal distribution, where the population is dominated by many small rocks but with occasional larger ones.
To model the distribution with any degree of accuracy, the geoprofessional needs to observe, at a minimum, the maximum rock size, and the average. Assumptions around the type of distribution, the mean and standard deviation can make a notable difference to the assessed energies. Having said this, it is important to note that the distribution of rock block sizes changes depending on distance from source as bigger rocks tend to travel further. The modelled distribution needs to reflect the distribution at the point of interest, which may differ from the distribution at the rockfall source area, particularly where fragmentation is significant.

For design purposes, the 95th percentile block size, bounce height and energy is typically considered. Although ONR 24810 (ASI 2017) recommends anywhere between the 94% - 99% boulder depending on the importance of the elements at risk, in practice it is very difficult to determine the size distribution to this level of accuracy.

It is important to note that for design purposes, the 95th percentile bounce height (or energy) of the 95th percentile rock block size is too conservative: instead, either the 95th percentile of the entire rock block size distribution should be used, or the mean height and energy if only the 95th percentile rock block is modelled, but not both.

6 FURTHER READING

More detailed explanation can be found in the following documents (among many others):

- Rockfall risk assessment principles: Massey et al (2012)
- MBIE (2016) Guidance for Passive Rockfall Protection Structures
- Design of rockfall fences: ONR24810

7 REFERENCES

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PART 9

DEBRIS FLOWS

Debris Flow Damage, Awataraiki Stream, Matata, following the 18 May 2005 debris flow McSaveney, et al (2005).

PART 9 – DEBRIS FLOWS

1 INTRODUCTION

Debris flows are mass movements that develop along drainage networks and involve generally dense fluids, composed of materials of different grain sizes, as well as entrained vegetation and variable amounts of water. Debris Flows, Lahars and Debris Floods are hazards that pose considerable risks to communities and infrastructure (Figure 9.1) due to their rapid speed to movement and destructive potential. Due to our mountainous landscape, there are many examples of housing and other infrastructure which are at elevated risk from these types of hazards.

While hillslope (or open slope) debris flows that form their own path down valley slopes are common, Part 10 of Unit 1 is intended to outline the characteristics of stream catchments subject to debris flows. It is principally based on current European and North American practices where there is considerable experience in dealing with debris flood and debris flow hazards. It provides a description of the field work that should be undertaken for steep stream catchments to allow assessment of the debris flow velocity, height and run-out distance for hazard and risk assessment and mitigation design purposes (design of mitigation is intended to be covered in much greater detail in proposed Unit 6).

As with other parts of this Slope Stability Guidance, it is not the intent of this section to provide a prescriptive format for the assessment of debris flow. Rather, the intent is to outline elements that the geoprofessional may need to consider when assessing debris flow hazards.

FIGURE 9.1. Damage to Road and Rail as a Result of the Jacobs Ladder Debris Flow, North of Kaikoura, February 2018. (photo supplied by NCTIR)

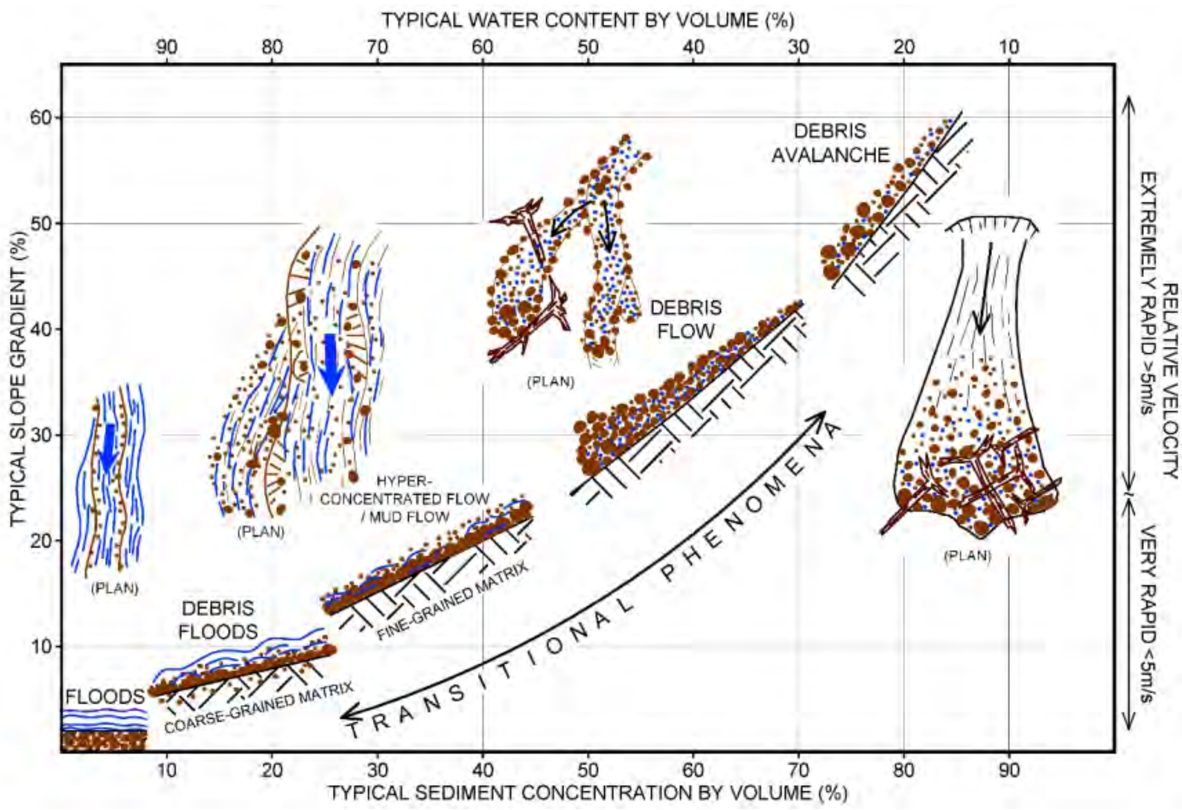


FIGURE 9.2 Steep Creek hazard processes as a function of sediment/water concentration, channel gradient and velocity. Cross-sections shown in correct location in relation to axes, but related plan sketches are displaced (Moase, 2017).

1.1 CLASSIFICATION

Clear-water Floods, Debris Floods, Hyperconcentrated flows, Debris Flows and Debris Avalanches are part of a continuum of processes and are primarily described based on sediment concentration, velocity, typical slope gradient and peak discharge. Figure 9.2 shows the general relationship between these processes.

There are many definitions in the literature used to describe these events. However, for the purposes of this guidance, the following definitions have been adopted, summarised from Moase (2017):

- **Debris Flow:** A very rapid to extremely rapid (5 – 10 m/s, 15 – 30 km/hr) surging flow of saturated non-plastic debris (which may include timber and other vegetation) in a steep channel (typically greater than 27% or 15°). The sediment concentration of debris flows generally is between 70 and 90 % by weight and >50 % by volume. Debris flows typically comprise one or many surges, each fronted by a concentration of boulders, with finer material behind (Figure 9.3). A debris-flow event may be composed of several of these surge fronts, spaced seconds, minutes or hours apart.

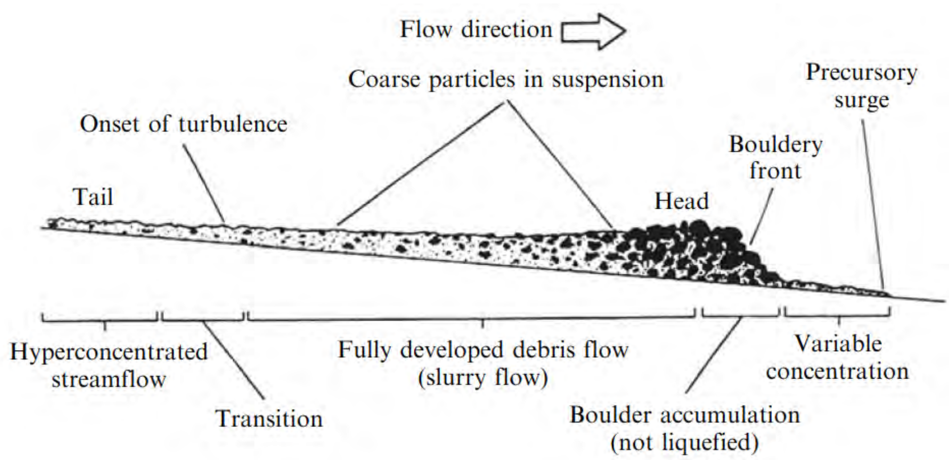


FIGURE 9.3 Cross-section of a typical debris-flow surge (Hungr, 2005)

- **Mud Flow:** Mud flows are similar to debris flows but incorporate soil with a plasticity index greater than 5%. Due to this increased plasticity and the associated high clay content which bonds free water, mud flows drain very slowly and have longer runout distances than (non-plastic) debris flows.
- **Lahar:** A debris, or mud, flow or flood composed principally of volcanic material. Lahars can occur during eruptions ("hot lahars") or during periods of high surface water runoff while the volcano is dormant ("cold lahars") (Hungr et al, 2014).
- **Hyperconcentrated Flow:** Transitional between debris floods and debris flows, having sediment concentrations between these two processes (Pierson, 2005). As shown on Figure 9.3, hyperconcentrated flows can occur at the distal end of a debris flow surge.
- **Debris Flood:** Very rapid flow (between 0.05 m/s and 5 m/s) of water, heavily charged with debris. Peak discharge is comparable to that of a water flood. Debris floods are distinguished from Clear Water floods when sediment on the bed begins to move together, *en masse*, and larger particles become suspended because of bedload destabilisation. Sediment concentration is typically less than 25 % by volume. Debris floods lack the surging, bouldery front which characterise debris flows.
- **Clear Water Flood:** Floods that typically transport sediment particle-by-particle as bedload or suspended sediment, generally comprising less than 4 % by volume.

2 MORPHOLOGY OF STREAM CATCHMENTS SUBJECT TO DEBRIS FLOW

A debris flow system can be divided morphologically into three main zones (Initiation, transport, deposition) as shown on Figure 9.4.

The **catchment** comprises the initiating or 'sediment production' area for the particular debris flow or flood. In this part of the system, sediment and debris is typically delivered to the upper stream channels by rock fall, rockslides, debris avalanches, debris flows, slumps and raveling.

The **main channel** comprises the confined transportation zone of the debris flow or flood, where sediment bulking may occur. In this process, rapidly flowing water entrains bed and bank materials either through erosion or preferential "plucking".

The **debris, or alluvial, fan** represents a depositional landform at the outlet of a steep stream catchment. The alluvial fan comprises the unconfined area of the system and represents the approximate extent of deposition of past flow and flood events (see Figures 9.1 and 9.4).

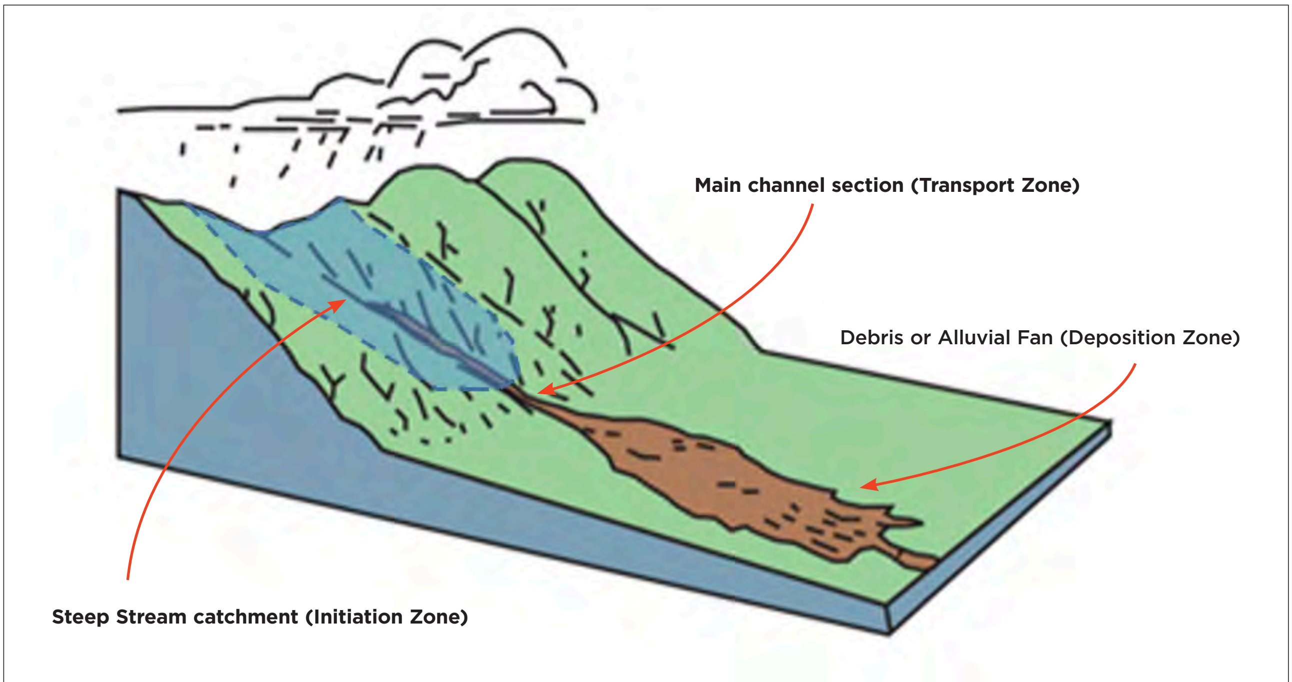


FIGURE 9.4: Typical Debris Flow Zones.

2.1 CATCHMENT CHARACTERISTICS

Debris flow catchments typically comprise hillslopes and small feeder channels, leading to a principal channel. The hillslopes deliver sediment and debris to the upper channels by rock fall, rockslides, debris avalanches, debris flows, slumps and ravelling (BGC, 2019).

Catchments subject to debris flows can typically be categorised (Figure 9.5) as being either:

- **Supply-limited:** meaning that debris available for mobilisation as a debris flow is a limiting factor on the magnitude and frequency of steep catchment events. Due to our young geology and steep topography, these catchment types are relatively rare in New Zealand. They require a significant recharge period prior to each debris flow event and exhibit a lower frequency of debris flow activity; or
- **Supply-unlimited:** meaning that debris available for mobilisation is not a limiting factor on the magnitude and frequency of debris flows. In this case, there is always an abundance of debris along a channel and in source areas so that whenever a critical rainfall (or another triggering event) is exceeded, an event will likely occur. The more severe the rainfall event, the higher the resulting magnitude of the debris flow or debris flood.

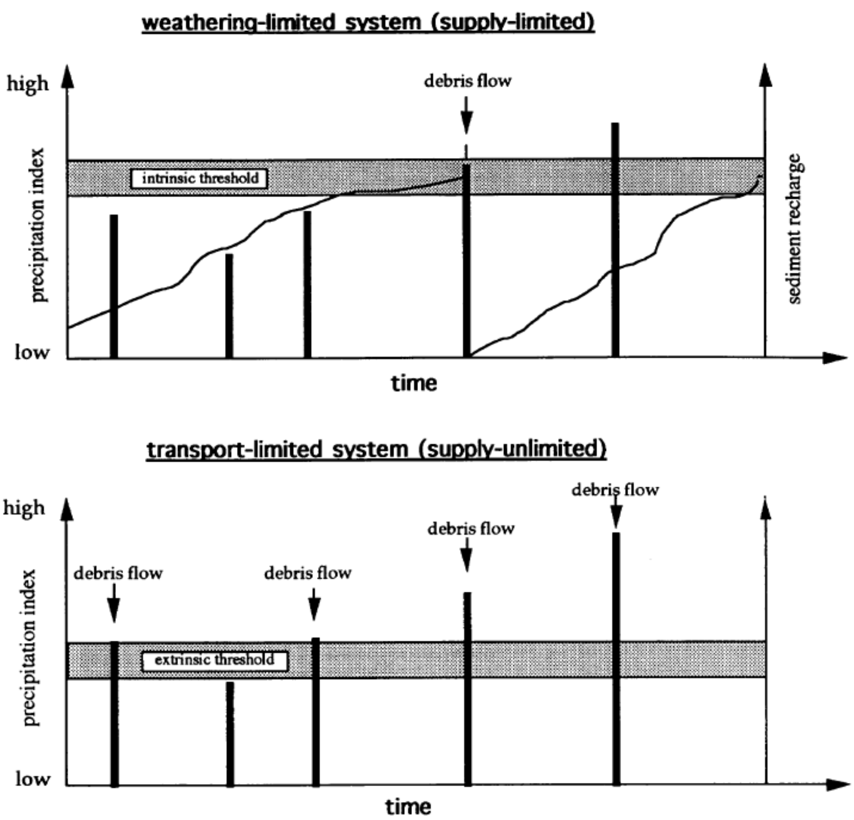


FIGURE 9.5A: Conceptual debris flow initiation frequency between supply-limited and supply-unlimited catchments. Bars indicate precipitation, rising lines indicate cumulative sediment recharge (Bovis & Jakob, 1999)

FIGURE 9.5B: Left - Supply unlimited catchment: Shallow landsliding supplying sediment and debris to stream, Matanga West Catchment, Golden Bay, December 2011 (Page et al, 2012). Right – Supply-limited alpine catchment, near Franz Josef (Bowman & Kailey, 2010). Sediment availability in the channel will rely on erosion from the rock slopes either side, which would take time to build up.

2.2 MAIN CHANNEL CHARACTERISTICS

The Main Channel broadly represents the zone of transportation, between the initiation zone (the catchment) and the depositional zone on the alluvial fan. Significant entrainment of sediment and vegetation may occur along the main channel. This is referred to as sediment bulking, as seen for example in Figure 9.6. Sediment bulking involves the phenomenon in which swiftly moving water incorporates bed and bank materials, either through erosion or selective 'plucking,' until a specific sediment conveyance capacity (saturation) is attained. Bulking can range from partial mobilisation of the top gravel layer in the channel substrate to, in the instance of debris flows, the entrainment of the entire loose debris to bedrock within the channel (Hungr et al, 2005).

2.3 FAN CHARACTERISTICS

Debris flow fans or alluvial fans are broadly cone-shaped landforms that are generated when the confined watercourse (creek or stream) becomes less confined. The resulting decrease in flow velocity promotes sediment deposition, forming the fan.

Partial material deposition or coarser grained material may occur on the upper part of the debris fan, with more complete deposition of finer grained material occurring on the lower part of fan. However, deposition and runout of debris flows are governed by several factors, especially a decrease in slope and a lack of flow confinement. Very coarse-grained debris flows, typically originating in small catchments, may start to deposit at gradients as high as 27°, while lahars may travel several tens of kilometres, arresting at gradients of only a few degrees (Jakob and Hungr, 2005).

FIGURE 9.6: Bank erosion adjacent to the main channel of a small debris flow catchment (photo courtesy ENGEO)

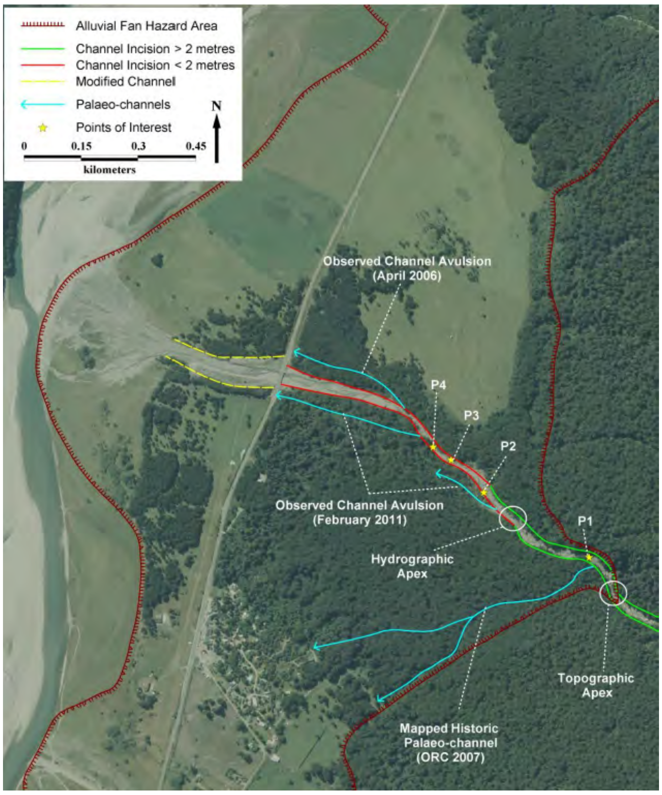


FIGURE 9.7. **(Left) Example of Avulsed Debris Flow, Gunns Camp, February 2020 (photo courtesy ENGEO). (Right) Key Features of the Pipson Creek Alluvial Fan (ORC, 2011)**

The dynamic nature of debris flow fans may pose a significant hazard to people and infrastructure located on them. Areas that were inactive (and have been built on) can reactivate and start actively eroding or receiving sediment. In addition, the debris flow process of channel plugging, backstepping of deposition toward the fan apex over one or more flows, avulsion and establishment of a new active channel means that a simple increase in distance from the fan apex may not reduce the probability of inundation (see Figure 9.7 for examples). Accordingly, simple run-out analyses based on F-Angle should be considered as a preliminary estimate only. Mapping of the fan extent and morphology becomes critically important.

As the debris flow reaches the apex of the depositional fan, the channel widens and coarse debris is expelled to the sides to form steep ridges or levees. The front may be bypassed by the finer liquefied debris travelling behind it and come to rest as a thick boulder train or lobe. Such forms, together with large boulders and other evidence of high discharges and impacts, constitute "silent witnesses," indicating past occurrences of debris flows (Hungr et al, 2002).

3 MORPHOMETRIC ASSESSMENT

A statistical analysis of Melton Ratio and watershed/catchment length can be used to provide an indication of whether a catchment is likely to produce clear water floods, debris floods or debris flows (Wilford et al., 2004). The Melton Ratio R is defined as:

$$R = \frac{H_b}{\sqrt{A_b}}$$

Where

H_b is the watershed/catchment relief (elevation difference between the highest and lowest point in a watershed (km) and

A_b is the watershed/catchment area in plan; (km^2)

The watershed/catchment length (unit: km) is defined as the planimetric straight-line length from the fan apex to the most distant point on the watershed boundary.

Morphometrically, catchment systems with high values of Melton Ratio (R) and shorter stream length are mostly prone to debris flows, and those with lower values of R and longer catchment stream lengths are mostly prone to floods (debris or clear water). Table 9.1 indicates the typical class boundaries between floods, debris floods and debris flows.

Welsh and Davis (2011) indicate that a Melton Ratio of >0.5 for debris flow may only apply to steep catchments set in mountain ranges characterised by long slopes at about the angle of repose, and not incised coastal catchments (such as the catchments that generated the Matata debris flows in 2005).

TABLE 9.1: Class boundaries using Melton ratio and total stream network length with consideration for NZ conditions (based on Welsh and Davies, 2011)

PROCESS	MELTON RATIO	CATCHMENT LENGTH (KM)
Clear Water Floods	< 0.2	All
Debris Floods	0.2 to 0.5	All
	> 0.5	> 3
Debris Flows	> 0.5	≤ 3

4 SITE ASSESSMENT

Site assessment of Debris Flow/Flood hazards for risk assessment or mitigation design purposes should consider:

- The characteristics of the steep stream **catchment** where the flow is most likely to be initiated (Section 4.1)
- Features from the main channel where debris is transported but where sediment and/or vegetation entrainment can occur (the **transportation zone** - Section 4.2).
- The characteristics of the **alluvial fan**, where most of the debris flow material is deposited (Section 4.3).

The following sections provide some detail of observations and analyses that should be considered in the catchment, within the transportation zone, and on the depositional fan to quantify the hazard.

4.1 CATCHMENT

The catchment comprises the initiating or 'sediment production' area for the particular debris flow or flood. In this part of the system, sediment and debris is typically delivered to the upper channels by mass movement processes such as rock fall, rockslides, debris avalanches, debris flows, slumps and raveling.

Elements that should be assessed in the Debris Flow / Flood catchment are summarised in Table 9.2.

TABLE 9.2: Geotechnical Assessment Elements - Catchment

ITEM	PURPOSE OR REASON	INFORMATION TO BE COLLECTED OR PREPARED	SOURCE OR REFERENCE
Catchment / Source Material / Land use	Critical for understanding the composition/ history of the underlying geology and associated debris; e.g. blocky source material vs muddy or volcanic	• Presence, and estimated thickness of soil, • Typical block sizes and estimate of the boulder sizes that could be transported • Presence of material available within the current channel.	Geological maps, Historical aerial imagery (particularly if this predates significant vegetation growth) Site Walkover Assessment, Drone Survey etc.
Evidence of previous mass movement within catchment	Critical to understand the availability of material for re-mobilisation (i.e., soil creep, landslides, rockfall, erosion etc.)	• Failure types, size; location, • Potential travel angle; • Potential volume, block sizes and shape; • Evidence of trajectory	Historical aerial imagery (particularly if this predates significant vegetation growth), Site Walkover Assessment, Drone Survey, LiDAR, DEM etc.
Historical Rainfall	Provides information on the historical rainfall events that the catchment may have received, which can be considered against rainfall depth or intensity for differing return period events	• Rainfall records, in particular during known high intensity rainfall events (cyclones, storms etc.), which can be assessed against historical debris flow initiations	Site observations; regional/ local council rain gauge records, HIRDS
Vegetation	Gives indication of occurrence of events; potential to be included in source of debris flow material	• Type of vegetation; changes in vegetation pattern, how well established (age); • Impact scars; loss of branches from trees to equivalent height	Historical aerial imagery, Site observations
Hydrology	Analysis of baseline flow characteristics (clear-water peak flows), verification of hydraulic capacity of existing infrastructure (e.g. culverts, channels, road crossings)	• Water flow data (if available), • Rainfall records, • Topographical data, • Soil/ rock type (e.g. possibility of karst and rapid rise of groundwater), • Catchment geometry, dimensions and elevations of existing infrastructure; anecdotal information	Site observations; regional/ local council rain gauge records, as-built records of infrastructure, HIRDS, LRIS, LiDAR, DEM

4.2 MAIN CHANNEL

The main channel comprises the confined transportation zone of the debris flow or flood, where sediment bulking may occur. This is the process by which rapidly flowing water entrains bed and bank materials and vegetation either through erosion or preferential "plucking".

Elements that should be assessed in the Confined Channel and on the Alluvial Fan are summarised in Table 9.3.

4.3 ALLUVIAL FAN ASSESSMENT

The alluvial fan represents a depositional landform at the outlet of a steep stream catchment. The alluvial fan comprises the unconfined area of the system and represents the approximate extent of deposition of past flow and flood events.

Elements that should be assessed in the on the Alluvial Fan are summarised in Table 9.4.

TABLE 9.3: Geotechnical Assessment Elements – Main Channel

ITEM	PURPOSE OR REASON	INFORMATION TO BE COLLECTED OR PREPARED	SOURCE OR REFERENCE
Main Channel Characteristics	Inform potential flow velocity, flow paths and avulsion	• Length, width and channel geometry; • Sediment at the base and depth of incision; • Evidence of erosion along banks; • Levees, strandlines, mudlines, trim line; bend angles; • Evidence of particle jamming, superelevation, yield rate, • Vegetation density, type and age of vegetation within the stream bed	Site observations, Drone survey, LiDAR data, DEM, depending on vegetation growth

TABLE 9.4: Geotechnical Assessment Elements – Alluvial Fan

ITEM	PURPOSE OR REASON	INFORMATION TO BE COLLECTED OR PREPARED	SOURCE OR REFERENCE
Fan morphology (Figure 9.8)	Critical for understanding the hazard; i.e. historic run out directions and extents.	• Dimensions and location of depositional lobes and levees; • Evidence of deposition and avulsion; • Changes in clast deposition; • Areas of erosion, presence of abandoned channels	Site observations; initially identified in historic aerial photographs or LiDAR / DEM and then assessed/ recorded onsite.
Vegetation	Gives indication of occurrence of events and potential for vegetation to be included in the source of debris flow material	• Type of vegetation and how well established it is; • Impact scars, • Proximity to channel	Site observations, aerial photographs
Subsurface Profile (Figure 9.9)	Evidence of historical debris flows to inform assessment of potential frequency/magnitude relationships, flow thickness	• Lack of stratification and particle sorting. • Clast orientation (typically random in debris flow deposits) • Presence of matrix supported angular to sub-angular cobbles and boulders; • Presence of buried topsoil or vegetation horizons	Logging of natural exposures, road cuts etc; test pits; (hand augers are likely to be of limited use); PQ machine boreholes

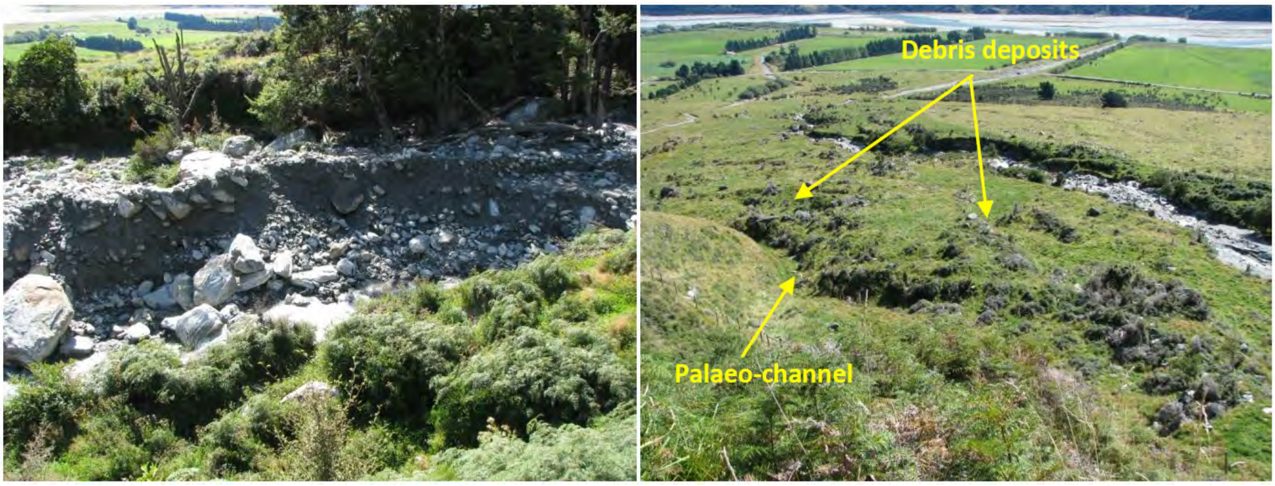


FIGURE 9.8. Flaxmill Alluvial Fan, showing recent channel aggradation (left), large debris deposits and paleo-channel (right). ORC (2011).

(a)

(b)

FIGURE 9.9. (a) Paleo debris flow material exposed in stream bank. (b) Note the unstratified, poorly sorted nature of the deposits and the matrix-supported angular clasts (Page et al, 2012)

5 ESTIMATING DEBRIS FLOW HAZARD

As outlined in Part 6, the annual risk of loss of life can be calculated as

$$P_{(LOL)} = P_{(L)} \ x \ P_{(T:L)} \ x \ P_{(S:T)} \ x \ V_{(D:T)}$$

For debris flow hazards:

- Estimating the probability of landslide occurrence ($P_{(L)}$) ideally should be based on an assessment of the catchment, channel and fan characteristics as described in Tables 9.1 to 9.3. Further commentary is provided in Section 5.1 following.
- The probability of travel (P_t) may be initially estimated by considering debris flow magnitude (volume) as this significantly affects the extent of run-out and inundation
- Vulnerability ($V_{(D:T)}$), particularly of buildings, is significantly influenced by flow velocity (primarily) and secondarily by flow height, both of which are related to debris flow magnitude and peak discharge.

5.1 ESTIMATING PROBABILITY OF OCCURRENCE

Debris flows typically initiate when heavy rain occurs in the presence of transportable materials. The rain generally needs to be intense, but in order to be able to mobilise material it will also (most often) be preceded by rainfall over sufficient time to enable saturation in the initiation area. Therefore, unlike clear water floods, where a volume of water can be anticipated under a certain return period event, a debris flow might not occur simply because high intensity rainfall has happened, and if it does occur, may have a volume smaller, or larger, than that expected for clear water.

Flow behaviour is complex, and any assessment involves significant uncertainty. Ultimately, the best information when assessing the probability of occurrence of debris flow in a particular catchment is based on historical information. In general terms, the frequency of smaller debris flow events may be able to be assessed via historical records, assessment of fan stratigraphy combined with dating of carbonaceous horizons, consideration of vegetation growth etc. to plot a frequency magnitude curve. The magnitude of significantly less frequent flows may need to be considered by assessing the hydrological opportunities or limitations of the steep creek catchment; small catchments are unlikely to be hydrologically capable of releasing very large debris flows.

Where a debris flow has not recently happened, the assessment of the probability of occurrence becomes much more difficult and is best undertaken through a thorough understanding of the morphology of the area of deposition and potential catchment. Several empirical approaches can be applied as described in Section 5.2. Alternatively, numerical modelling could be considered. However, as with other analyses, both are subject to some uncertainly such that any outputs need to be carefully calibrated to all available site information and assumptions and sensitivity clearly set out.

5.2 ESTIMATING DEBRIS FLOW PARAMETERS

Flow height, and most importantly, velocity at the point of impact with an element at risk are the key parameters for assessing debris flow risk and as the basis for the design of mitigation works. Figure 9.10 shows a simple flow chart for the sequence of assessment of these parameters. Debris flow volume (M) can be estimated as described in Section 5.2.1 below, and then can be used to derive an estimate of the associated peak discharge (Q_p), flow velocity (V), height (h), the total travel distance (L) and/or the run-out distance of the fan (L_f).

5.2.1 Debris Flow Volume (M)

In terms of hazard evaluation, determining the potential debris flow volume is a crucial parameter for establishing

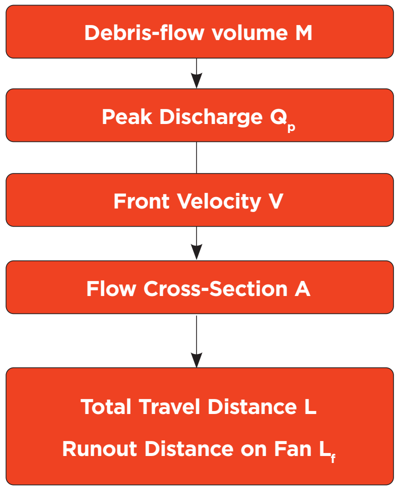


FIGURE 9.10. Flow chart for estimating Debris Flow Parameters (Fuchs et al, 2008)

the overall Magnitude / Frequency relationship and assessing potential inundation hazards. Estimating the magnitude of a Reasonable Worst Case (RWC) debris flow is also essential for design purposes and may also prove valuable for risk assessment purposes.

Flow volume can be estimated using, in order of preference, direct assessment, empirical relationships, or hydraulic assessment considering a bulking factor, as described below. A combination of approaches is likely required where historical information is unavailable.

Direct Assessment

Direct assessment involves measuring potential debris flow volumes based on site-specific assessment. Any debris flow volume comprises three elements:

- The volume of the initiating failure or failures;
- The volumes entrained along the transport reach (the 'yield rate'); and
- Volumes deposited along the transport reach (however this is a smaller quantity compared to the previous two).

Potential volumes of the initiating failures can be estimated from observations of the catchment area as described in Table 9.2 considering the proportion of the catchment that could fail in a given return period event. Frequent events would be expected to involve one or two small failures, whereas a RWC event may involve a relatively large proportion of the catchment.

The yield rate for a particular channel can be estimated by assessing the volume of erodible material per metre of channel length following the method of Hungr et al, (2005). The concept divides a debris flow catchment into a number of 'reaches'. As shown schematically in Figure 9.11, each reach is approximately constant in terms of:

- Channel slope angle, width and depth, bed material
- Bank slope angle, height, material and stability
- Tributary drainage area or discharge.

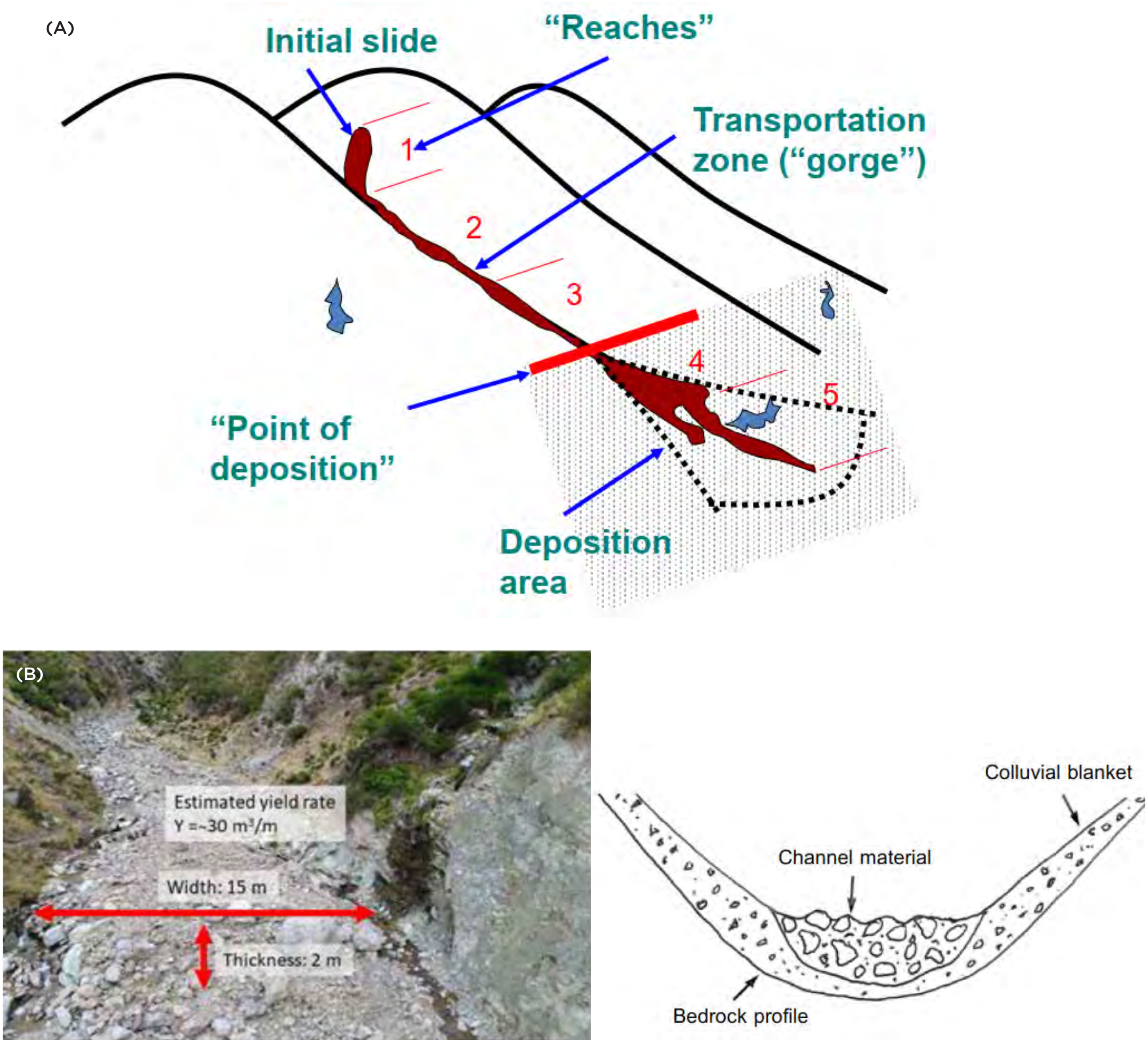


FIGURE 9.11. (A) Concept of 'reaches' along the length of a debris flow (taken from a presentation by Hungr (2016); (B) Yield Rate concept in each reach (Golder Associates, 2019).

Once the applicable yield rates are estimated, the volume (V) of the debris flow can be estimated by

$$V = V_{initial} + \sum V_{point} + \sum_{i=1}^{n} Y_i L_i$$

Where

$V_{initial}$ is the initial volume of the debris flow released from the main source areas

V_{point} are the volumes from other point sources (tributary channels, secondary failures etc0

Y_i is the assessed yield rate at the i-th section of the channel, and

L_i is the length of the i-th section of the channel

Empirical Volume Relationships

Figure 9.12 provides a summary of some empirical relationships that have been developed relating catchment area to debris flow volume for granular debris flows. As can be seen in the relationships shown by Marchi et al (2019), significant variation is apparent between the 50th percentile and 99th percentile volume estimate, with all other relationships located between these two. Which relationship is more appropriate to adopt depends on the intended analysis:

- For risk assessment purposes, volumes closer to the 50th percentile may be appropriate.
- For design purposes, it is suggested volumes towards the upper end of the range (closer to the Marchi et al 98th or 99th percentile relationships) are adopted.

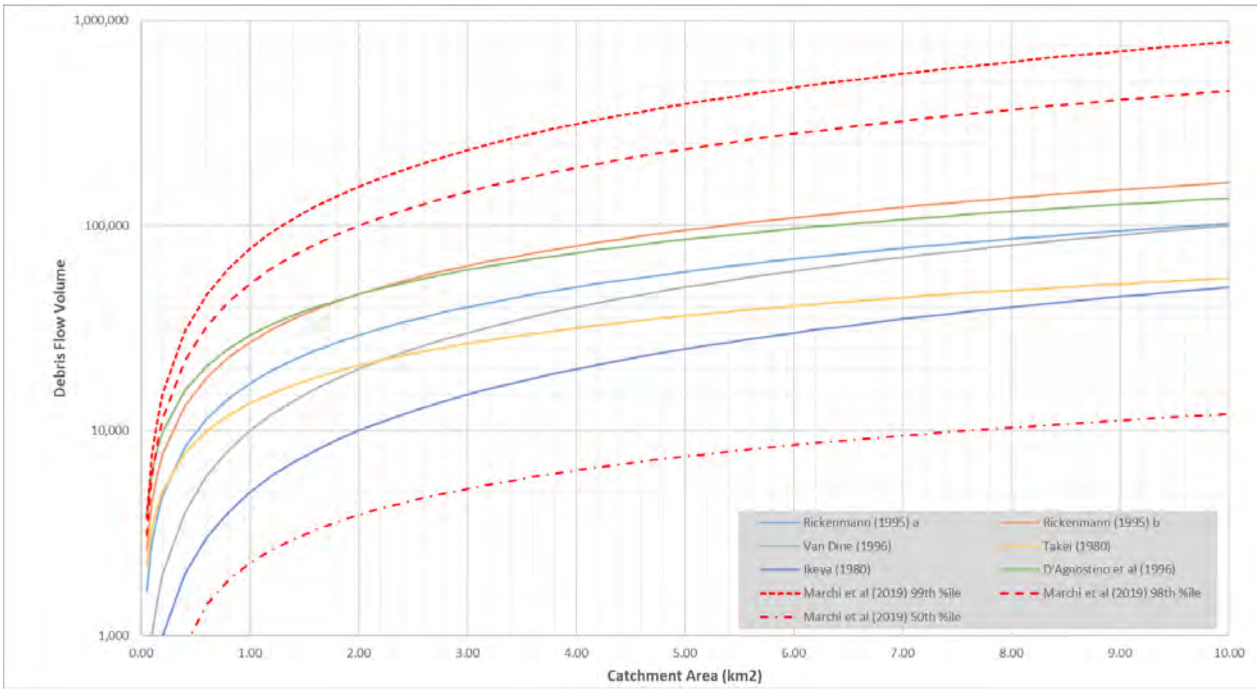


FIGURE 9.12. Catchment Area in Relation to Potential Debris Flow Volume (M).

Hydraulic Assessment

As an alternative to the methods outlined above, debris volumes for a range of annual recurrence intervals (ARIs) can be assessed by undertaking hydrological analyses and then multiplying the clearwater flood volumes by a debris bulking factor. While this is a useful method to assess potential debris flow or flood volumes, it is important to understand that a debris flow may not be initiated simply because a particular ARI has been exceeded: debris flow initiation depends on a number of other factors in addition to rainfall.

Bulking factors depend on both the type of event (debris flows have a higher bulking factor compared to debris floods) and the location of the element at risk (higher bulking factors are applicable close to the alluvial fan apex, whereas lower factors are more appropriate in more distal locations). As a preliminary guide, volume bulking factors as outlined in Table 9.5 could be considered.

5.2.2 Peak Discharge (Q_p)

Knowledge of the peak discharge and the associated flow velocity are important when evaluating the conveyance capacity of stream channel reaches or critical cross-sections as, for example, under bridges.

Peak discharge can be assessed empirically from event magnitude, as shown in Figure 9.13, Table 9.5, or via hydraulic assessment considering catchment area and clearwater runoff volume. Hungr (2005) suggests that Q_p should be bulked from the clear water peak (Q_f) by the following factors (note these are not the same as the volume bulking factors outlined in Section 5.2.1 due to surge behaviour):

- Debris Floods: 2 to 3 x Q_f
- Debris Flows: 10 to 50 x Q_f

TABLE 9.5 Indicative bulking factors for debris floods and debris flows

VOLUME BULKING FACTOR	APPLICABILITY
1.0 – 1.25	Clear-water and debris floods; projects located at the downstream end of alluvial fans where the flow is widely dispersed and diminished.
1.25 – 1.67	Hyper-concentrated flows; projects located in the lower portion of debris flow alluvial fans and outside of flow channels, where flows have dispersed
1.67 – 3.5	Projects located near the alluvial fan apex of Debris Flows where flows are likely to be undispersed

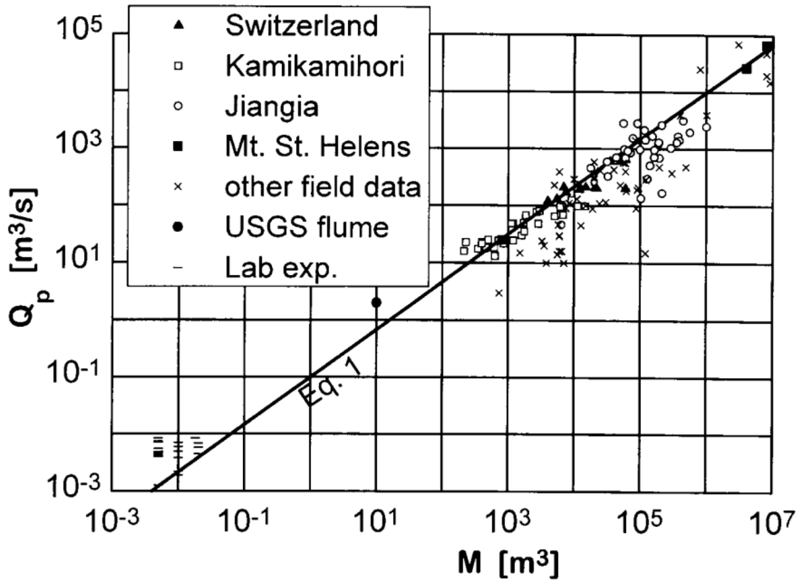


FIGURE 9.13. Peak discharge (Q_p) of debris flows vs debris-flow volume (M); Rickenmann (1999)

5.2.3 Debris Flow Velocity (V) and Flow Height

The velocities of debris flows vary widely due to differences not only in the character of the debris, such as grain concentration and grain size distribution, but also in the shape of the course of passage such as its width, slope, etc. Observed velocities have been reported between about 0.5 m/s and 20 m/s (Takahashi, 1981).

The flow height is considered to be the height of the top of the flow above the ground surface. In general terms the flow height is dependent on the discharge and confinement, meaning flow heights are typically greater for debris flows that are confined by steep gullies or incised channels.

Many empirical relationships have been developed that relate peak discharge to flow velocity and flow height, a few of which are set out in Table 9.7. Note however that these formulae give the peak velocity in the confined channel. On the alluvial/ debris fan, where the debris flow becomes unconfined and is able to widen, these formulae would overestimate the velocity.

5.2.4 Runout Distance and Extent of Inundation

Runout distance varies considerably depending on the:

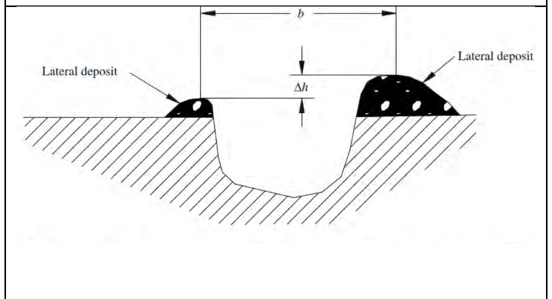
- Type of movement: fluidised debris flows tend to travel further than dry flows or avalanches
- Movement volume: larger volumes typically travel further, and
- Degree of confinement: confined flows travel further than unconfined flows.

As an initial estimate the F-angles provided in Table 9.6 could be used. However, these values are conservative, and should be used as an <u>initial guide</u> only.

TABLE 9.6 F-angles (°) for channelised flows and open-slope avalanches for different landslide volumes (from de Vilder and Massey, 2020). Refer Part 6 for further information on F-angle.

Landslide Volume (m³)	10%-Passing Fahrböschung Angle	
	Channelised Flow	Avalanche
10	22	38
100	18	36
1000	14	34
10,000	11	32
100,000	9	30
1,000,000	7	28

TABLE 9.7 Selected Empirical Relationships. Note that these relationships typically have been developed based on specific datasets and areas, and the calculated outputs should be treated as approximations only.)

PEAK DISCHARGE		FLOW VELOCITY		FLOW HEIGHT		RUNOUT DISTANCE AND AREA OF INUNDATION	
Ikeda et al. (2019) Rickemann (1999)	$Q_p = \alpha \cdot M^{0.833}$ Qp= the debris-flow peak discharge [m³ /s], M = Debris flow magnitude [m³], and α = tend to approximate 0.01 if the debris flows are muddy but approximate 0.1 if the flows are granular.	Volkwein et al., (2011)	$v = 2.1 \cdot Q_p^{0.34} \cdot I_s^{0.2}$ v = velocity at the front of the flow Q_p = peak discharge I_s = tangent of the slope inclination in degrees	Volkwein et al., (2011)	$h_{fl} = \frac{Q_p}{v \cdot b_u}$ Q_p = Design debris flow discharge b_u = bottom width of channel section v = velocity h_{fl} = flow height (h_{fld} = design flow height) For estimation of flow height for debris flows that are not confined by topography or have avulsed from their channel, b_u should be considered based on previous channel widths evident from a morphological assessment of the debris fan.	Hurlimann et al, (2008)	$L_{max} = 1.9M^{0.16}H^{0.83}$ $L_{fan} = 15V^{1/3}$ Where Lfan = runout distance from the apex of the alluvial fan
Mizuyama et al. (1992)	$Q_p = 0.135 \cdot M^{0.78}$ For granular debris flows $Q_p = 0.0188 \cdot M^{0.79}$ For muddy debris flows Q_p = design debris flow discharge v_{DF} = total debris flow volume (m³) Q_{Pd} = peak discharge (typically between 5 m³/s and 30 m³/s)	Volkwein et al., (2011)	$v = \frac{1}{n_d} \cdot h_{fl}^{0.67} \cdot I_s^{0.5}$ n_d = pseudo-Manning value *lies between 0.05 s/m^1/3 and 0.18 s/m^1/3, while the values for granular debris flows lay between 0.1 s/m^1/3 and 0.18 s/m^1/3* I_s = tangent of the slope inclination in degrees h_{fl} = flow height of the debris flow v = flow velocity	Prochaska et al (2007)	$v = \sqrt{\frac{rg}{k}\frac{\Delta h}{b}}$ k = a correlation factor related to viscosity and vertical sorting that exists in coarse grained soils. Prochaska et al (2007) suggest that this may be 1.0 in most cases, but could up to 5.0 b = surface width of flow v = mean velocity r = mean radius of curvature, in plan g = acceleration due to gravity Δh = elevation difference between the two sides of the flow	Area of Inundation Jakob (2005)	$B_v = 200V^{2/3}$ for volcanic debris $B_b = 20V^{2/3}$ for bouldery debris
		VanDine (1996)	$v = \frac{\gamma \sin\theta\, h^2}{l\, V}$ θ = channel gradient h = flow depth γ = unit weight of debris mass V = dynamic viscosity of debris mass (see, for example, Kostynick et al, 2022) I = a constant based on the cross sectional shape of the channel (=3 for a broad channel, =8 for a semi-circular channel)		Note here that Δh is a super elevation (so not flow height per se, at a curved portion of the flow channel, as shown in the diagram below) 		

Alternatively, run-out distances could be judged using the empirical formulas provided in Table 9.7.

The area of inundation provides a measure of debris flow mobility and potential consequences (Jakob, 2005). Bouldery debris flows will spread a smaller distance compared to muddy (mostly volcanic) debris flows because the latter will spread over larger areas due to their high mobility.

5.3 NUMERICAL MODELLING

Where the results of initial assessment outlined above suggest that there may be a significant risk, more sophisticated numerical analyses may be required.

There are several software packages available that may be used to model debris flow/ debris flood behaviour, in particular the potential extent of run-out, point velocity and flow thickness. Commonly used software for modelling debris flows include RAMMS::Debris Flow, Flow-R and FLO-2D. Some of the benefits and limitations of each package are summarised in Table 9.8 (partly based on Cesca & D'Agostino, 2008, Horton et al, 2013). This is not an exhaustive list, and geoprofessionals may wish to consider alternatives which may be more suitable for their project needs.

All of the currently available modelling packages simplify flow behaviour and properties to some extent and should be considered as tools in the toolbox only.

As with other software packages used in geotechnical engineering, the quality of the output cannot be better than the input and therefore, careful calibration of the model by geomorphic observations and ground truthing remains critical (Figure 9.14). In addition, model results are dependent on high resolution digital elevation models, which are not always available and can often not be obtained within the project budget.

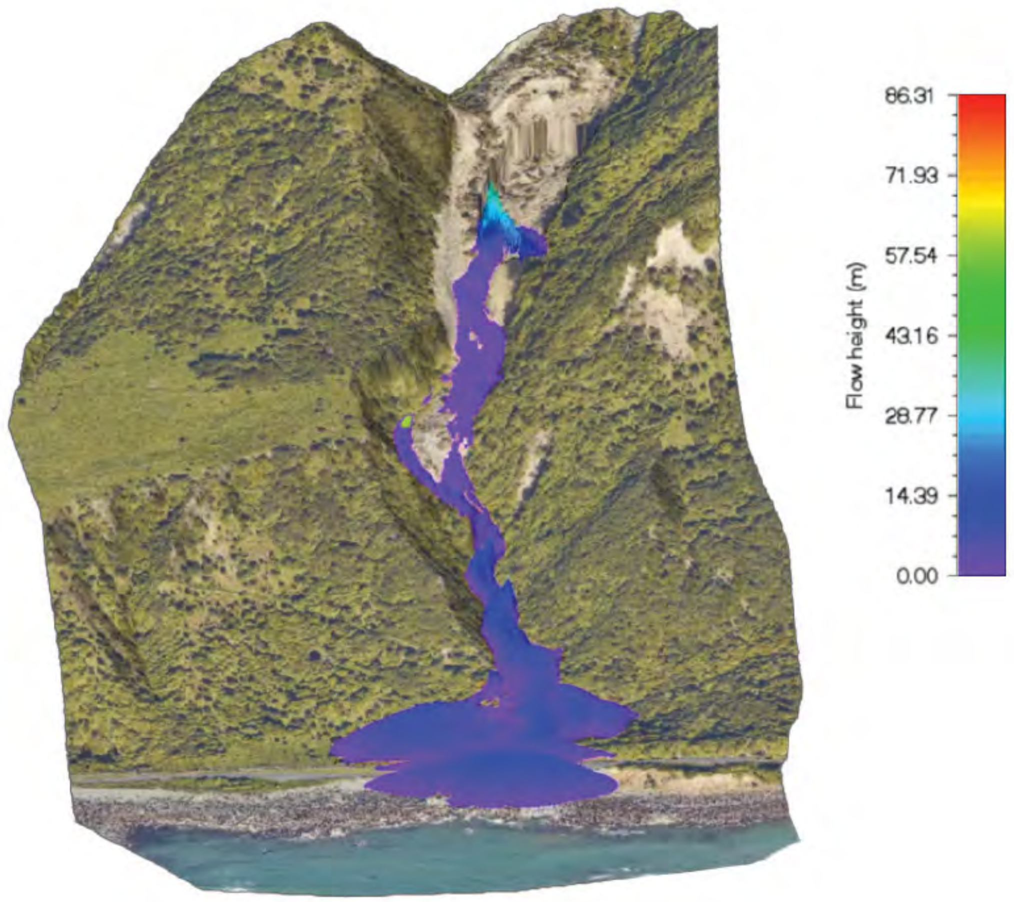


FIGURE 9.14 RAMMS back analysis output for the February 2018 debris flow at Jacobs Ladder (compare Figure 9.1). Darafshi & Borella (2020)

TABLE 9.8 Benefits and limitations of some commonly used debris flow software packages

SOFTWARE PACKAGE	BENEFITS	LIMITATIONS
RAMMS::Debris Flow	• Developed specifically to simulate the runout of muddy and debris-laden flows in complex terrain • Widely used • Combines numerical solution methods with helpful input features and user-friendly visualization tools • Can be used for modelling small and extremely large debris flows • Many of the input and output features have been optimized to allow geoprofessionals to – define event scenarios, – evaluate simulation results, and – predict the influence of proposed structural mitigation measures on the runout of debris flows.	• Debris flow simulation using the RAMMS model requires a number of input data including a DEM, peak runoffs, and soil volume • Model calibration using debris-flow post-event survey field data is an essential step (and should be done before application of the model)
Flow-R	• A distributed empirical model for assessing regional susceptibility to debris flows • Successfully applied to different case studies in various countries. • Allows for automatic source area delineation and for the assessment of the propagation extent. • Choices of the datasets and the algorithms are open to the user. • Also suitable for assessing other natural hazards such as rockfall or snow avalanches.	• Only allows identification, at a preliminary level of detail, of potential debris-flow or debris-flood hazard and modelling of their runout susceptibility at a regional scale • Cannot simulate bank erosion, channel scour and aggradation, all of which can affect flow behaviour and thus risk • Cannot model avulsions that are likely at culverts and bridges and which could redirect flow out of the channel
FLO-2D	• FLO-2D is a flood routing model that combines hydrology and hydraulics • Developed in 1987 to predict mudflow hydraulics • Since adapted to conduct any sort of overland and channel modelling type (e.g. urban flood mapping, alluvial fans, coastal flooding). • Uses QGIS and the FLO-2D Plugin to build models. • Ability to integrate different types of geospatial data e.g., LiDAR, aerial images, shape files, contour maps and DEM • Can import HEC-RAS geometry cross-sections	• Primarily a flood routing model • Grid element represents single elevation, Manning's n value, and flow depth • Hydraulic structures and rating table are developed outside of the model • Rapidly variable flow (i.e. a dam breach) is not simulated • 1D channel flow (no secondary currents, or vertical velocity distributions)

6 ESTIMATING VULNERABILITY TO DEBRIS FLOW

As outlined in Part 6 of Unit 1, the degree of physical damage to property and infrastructure can be considered in terms of a 'damage state', which describes the amount of damage in relation to the ability of the building or infrastructure to function normally. In contrast, damage ratio describes economic loss. It is calculated by dividing the cost to repair a damaged asset by the cost of replacing the asset.

Massey et al (2018) present two figures which compare debris flow velocity (Figure 9.14) and debris height (Figure 9.15) to damage state or damage ratio. While Massey et. al. expresses some concern that there appears to be no obvious statistical relationship between the damage state and debris height or velocity for local and international data, these graphs remain very useful in assessing potential building structure vulnerabilities to debris flows and floods.

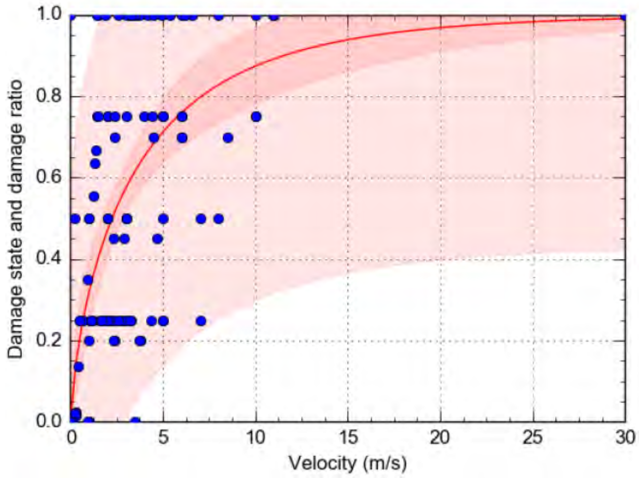


FIGURE 9.14. **Residential Building Damage State v Debris Flow Velocity (Massey et al, 2018). The darker red and lighter red shaded areas represent the 1st standard deviation and 95th% confidence range respectively. Flow Height not considered in this graph.**

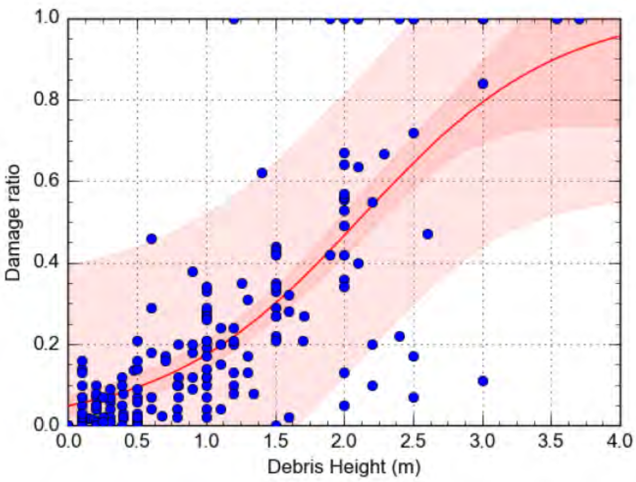


FIGURE 9.15 **Residential Building Damage Ratio v Debris Flow Height (Massey et al, 2018). The darker red and lighter red shaded areas represent the 1st standard deviation and 95th% confidence range respectively. Flow velocity is not considered in this graph.**

6.1.1 Intensity Index

Developed as a method of determining building damage from debris flows, Jakob et al (2012) define Intensity Index (IDF) for building damage, as follows:

$$I_{DF} = dv^2$$

Where d is the maximum expected flow depth
v is the maximum flow velocity.

This is a simple and useful measure for rapid failures (debris flows and debris avalanches). A number of other metrics have been developed (see Massey et. al. (2019) for more detail), however the intensity index is useful as it is very simple, relying on only two parameters. Intensity Indices in relation to building damage for rapid landslides are presented in Part 6, Table 6.5.

7 DEBRIS FLOW MITIGATION

Should the assessment of debris flow or flood hazard suggest that velocities and flow depths are sufficient to result in a level of risk that is unacceptable to the Project or to the element at risk (e.g. home or highway), then some form of mitigation that reduces the risk to acceptable levels will be required.

In simple terms, mitigation measures for debris flow and flood hazards can be broadly categorized into two groups: non-engineered and engineered, as illustrated in Figure 9.16. Non-engineered measures entail minimal direct engineering intervention as there is no effort to prevent, modify, or control the event (Van Dine, 1996). By way of example when a debris flow hazard is identified and evaluated, options such as area avoidance, application of land use regulations, public notification and education, or the establishment of a warning system are implemented.

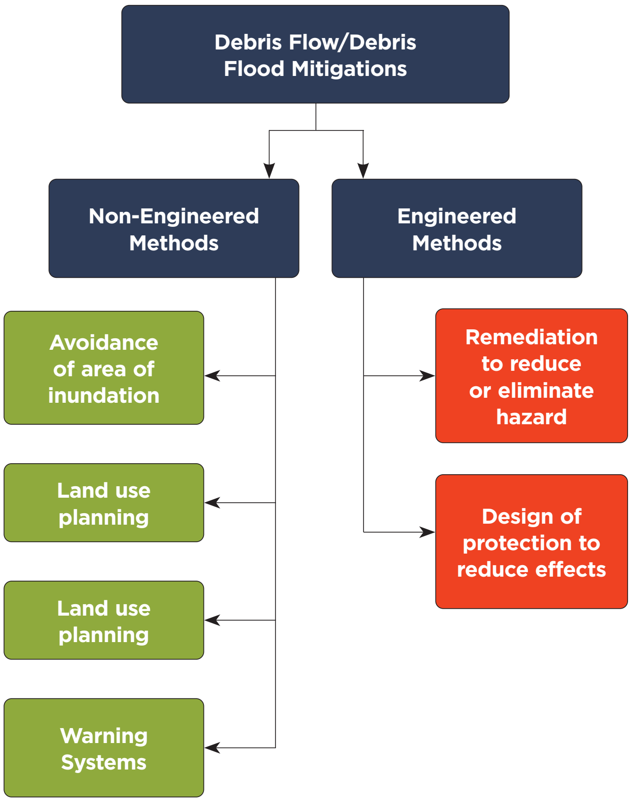


FIGURE 9.16 Mitigation Measures for Debris Flow/Floods (modified from Van Dine, 1996)

SCREENING CRITERIA FOR DEVELOPMENT IN DEBRIS RUNOUT ZONES: TAURANGA CITY COUNCIL'S APPROACH

Some Territorial Authorities (TAs) in NZ have specific screening criteria for development within debris runout zones, e.g. Tauranga City Council through their Infrastructure Development Code (IDC) Section 10.3 (e) requires:

"Nominated building platforms located within the 4H:1V downslope (debris) runout zone (shall be) protected by way of a debris protection measure where (specified) slope stability factors of safety have not been achieved." The IDC provides an explanatory note that these "4H: 1V runout zones are recognised as general failures zones based on the observation of numerous failures (and rigorous studies) within the Tauranga area. The 4H: 1V runout zone is the limit of debris runout zones for most failures involving sensitive (volcanic) ash soils."

The 4H: 1V runout zones are mapped on the Council online GIS system ('Mapi') so that geoprofessionals and Council consenting officers can use this as a screening tool for proposed developments and the need for protection measures.

On the other hand, engineered mitigation methods involve engineering interventions subsequent to hazard identification and assessment. These approaches may encompass remediation efforts aimed at reducing or eliminating the potential for a debris flow, or (most commonly) the design and construction of protective structures to reduce or control the impact of the event.

Mitigation options for debris flows are further outlined in Part 7 of Unit 1. The design of mitigations for Debris Flood and Flow hazards will be discussed in detail in proposed Unit 6.

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PART 10

MITIGATION AND DESIGN PRINCIPLES

Anchor Installation, Kaikoura Coast, January 2018. Photo courtesy NCTIR

PART 10 – MITIGATION AND DESIGN PRINCIPLES

1 INTRODUCTION

Many methods of slope stabilisation have been implemented effectively in the past. The appropriate solution for a given site depends on the particular set of project conditions and constraints. Selection and design of slope stabilisation measures should always begin with a sound understanding of the nature of the landslide (Part 2) Engineering Geological Model (Part 5), likelihood and consequences of failure (the level of risk; Part 6).

Once the Engineering Geological Model has been developed, and risks are understood, the traditional[1] design process for mitigating existing or potential instability typically involves the following steps:

1. Develop target performance criteria and objectives based on the project requirements, site specific conditions, regulatory requirements.
2. Identify possible mitigation measures that meet the functional requirements (design criteria, feasibility),
3. Consultation with stakeholders as required,
4. Undertake conceptual design of these measures.
5. Select the optimal solution by comparing features of the possible mitigation measures such as cost, technical effectiveness, impact on the environment (sustainability and circular design elements), aesthetics, etc.
6. Complete detailed design of agreed optimal solution
7. Implement selected option (construction and construction supervision).
8. Monitoring and maintenance.

This part of the guidance provides a preliminary overview of **Steps 2 to 4** of this process including mitigation measures and considerations to guide selection of mitigation.

2 CAUSES OF INSTABILITY

It is essential to understand the causes of instability to develop effective strategies for mitigating failed slopes or for anticipating changes in key properties and conditions that may affect new slopes (Duncan et al, 2014).

A slope failure occurs when the shear stress required for equilibrium is greater than the shear strength of the soil or rock. This condition can be reached either through a decrease in shear strength of the soil or rock or through an increase in shear stress. Important processes that contribute to slope instability include:

Changes in Water Condition

- Increased pore pressure/reduced effective stress – most often occurs due to intense rainfall, but other sources such as snowmelt, de-vegetation of slopes and increased irrigation increase water input and infiltration.
- Water pressure in tension cracks at the top of the slope, within the slide mass and/or within rock mass defects
- Increase soil weight from increased water content
- Drop in water level at the bottom of the slope (rapid drawdown)

Changes in Material Strength

- Weathering and cracking in rock – weathering affects rocks and indurated soils through a variety of physical (erosion, freeze-thaw), chemical, and biological processes (plant root wedging, burrowing animals), and can reduce shear strength and/or cause cracking.
- Clay behaviour, including:
  - Some NZ clays (in particular halloysite and smectites and other interlayered clays derived from volcanic deposits) incorporate water in their structures and swell. Swelling causes an increase in void ratio
  - development of low friction angles due to their plate like atomic structure,
  - creep under sustained loads,
  - leaching ("quick" clays)
  - loss of material strength in halloysite type clays in response to hydration or disturbance (EQ shaking or rainfall)
- Strain-softening – brittle soils are subject to strain softening making progressive failure possible. The peak strength cannot be counted on to mobilise simultaneously at all points on a shear surface.

[1] There are a number of alternative design pathways, which could include contractor led or contractor partnered design solutions etc

Changes in Load

- Vibrations and earthquake shaking – cyclic loading can increase pore pressure and break bonds between soil particles, resulting in strength loss. When sands and non-plastic silts are saturated, this can lead to liquefaction. Vibrations and earthquake shaking also load the slope increasing the shear stress.
- Increase in loads at the top of the slope (due to surcharge for example).
- Excavation or erosion at the toe of a slope

Slope failures are commonly the result of a combination of factors rather than a single cause. Often, the trigger of a landslide is just the final factor among others that have contributed to the failure (Sowers, 1979). This highlights the importance of considering several potential causes during the process of evaluating slope stability and designing mitigation.

WATER IN LANDSLIDES

"Groundwater is the most important single factor in triggering landslide events" **(Waltham, 1994). Water has destabilising effects on both the shear strength (increased pore pressure, changes in the properties of clayey soils, acceleration of weathering) and the shear stress (increasing soil weight, removal of soil at base of slope through erosion, etc.).**

Water and its contribution to instability should always be considered when assessing the potential causes of a particular slope failure. Provision of drainage for groundwater and surface water is generally the most successful slope stabilisation method (Committee of Ground Failure Hazards 1985). However, effective implementation of drainage mitigation involves a thorough understanding of the ground model and groundwater conditions.

For design of new slopes, the geoprofessional should pay particular attention to how seepage conditions and groundwater regimes could change over the lifetime of the slope. These changes could be due to natural causes such as rainfall events and snowmelt, or artificial ones such as de-vegetation, irrigation, or increased surface runoff due to development near the site.

3 TARGET PERFORMANCE CRITERIA

A critical component of any slope hazard mitigation strategy is establishing the required target performance criteria. Slope performance criteria define the levels of acceptable risk or service that a slope failure event needs to meet. Analysis may demonstrate that a slope meets the required level of performance or may reveal that mitigation measures are needed to reduce the risk. The appropriate choice of specific criteria can only be determined following a clear understanding of the desired outcomes, and knowledge of the local regulations. There are broadly two target performance criteria approaches:

- **Risk-Based** - For hazards assessed within a risk framework (typically rapid evacuative landslides such as rockfall, debris flows), the performance criterion is often defined as an acceptable or tolerable level of risk, either risk to life (fatalities/year), or property ($/year).
- **Factor of Safety (FoS)** – This is the ratio of the shear strength divided by the shear stress required for equilibrium of the slope. It is the answer to the question "By what factor could the shear strength of the soil be reduced before the slope would fail?" (Duncan et al 2014). Minimum values of FoS against slope failure are commonly used as performance criteria for soil slopes (and are discussed in Part 8).

3.1 RISK-BASED PERFORMANCE CRITERIA

Where a Risk Assessment for a slope hazard has been carried out, the derived risk is compared to a threshold of "Tolerable" and/or "Acceptable" Risk to life or property. Tolerable and Acceptable Risk are defined by AGS (2007) as follows:

Tolerable Risks: *those that society can live with so as to secure certain benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.*

It is the threshold of risk that is tolerated because the cost to reduce the risk outweighs the benefit of that reduction.

Acceptable Risks: *those which everyone affected is prepared to accept. Action to further reduce the risk is usually not required.*

Acceptable risks are around one order of magnitude lower than the tolerable risks (AGS 2007). AGS (2007) suggests that for most development in existing urban areas, criteria should be based on tolerable risk due to the trade-off between the risks, the benefits of development, and the cost of risk mitigation.

Defining tolerable risk is a complex, context-specific task and can vary across different regions, cultures, and hazard types. The process of determining tolerable risk is the responsibility of the regulator (AGS 2007). Guidance for regulators on setting threshold criteria is included in MBIE's draft Landslide Planning Guidance (MBIE, 2023).

Within New Zealand, quantitative life risk assessments are much more common than those for property, and tolerable thresholds for property risks are rarely published in literature. For these reasons the below discussion on risk criteria focusses on life risk.

3.1.1 Annual Individual Fatality Risk

Risk to life is generally expressed as Annual Individual Fatality Risk (AIFR) and is defined by AGS (2007) as the probability that the individual most at risk is killed in any one year.

At the time of writing, New Zealand does not have national guidelines for determining tolerable life-risk limits. In Australia, AGS (2007) has recommended tolerable AIFR limits of one in 10,000 (1×10^{-4}) for existing slopes and developments, and one in 100,000 (1×10^{-5}) for new slopes and developments. A 1×10^{-4} limit was widely adopted in slope hazard assessments on Christchurch's Port Hills following the 2010-2011 Canterbury Earthquakes. Other examples of AIFR criteria used in practice internationally and in New Zealand are included in MBIE's Draft Landslide Planning Guidance.

3.1.2 Societal Risk

Societal risk tolerance criteria are generally communicated through an F-N curve or risk tolerance chart (see, for example the MBIE (2023) Landslide Planning Guidance; Taig et al, 2011; AGS,2007 and Sim et al 2022). The F-N Curve compares the number of fatalities (N) for a landslide event with the probability (or frequency) of that event (F) (Sim et al. 2022). These charts reflect the idea that society has a lower tolerance for hazard events that result in multiple fatalities.

An F–N curve consists of a series of data points that denote each scenario analysed. It is a combination of scenarios into a single curve that defines the probability (or frequency) of "N or more" fatalities of the complete dataset (Strouth and McDougall 2021).

An often-cited F-N criteria is that developed for use in Hong Kong (Figure 10.1a) and an example of a chart with a single line of risk demarcation is included as Figure 10.1b. It is important to note that Societal risk tolerance varies widely across regions and hazards, and tolerability criteria are not always directly transferrable from one region to another (MBIE, 2023).

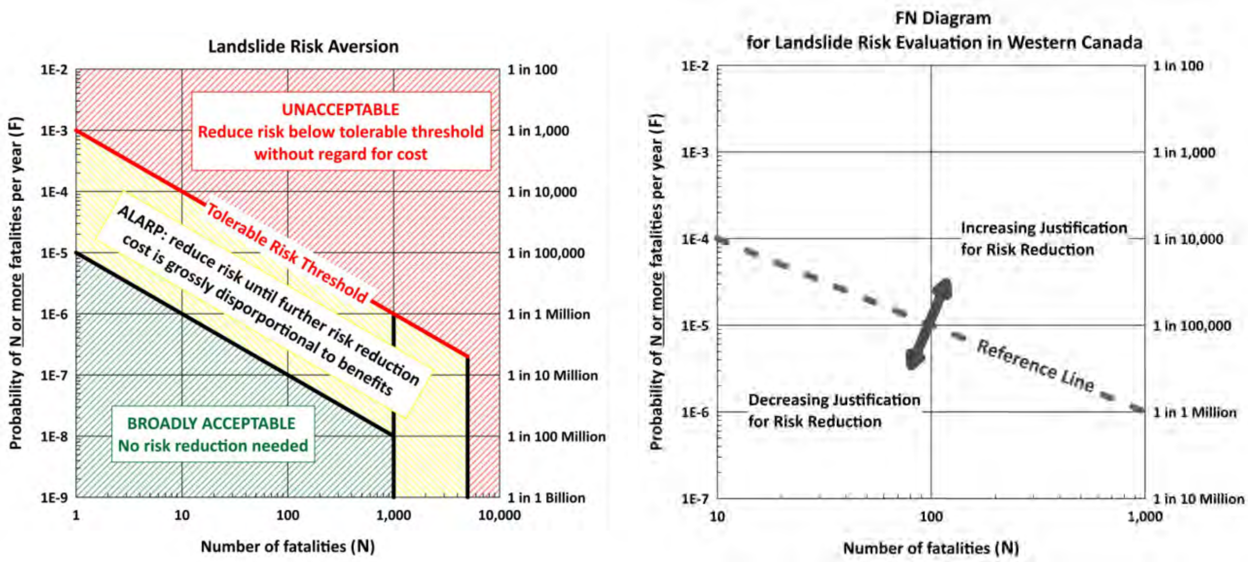

Figure 10.1a: A risk tolerance chart for societal risk developed for use in Hong Kong

Figure 10.1b: A proposed modified F - N chart for low population areas showing a single line of risk demarcation

FIGURE 10.1. Examples of Risk Tolerance Charts (from Strouth and McDougall, 2021)

MULTIPLE MOVEMENT TYPES

Experience from Kaikoura has highlighted the need to assess the potential for multiple movement types in a single triggered event. For most landslides, failure includes a continuum of processes. Failure can be initiated as a rockfall, then become a debris avalanche or flow as rocks break up on impact with the ground. Or these movement types could exist simultaneously. If there is sufficient uncertainty in the dominant movement type, the geoprofessional may need to assess these various modes separately to determine consequences of each and appropriate mitigation strategies that address multiple modes.

3.2 FACTOR OF SAFETY (FOS)

3.2.1 Commonly Used Values

Minimum factors of safety have traditionally been used in geotechnical engineering practice to define acceptable levels of safety for soil slopes.

Within New Zealand, some territorial authorities and Crown entities have provided guidance on minimum FoS values for use in specific locations or design of specific assets (e.g. dams). However, in many situations the adoption of the minimum acceptable FoS is left to the geoprofessional and consequently it is common practice for "typical" values to be used.

It is important to note that the documents which present these criteria also generally provide advice on the specific conditions of applicability, level of investigation, and the level of conservatism in selection of soil properties. Use of 'typical' values of FoS without regard for the uncertainty and level of conservatism in the input parameters (soil material strengths and groundwater table) and consequences of failure is generally inappropriate.

The choice of an appropriate target FoS depends on several considerations including the quality of the data and interpretation, and construction quality as well as the likely consequence of failure. The geoprofessional is responsible for ensuring that the FoS is appropriate. Further discussion on developing appropriate target FoS within a risk framework will be included in proposed Unit 4.

Commonly used FoS criteria for various loading conditions are discussed in Table 7.1 in Part 7. Methods of accounting for and incorporating uncertainty are discussed in Section 4.2.2.

3.2.2 Earthquake Load Case - Displacement Limits

Target limits on slope displacement are often applied in the earthquake load case, where resisting all slope movement is impractical within normally accepted levels of mitigation expenditure. In these cases, a FoS less than one under seismic loading may be acceptable provided deformations are within acceptable limits. These limits are defined taking into consideration the required seismic performance of the slope and affected structures. Estimations of displacement are discussed in more detail in Part 8.

3.2.3 Factor of Safety and Probability of Failure

The calculated FoS of a slope is not the same as its probability of failure because the distribution of FoS depends on the uncertainties of the inputs. As an example, it is possible that a slope with a calculated FoS of 2.0 based on poorly understood parameters has a higher probability of failure than a slope with a calculated FoS of 1.5 where the parameters are better understood (Figure 10.2)

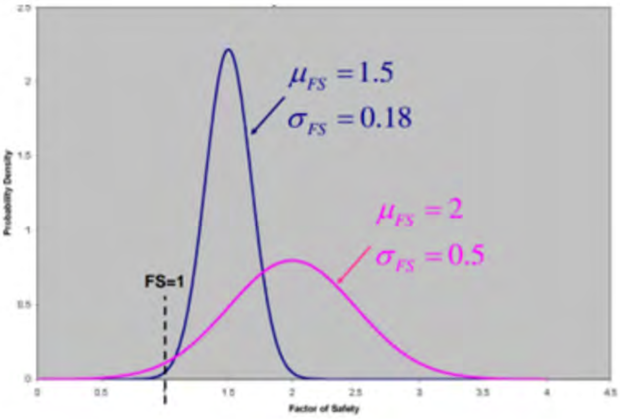

FIGURE 10.2: Two Distributions of FoS (Prof. Griffiths, Colorado School of Mines).

Silva et al (2008) developed a relationship between FoS and probability of failure for several categories of projects which is shown in Figure 10.3. This chart can be used to get an initial approximation of probability of failure from FoS. The projects are categorised according to their level of engineering (intensity of investigation, analysis, documentation, and construction) reflecting the uncertainty in performance, with projects of a high level of engineering having a lower level of uncertainty and vice versa.

Where:
Category I are facilities designed, built, and operated with state-of-the-practice engineering. Typically, these facilities have high failure consequences.
Category II are facilities designed, built, and operated using standard engineering practice. Many ordinary facilities fall into this category.
Category III are facilities without site-specific design (but with generic design) and/or with sub-standard construction or operation. Temporary facilities and those with low failure consequences often fall into this category.
Category IV are facilities with little or no engineering.

Probabilistic slope stability analyses allow for a site-specific determination of probability of slope failure (P_f). The probability of failure can then be used within the risk-framework to determine the risk and compared with risk-based performance criteria. Probabilistic slope stability methods are discussed further in Part 8.

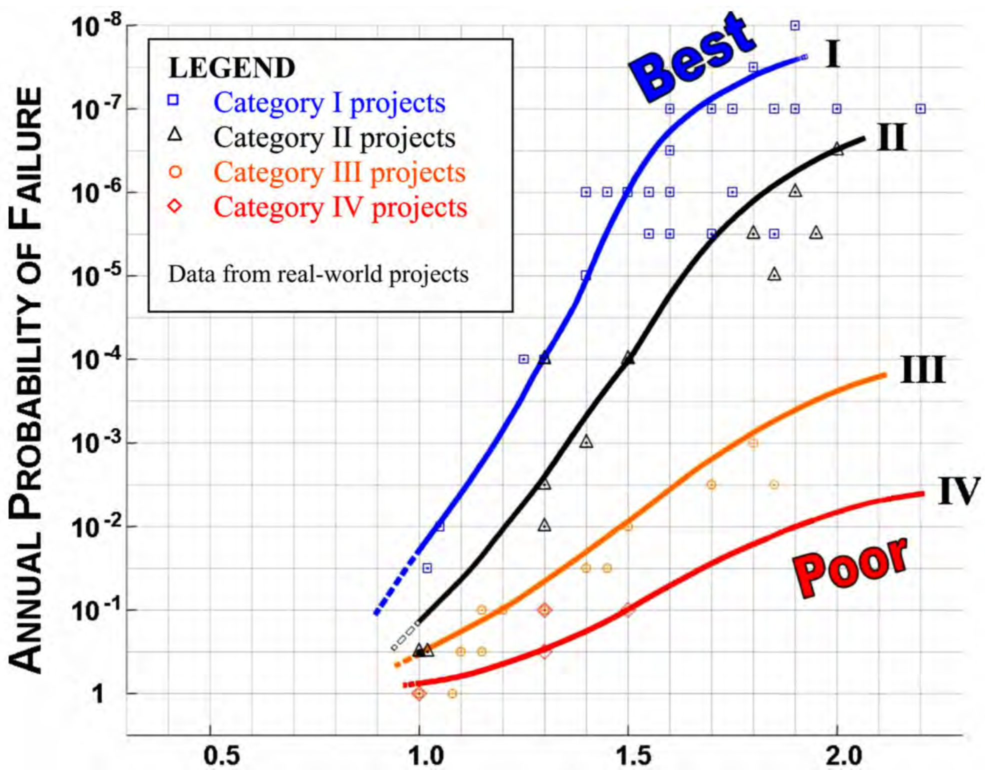

FIGURE 10.3: Factor of Safety versus annual probability of failure (taken from Silva et al 2008)

4 CONSIDERATIONS IN SELECTING MITIGATION STRATEGIES

A FUNDAMENTAL

The ground model and required performance criteria form the basis for selecting technically suitable mitigation strategies. These are the solutions that can achieve the required performance for the site-specific conditions. Of solutions that are technically suitable, careful consideration of various factors is required to determine an appropriate or optimal solution. The optimal solution will be unique to the project, incorporating project-specific constraints and features of importance to the stakeholders.

In all cases early involvement of stakeholders and contractors is wise. This will define project constraints, highlight considerations of importance, and provide an early indication of risks to the project timeline and cost.

Key considerations when selecting a preferred mitigation include:

- **Safety** - Slope instability and its mitigation are inherently hazardous. The consideration of safety is of utmost importance throughout the design and construction and operation phases. This is discussed in more detail in Section 7.

- **Legal and regulatory requirements, property ownership:** Requirements by the relevant territorial authority, regional council, or Crown entity (e.g. Waka Kotahi) may dictate the available mitigation strategies. Works near or within protected land and waterways (or needing to traverse these areas for access) are likely to be limited to those that have minimal impact on, or result in an improvement to, the environment. Neighbouring landowners may not permit works (such as ground anchors) extending onto their land. These issues may constrain the solutions available and should be identified early in the project.

  Early stakeholder input and involvement is important to ensure that the selected mitigation strategies are socially and legally acceptable. Key stakeholders could include impacted landowners, local Iwi, the relevant consenting authority (often the local Council), infrastructure owner and others impacted by or with an interest in, the construction.

- **Tikanga ā-iwi Values:** Where mitigation solutions may impact access to the sea and foreshore, careful consideration should be given as they may impact customary use rights. In particular, mitigation options that seek to avoid a landslide hazard (transport corridor realignment for example) can restrict access and will require consultation with local Iwi.

- **Cost:** The costs and benefits of each mitigation strategy should be evaluated, considering the potential reduction in risk, and the financial resources available. Ideally the financial benefits of the risk reduction should exceed the costs of achieving the reduction. This will be of key importance to territorial authorities and other asset owners who likely need to prioritise the use of their financial resources. While cost needs to be considered, it should not allow inadequate or inappropriate design: design of a temporary or lesser design life solution might be suitable.

- **Time Frames:** How much time is available? If time frame is critical, solutions that can be quickly or progressively implemented will be preferred. If timeframes are more generous, there may be time to optimise solutions, or select solutions that can only be constructed during certain seasons. There may be long lead times on equipment, or the manufacturing of elements required for some solutions (e.g. rockfall protection structures). Early contractor and manufacturer involvement can help identify these constraints.

- **Access/Constructability:** Can the site be safely and legally accessed, and by what types of construction equipment? If the potential solution requires maintenance, can that maintenance be carried out in accordance with HSWA requirements? Has maintenance been considered as part of Safety by Design (Refer Section 7)? The ability to safely access a site for construction or maintenance of a mitigation solution may limit the available solutions. In some situations, a phased approach is required by which an initial level of stabilisation is undertaken with the aim of providing an appropriate level of safety for construction of the main stabilisation works. Careful consideration is required to ensure that the risk of carrying out the initial works is not greater than the risk that work is attempting to manage.

- **Available Resources and Technical Expertise:** The equipment, materials, and technical expertise required to design and construct a particular mitigation strategy must be available for the location and within the timeframes of the stabilisation project.

- **Durability and Sustainability of materials:** The sustainability of the mitigation strategy should be considered, including, its ability to withstand changing environmental and stability conditions, durability of its elements, material sources for its construction and its adaptability over time.

- **Environmental and social impact:** Physical impacts on the natural environment, cultural heritage, and local communities should be assessed. Consultation with Iwi will need to be undertaken for some projects. The need for assessment of the carbon footprint of the works is likely to become more widespread, with increasing importance placed on selecting those solutions with lower environmental impact.

- **Maintenance:** Maintenance requirements may vary widely between mitigation options. Long term maintenance requirements and associated cost should be adequately considered during solution selection and design. Over the course of the NCTIR project several novel solutions and modifications to proprietary systems were developed to make maintenance easier and cheaper. Some examples include a Self-Cleaning Canopy (SCC), Timber Debris Interception (TDX) Wall, and Machine Access Debris Flow Bridges (Revell et al, 2021).

- **Technical Effectiveness:** Of the solutions that meet the target performance criteria, and are feasible, there will be a range in the level of risk reduction that each can achieve. The geoprofessional is responsible for communicating these differences in risk reduction to the owner.

- **Aesthetics:** Some mitigation measures will have a much larger visual impact on the landscape than others, and the sensitivity to these impacts will vary project by project. The degree to which aesthetic impacts need to be considered should be defined early in the project.

Evaluation of the trade-offs between different mitigation strategies is often required.

5 MITIGATION STRATEGIES

This section provides an overview of a range of strategies to consider for reducing the risk associated with slope failure. Methods can be broadly divided into two categories: engineered measures and non-engineered measures.

- **Engineered strategies** are those that aim to (1) reduce the probability that an instability event is triggered (stabilising measures), or (2) reduce the consequences of the event (protection measures). Those measures that target the source of instability are categorised by the slope material type (soil or rock), and those that that target the consequence of failure are categorised by movement type (debris flow, rockfall).

- **Non-engineered measures** are those that do not attempt to directly prevent, modify, or control an instability. These measures reduce risk by reducing the consequence of the failure and include avoidance, land use planning and regulation, vegetation planting, monitoring and warning systems, and education. We are using the term "non-engineered measures" to be consistent with existing terminology, but it can be a misnomer - some types of avoidance measures, such as tunnelling or bridging, involve substantial engineering design but bypass the problem and do not attempt to reduce the probability of an event.

Commonly used mitigation strategies are outlined in Tables 10.3 to 10.5 and illustrated in Figure 10.4, but many more strategies exist. More comprehensive discussion of mitigation measures can be found in Turner and Schuster (1996), NZGS (2016) Rockfall Passive Protection Design Guideline, Turner and Schuster (2012) and VanDine (1996) for debris flow hazards.

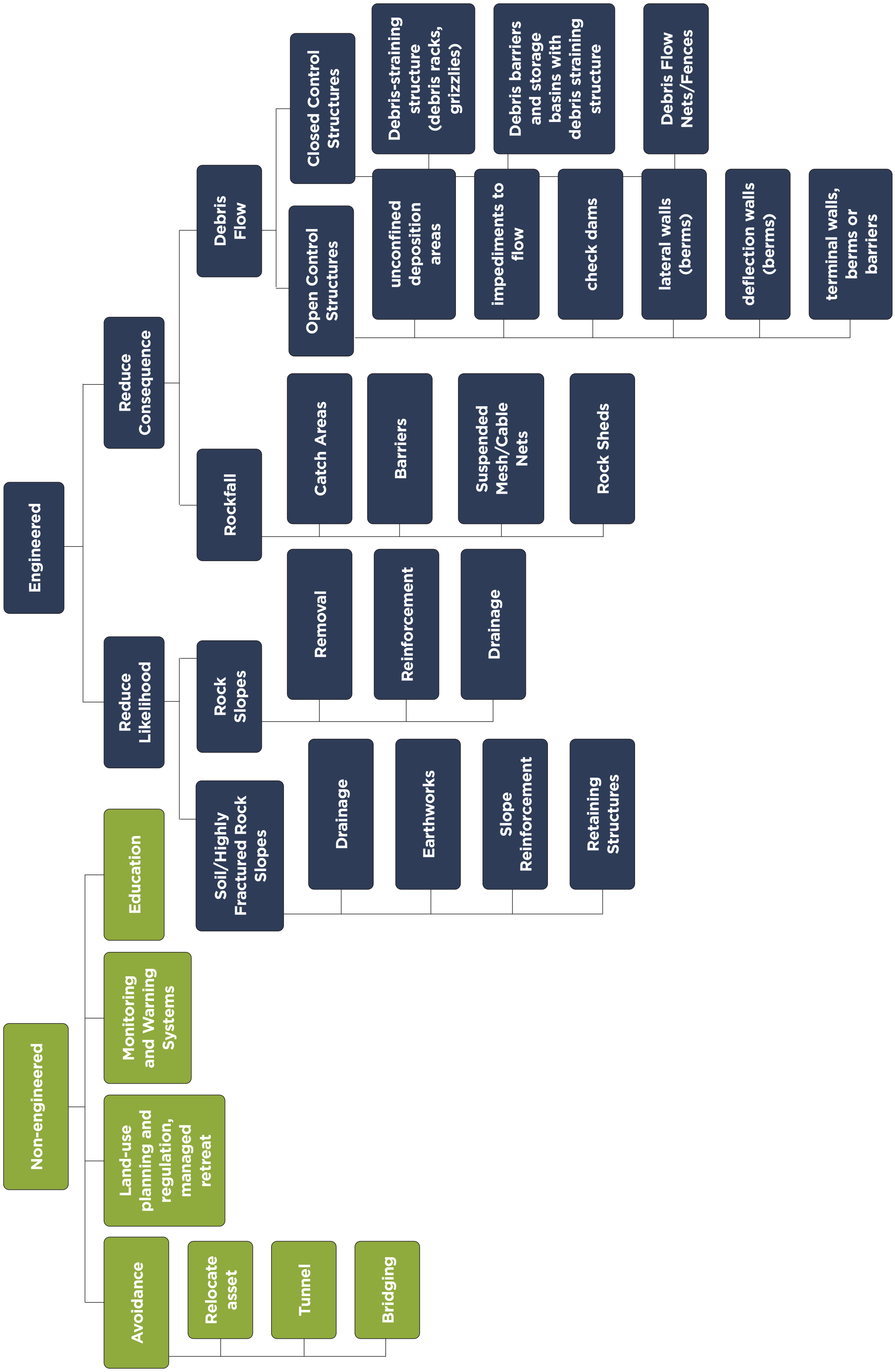

FIGURE 10.4: Summary of mitigation types.

TABLE 10.3: Summary of Non-Engineered Mitigation Options

CATEGORY OF MITIGATION MEASURE	DESCRIPTION/PURPOSE	TYPE OF MITIGATION MEASURE	ADVANTAGES/ WHERE USED	LIMITATIONS
Quick Risk Reduction Options	Partial or complete removal of source rock to reduce the occurrence of rockfall for immediate safety improvements	Scaling Sluicing	Useful in areas with limited source area. Relatively inexpensive unless prolonged helicopter time for sluicing is required. Proximity to, or availability of water supply.	Provides short term benefit (but difficult to quantify). Unlikely to provide a permanent solution. If not successful, sluicing may *increase* likelihood of failure occurring. **Safety of roped access workers needs to be carefully evaluated and controlled.**
Avoidance or Retreat	This solution involves relocating the asset or bypassing the potential instability. It is a good approach if considered early, as it removes the risk associated with the instability.	Relocate the Asset	Can be an alternative anywhere. Most often employed for new developments or infrastructure	Often not viable due to lack of an alternative location, cost to relocate, and/or need to maintain access. Need to ensure conditions at new location/alignment are an Improvement.
		Bridging – bridge/ viaduct constructed to span or locate adjacent to an instability or active rockfall or debris flow runout area.	Typically used for sidehill linear infrastructure such as a road where the terrain is relatively steep, and/or where space is available	Costly; Loading on bridge piles due to landslide movement are difficult to predict and can result in unviable designs.
		Tunnel located underneath the basal shear surface of the landslide	Usually only considered where the risk is significant, the infrastructure is critical, and other measures are inadequate or not possible, i.e., where the benefits offset the high cost.	Costly; may be introducing new hazards associated with confined space.
Land-Use Planning	Can be used by regulators and developers (on advice of geoprofessionals) to limit development in high-risk areas (zoning and land use restrictions) or provide specific guidelines for development in these areas. The Draft National Landslide Planning Guidance (MBIE) provides advice for regulators on land-use planning to address slope hazards.	Land-Use Planning	Minimising potential economic loss caused by landslides (recovery, reconstruction efforts), allows for long term sustainable developments and resilient infrastructure.	Use is often limited to relevant authorities.
Monitoring and Early Warning Systems	Aim is to gather information used to avoid or reduce the impact of slope movement. The key is to identify and measure small but significant factors that precede a landslide or identify when a failure is imminent or has occurred and allow measures to be implemented to prevent a harmful consequence.	Visual inspections (in person or remote cameras), survey (manual through to total station), telemetry (of inclinometer, crack meter, tilt meter, groundwater levels, rainfall etc.), remote sensing	Can be very cost effective when compared with engineered stabilisation solutions. Allows expenditure on a mitigation to be deferred until absolutely needed. Useful in remote locations or where cost of stabilisation is not yet justified.	False alarms may result in conservative responses such as evacuation, partial or full road closure etc.); development of effective threshold values may be difficult if mechanism of landsliding not well understood; warnings may not allow enough time for consequence reducing measures (e.g. evacuation). Does not decrease the risk to property
Education	These methods aim to educate those potentially affected by a instability to allow for consequence reducing measures. These measures are often employed alongside other mitigation measures.	Warning signs communicating risk (typically for rockfall hazards) Public preparedness (understanding landslide triggers; escape routes) Emergency management.	Can be cost effective when compared with engineered stabilisation solutions. Useful in combination with monitoring and early warning in remote locations or where cost of stabilisation is not justified.	It is difficult to quantify the impacts of these measures. Can require ongoing effort over a period of time. These measures tend to target life-risk but usually do not reduce risk to property.

TABLE 10.4: Summary of Engineered Mitigation–- Stabilisation Measures (reduce likelihood of failure)

SLOPE MATERIAL TYPE	CATEGORY OF MITIGATION MEASURE	DESCRIPTION/ PURPOSE	TYPE OF MITIGATION MEASURE	DESCRIPTION/ ADVANTAGES/WHERE USED	LIMITATIONS
Soil and Highly Fractured Rock	Drainage	Diversion of surface water away from the location of the landslide; Removal of or provision of a managed egress for groundwater from the landslide	Surface Drainage	Surface drainage measures can require relatively minimal engineering design and may have relatively low cost. Provision of surface drainage is strongly recommended as part of the treatment of most landslides or potential landslides (Cedergren 1989). Slope vegetation can aid in reducing infiltration of surface water.	Must be maintained to keep functional. May need to be combined with other methods to achieve the required performance
			Subsurface Drainage - Horizontal Drains - Pumping Wells and Stone Columns - Drainage Galleries - Trench Drains	Lowers groundwater level/s in the landslide, decreasing pore pressure, leading to increased shear strength of soil. Often less expensive than other mitigation measures.	Reduced effectiveness where slide mass has very low permeability. Maintenance required for many types of subsurface drainage. Designers must have a reasonable understanding of the ground model, groundwater conditions, and the causes of landslide movement, otherwise money spent on drainage may be misdirected or the benefits not optimised
	Excavation and Earthworks	Involve changing the geometry of the slope or replacing weak soil with stronger soil. Some strategies reduce the forces driving an instability (removing load at head of slope, regrading), while others provide stabilising forces (buttress or shear key).	Unloading, benching, Buttress, shear key, gravity berm.	Buttress fills are constructed in front of a slope or at the base of the slope and can be constructed of high strength soils providing weight and strength. They can be combined with a shear key (a trench at the base of the slope where in-situ soil is replaced with high strength compacted soil). A gravity berm is constructed of uncompacted fills that provides weight as a stabilising force.	May not be effective in the case of deep-seated landslides Requires a firm foundation, Shear key requires space at the base of the slope and careful analysis and consideration of stability and safety during construction as excavation at base of slope removes some stabilising force temporarily.
			Regrading/Changing Slope Line or removing load at the head of a slope	Can be a cost-effective solution	Results in loss of usable land at the top and/ or bottom of the slope. Need a disposal location for removed soil. Careful thought around sequencing of works to maintain safety through construction
			Excavate and Replace	Typically used on smaller landslides with a shallow depth of poor soils, and where the soils need to be replaced to regain useable area. Incorporates other methods such as changing the slope line, drainage, and shear key.	Need a place to dispose of material. Often not suitable for deep seated landslides due to volume of material; need to ensure slope is stable enough to provide safe access for excavation

TABLE 10.4: Summary of Engineered Mitigation-- Stabilisation Measures (reduce likelihood of failure) (continued)

SLOPE MATERIAL TYPE	CATEGORY OF MITIGATION MEASURE	DESCRIPTION/ PURPOSE	TYPE OF MITIGATION MEASURE	DESCRIPTION/ ADVANTAGES/WHERE USED	LIMITATIONS
Soil and Highly Fractured Rock	Retaining Structures	Retaining structures impose a stabilising force to slopes (anchored walls, gravity walls) and in some cases increase the shear strength along a potential failure surface (soil nails, MSE walls).	Anchors and Anchored walls	Does not require slope movement before imposing restraining forces.	Requires competent soil or rock into which to extend anchors. Can be costly.
			Gravity walls, Mechanically Stabilised Earth (MSE) walls, Soil Nails and Soil Nail Walls	These solutions are relatively flexible and versatile, able to be adapted to many situations. Often used to improve the stability of new slopes where space constraints require steep slopes.	Movement must occur to develop full resistance, however, this may be negligible. Can be costly.
	Reinforcing Piles and Drilled Shafts	Slope stability can be improved by installing piles or drilled shafts through a sliding mass and into stable soil/ rock below the slide plane.	Drilled shafts are preferred over driven piles as they have a smaller adverse effect on stability (Duncan 2014) and driven piles may not be able to penetrate into dense soils at depth. (Secant/tangent pile walls are examples of drilled shaft retaining walls)	For stabilisation of an existing slope, drilled shafts are typically installed in one or more lines parallel to the crest of the slope, usually spaced 2-4D near the centre of the sliding mass (Duncan 2014). Can be used over a range of soil/rock conditions. Can stabilise deeper seated slides. Cause less impact to the slope and surrounding area than some methods	Movement must occur to develop full resistance, however, this may be negligible. More costly than other methods.
	Vegetation (often combined with manufactured elements)	Vegetating slopes improves stability against shallow sliding by protecting against erosion, root reinforcement and reducing pore pressures through rainfall interception and evapotranspiration (Wu 1994)		Useful for most stabilisation projects in conjunction with other methods. Cost effective. Sustainable Provides a natural aesthetic	Requires time for plant/root growth May require maintenance (watering, etc.) Often needs to be used with other mitigation solutions as does not improve deep instability. Can be challenging to establish and vulnerable to external factors (weather, animal activity, people not realising vegetation is a stabilisation measure and removing, etc.).

TABLE 10.4: Summary of Engineered Mitigation-- Stabilisation Measures (reduce likelihood of failure) (continued)

SLOPE MATERIAL TYPE	CATEGORY OF MITIGATION MEASURE	DESCRIPTION/ PURPOSE	TYPE OF MITIGATION MEASURE	DESCRIPTION/ ADVANTAGES/WHERE USED	LIMITATIONS
Rock[1]	Removal	Partial or complete removal of source rock to reduce the occurrence of rockfall. May include modification of the slope profile to remove features that act as launch points for falling rock.	Scaling (hand tools, air bags, light blasting, water blasting, sluicing, excavator)	Useful in areas with limited source area. Relatively inexpensive.	Effective for short-term, but less effective as long-term solution. May need regular scaling programme or additional measures to maintain desired level of protection. May expose further susceptible rock face.
			Blasting (large scale), Slope reshaping.	Can result in a more stable rock face with reduced maintenance costs.	Potential for damage from flying rock and/or debris. Possible right of-way and environmental issues. Debris consisting of smaller rock blocks may pose new risk, particularly for short term.
	Reinforcement	Secure source rock in place to reduce the occurrence of rockfall	Dowels, Shear Pins (untensioned), Rock Bolts (tensioned).	Can be used for individual blocks (spot bolting) or for rock masses (pattern bolting).	Slope access difficulties, Effectiveness affected by block size.
			Shotcrete.	Useful for small block size, erosion protection.	Reduces slope drainage. May be affected by freeze-thaw conditions or by earthquake shaking. Visual impacts. Quality and durability affected by skill in application.
			Buttress	Typically for support of key block.	Height limitations. Slope access difficulties. Visual impacts.
			Cable lashing, Walers, Lagging.	Useful for individual blocks	Typically, movement must occur to develop full resistance. Visual impacts.
			Anchored mesh, Cable nets.	Useful over large areas for a range of block sizes.	May accumulate rockfall debris. May change loading conditions if significant debris accumulates.
	Drainage	Removal or reduction of surface water and/ or groundwater to reduce the occurrence of rockfall. Commonly used with other mitigation techniques.	Surface Drainage	Used where surface flows affect rock face stability	Slope access and layout difficulties. Requires regular maintenance. Environmental issues
			Subsurface Drainage – Weep drains	Used in areas where groundwater affects rock face stability.	Difficult to quantify the need and verify effectiveness. Requires regular maintenance.

[1] Rock measures adapted from NZGS (2016)

TABLE 10.5: Summary of Engineered Mitigation - Protection Measures (reduce consequence of failure)

FAILURE MOVEMENT TYPE	CATEGORY OF MITIGATION MEASURE	DESCRIPTION/ PURPOSE	TYPE OF MITIGATION MEASURE	ADVANTAGES/WHERE USED	LIMITATIONS
Rockfall[1]	Catch Areas	A shaped catch area, usually constructed at the base of a slope, that is used to contain rockfall.	Ditch/berm.	Often used along transportation corridors. Can retain large volumes. May be combined with other types of structures	Right-of-way limitations. Large area may be required for high slopes. Requires regular clean-out and maintenance to preserve effectiveness. Material can roll through, especially in over-design events.
	Barriers	Wall-type structure used to intercept and contain falling rock.	Rigid Barrier – stiff materials (concrete, timber).	Used for relatively lower energy impacts; can have small footprint area.	Stiff materials are more prone to damage by higher energy events; typically not useful for high energy impacts
			Rigid Barrier – deformable materials (earth embankment, mechanically stabilised earth wall, gabion wall)	Capable of sustaining multiple high energy impacts, depending on construction. Design life is less affected by aggressive (corrosive) environment. Facing can be adapted for aesthetic requirements.	Construction must be on relatively flat slopes (< about 20 deg). Can require relatively large footprint area. Requires regular inspections and clean-out. May need to consider slope stability and surface drainage issues depending on location.
			Flexible barrier (rockfall fence).	Can be installed in difficult-to-access locations; has relatively low mass. Can be installed quickly.	Require space for downward deflection. Requires regular inspections and maintenance, including clean-out. Clean-out can be difficult if installed in difficult-to-access location. Possible issue if multiple impacts per event are anticipated. Design life more affected in corrosive environmental conditions.
	Suspended Mesh/ Cable Nets	Flexible wire or cable mesh structure suspended across a chute or over a rock face. Used to intercept rocks, attenuate their energy, and direct them into a catch area.	Hybrid drapery, Attenuator.	Useful for high rockfall frequency and where debris can be guided into a collection area. Required maintenance isa less than for flexible barriers. Can be installed in difficult-to-access locations.	Usually requires a debris collection area. Must consider debris and snow loads on anchors. Generally limited to rock sizes less than about 1.2m, depending on mesh type. Design life more affected in corrosive environmental conditions.
	Rockfall Canopy	A rockfall canopy consists of steel netting secured over a corridor		Used where transportation corridors sidle steep slopes. An alternative to rock sheds.	Can be expensive. Limited to steep sided slopes.
RockFall and Debris Flow	Rock Sheds	Covered, usually concrete, structure used to intercept and divert rockfall. Typically, only used for transportation routes.	Rockfall Shed.	Used in steep-sided valleys with high rockfall frequencies. Can divert water flow and small debris flows. Relatively low maintenance. Typically used only for transportation routes.	Must consider effects below the protected corridor. Expensive.

Table 10.5: Summary of Engineered Mitigation - Protection Measures (reduce consequence of failure) (continued.)

FAILURE MOVEMENT TYPE	CATEGORY OF MITIGATION MEASURE	DESCRIPTION/ PURPOSE	TYPE OF MITIGATION MEASURE	ADVANTAGES/WHERE USED	LIMITATIONS
Debris Flows[2]	Open Control Structures	Designed to constrain or control the flow of a channelised debris flow.	Unconfined deposition areas.	These are areas on a debris fan designed and prepared to spread out channelised flow allowing debris to settle out and accumulate. Can be combined with a flow impeding structure or terminal barrier. This method is best suited to large fans with low gradients and few to no artificial structures or developments. Cost effective.	Requires space. Requires maintenance / clean-out following an event.
			Impediments to flow (Baffles) –	These are structures or barriers placed strategically within the debris flow path used to slow down debris flow and encourage deposition.	Requires space. Often these structures are designed as sacrificial and require replacement following an event. Requires considerations to ensure baffles do not add to the debris flow mass.
			Check dams – a series of structures built across a channel to reduce gradients and minimise scour of the channel	Used in the transport zone of the debris flow Less space requirements as solution aims to mitigate the flow before it reaches the debris fan	May require maintenance / clean-out following an event. Requires significant engineering design. Expensive.
			Lateral walls or berms/ Deflection walls or berms –	Berms or walls constructed parallel to the debris flow path to constrain or control the lateral movement of the debris flow. Lateral berms aim to constrain the flow to a straight path while deflection berms deflect the flow to another part of the fan to protect a structure or decrease the gradient encouraging deposition. Usually used to protect an area or structure on the debris fan.	Lateral berms are typically not designed to encourage deposition, so other measures may be required to protect structures further down the flow path. If deposition does occur, debris must be removed. Consideration must be given to erosion protection and armouring the front face.
			Terminal walls, berms, or barriers	These are constructed across the path of debris flow to obstruct the flow and encourage deposition by increasing the width of flow path. Often built with a deposition area upstream.	Requires maintenance / clean-out following an event. Located as far as possible downstream from the apex of the fan so space on the fan is required.
	Closed Control Structures	These structures are designed primarily to contain channelised debris flow	Debris straining structures and barriers (debris flow fences/nets, slot barriers, debris racks)	These are barrier structures that separate the coarse debris from the fine debris and water encouraging the coarse debris to be deposited. There are many types and can be constructed of a wide variety of materials. Often used to prevent culvert openings and bridge clearances from becoming blocked (VanDine, 1996)	Requires maintenance / clean-out following an event. Often the costliest option.

[1]Rockfall measures adapted from MBIE (2016)
[2]Mitigation options for debris flow have been adapted from VanDine (1996). The options presented are limited to those implemented within the watershed channel or debris flow fan.

6 MITIGATION DESIGN

A broad framework for the mitigation design process is outlined in Figure 10.5 below. The specific design process for a particular project can vary widely depending on the nature of the instability (rockfall versus soil slope instability) and the mitigation method selected.

The assessment methods used to analyse the hazard (e.g. rockfall software, slope stability software) can usually also incorporate elements of mitigation, and hence can be used to design the mitigation. Mitigations that include structural elements will require design of those elements in accordance with relevant standards and guidance.

It is not practical to cover details of design for all scenarios here, but many resources are available to aid in design of specific mitigations. Some additional resources are listed in Table 10.7.

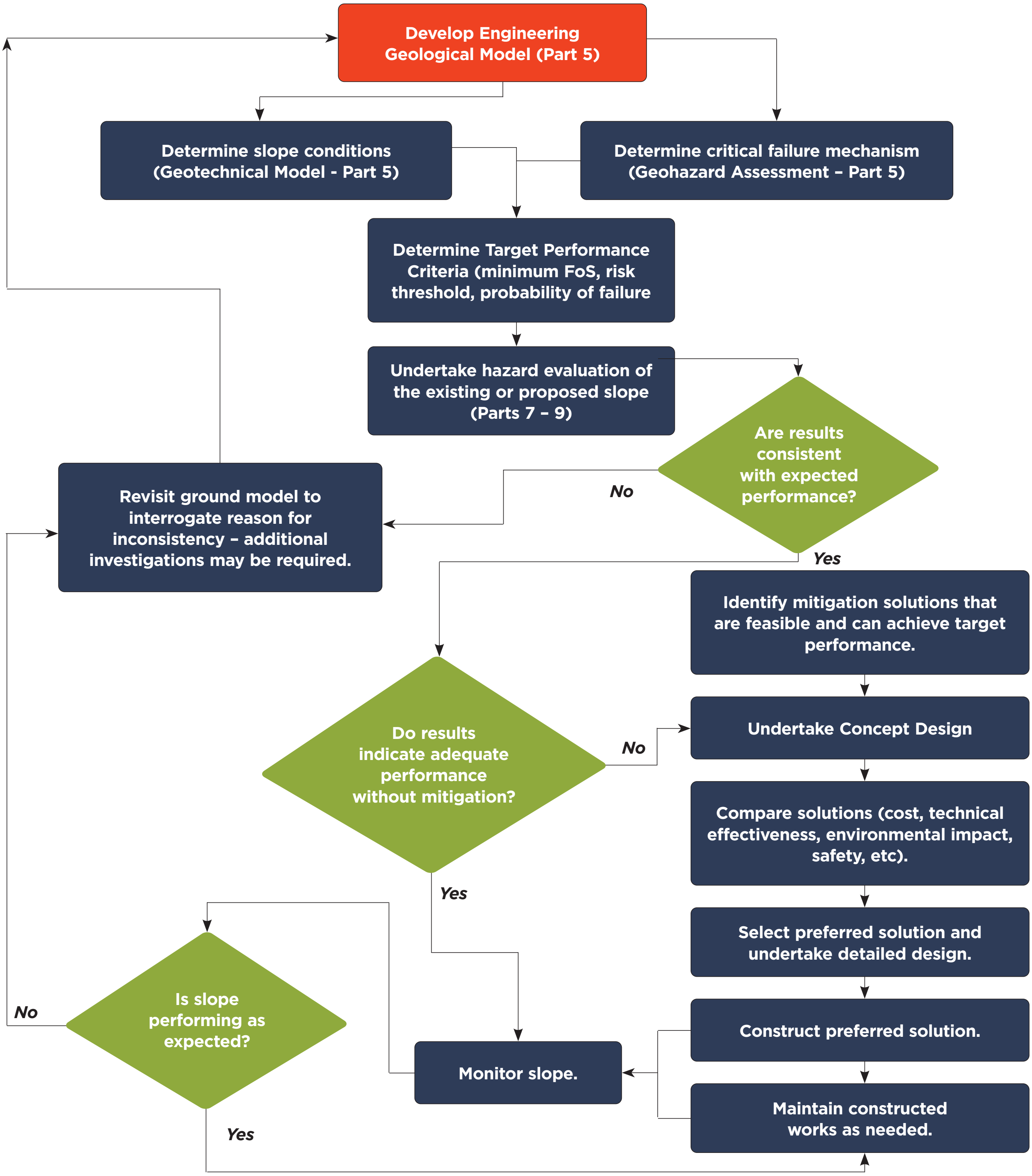

FIGURE 10.5: General Overview of Slope Mitigation Design Process

TABLE 10.7: References for Design of Mitigation Measures

MITIGATION TYPE	RESOURCES
Passive Rockfall Protection Structures	MBIE (2016). Rockfall: Design Considerations for Passive Protection Structures.
Debris Flow Fences/Nets	Berger et al (2021). Practical guide for debris flow and hillslope debris flow protection nets. Moase (2017), Guidance for debris-flow and debris-flood mitigation design in Canada
Anchored Walls/ Rock Bolts and Anchors	NZGS (2023). Ground Anchors: Design and Construction Guideline. DRAFT for Comment. FHWA (1999) Geotechnical Engineering Circular No. 4: Ground Anchors and Anchored Systems. MBIE (2021). Module 6. Earthquake resistant retaining wall design
Soil Nails	FHWA (2015). Soil Nail Walls Reference Manual
MSE Walls	FHWA (2009) Geotechnical Engineering Circular No. 11: Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines.
Drilled Shaft	Duncan (2014). Soil Strength and Slope Stability
Retaining Walls (general)	MBIE (2021). Module 6. Earthquake resistant retaining wall design

A more detailed unit on mitigation design is planned (Unit 3) and will include design examples.

7 SAFETY BY DESIGN

Landslides pose an inherent risk to safety, which should be a primary consideration through all phases of a mitigation project, from site assessment and design to construction and operation. Under the Health and Safety at Work Act 2015 (HSWA), everyone (including owners, designers, and contractors) has an obligation to manage risk, consult and coordinate to improve safety.

The inclusion of safety considerations in the design process plays a crucial role in fulfilling the requirements outlined in the HSWA. The primary objective of Safety by Design (SbD) is to incorporate risk identification and assessment methodologies during the early stages of the design process. The aim is to eliminate or, if not feasible, minimize potential health and safety risks associated with the construction, operation and maintenance of the mitigation strategy over its design life. Figure 10.6 illustrates the importance of integrating safety considerations early in the life of a project.

Worksafe New Zealand has produced good practice guidelines on Health and Safety by Design. These guidelines outline the role of designers in managing health and safety risks, key principles that designers should follow, and specific considerations during design. This document should be referred to for further information on implementing Safety in Design.

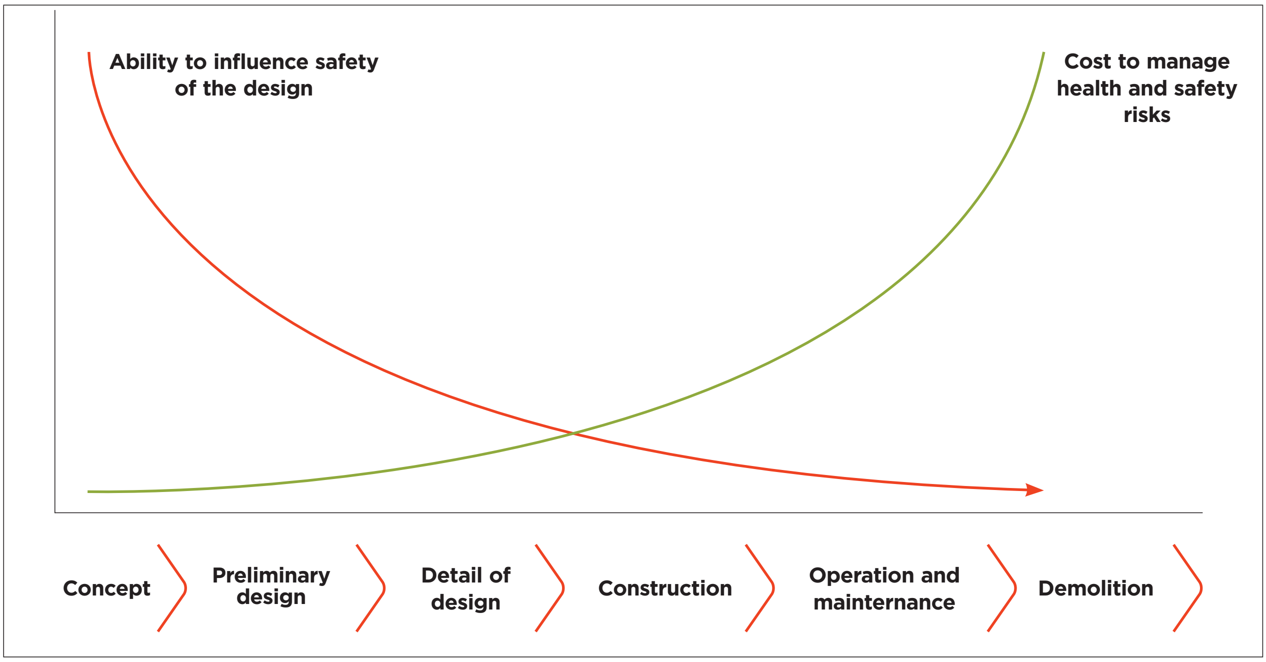

FIGURE 10.6: Symberszki chart of influence over a products' lifecycle (reproduced from Worksafe NZ Health and Safety by Design)

KEY H&S LEARNINGS FROM THE NCTIR PROJECT (REVELL ET AL 2021)

- **From a SbD perspective, upslope barriers and fences will require significantly more safety considerations during their construction and will have increased maintenance issues compared to drapes, attenuators and other self-clearing structures that require non-specialist contractors to clear debris with standard equipment at road level.**
- **The risk to workers on slopes (roped access) and at the base of slopes needs to be critically reviewed during any recovery project, especially following seismic events where aftershock sequences can be complex and extended. This review must consider other solutions to placing people on the slopes, especially for scaling.**

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PART 11

EMERGENCY AND POST EMERGENCY RESPONSE

Priscilla Crescent Landslide, June 2013

PART 11 – EMERGENCY AND POST EMERGENCY RESPONSE

1 INTRODUCTION

On occasion, and more frequently during and following storm or earthquake events that may result in a state of emergency being declared, geoprofessionals may be called upon to provide emergency response assessments of landslides that have already occurred or are perceived to be imminent.

Emergency responses can involve several stages. In a large emergency, all the following stages may occur, in series or in parallel, while for less severe events only a few stages may be required:
1. Initial response / Rapid Impact Assessment
2. Infrastructure assessments
3. Rapid Building Assessment (RBA) and RBA Placard Re-assessments
4. Interim Use Evaluation
5. Detailed Damage Evaluation
6. Insurance Assessments
7. Private Remedial Assessments

When undertaking an emergency response inspection, the geoprofessional should ensure that:
- As the highest priority, their safety and the safety of others is maintained.
- The inspection is undertaken in an empathetic manner.
- Their role, responsibility and limitation of advice is clearly defined.
- Their decision-making process and reporting procedure are clearly defined when property placarding on behalf of the territorial authority is required as part of the RBA process.
- Observations and decisions are carefully documented with brief justifications.

A useful introduction to the process for managing buildings in an emergency is provided on the MBIE website (link provided at end of this part of the guidance).

There may be some situations where the assistance of a geoprofessional is required when a state of emergency has not been declared (for example a landslide affecting a single house or infrastructure element such as a transmission tower). In this case, the geoprofessional will need to be guided by the relevant authority. This could be the local Council or an Asset Owner (Waka Kotahi, KiwiRail, Transpower as examples).

2 EMERGENCY RESPONSE

2.1 INITIAL RESPONSE / RAPID IMPACT ASSESSMENT

When a local or national state of emergency has been declared, emergency services and CDEM (Civil Defence Emergency Management) Group representatives will complete initial assessments on the ground, and possibly from the air, and collate other information on overall impacts. This is usually undertaken under the control of the emergency services (typically Urban Search and Rescue (USaR), which is led by Fire and Emergency New Zealand), working under the Civil Defence and Emergency Management Act.

The goals are to identify:
- which areas and people are affected and how severely; and
- which buildings, structures, interrupted services, and secondary hazards pose significant public safety concerns.

Rapid impact assessments build situational awareness to support establishing priority needs in the response. They are a key source of information when making decisions about whether a state of emergency should be declared or transition period notified; and if so, whether a rapid building assessment operation is needed and where.

The stages in a Rapid Impact Assessment comprise some or all of the following:
1. Wide area assessment
2. Sector assessment
3. Primary search and rescue
4. Secondary search and rescue
5. Full-coverage search and rescue

The initial response is generally undertaken by LandSAR in rural areas or USAR in urban areas.

2.2 INFRASTRUCTURE ASSESSMENTS

Following on from the Rapid Impact Assessment (which should have identified the critical infrastructure that has been damaged or put at risk), more detailed assessments are undertaken on behalf of the responsible utility owner (e.g. Waka Kotahi).

Each asset owner will have their own standards and processes which should be referred to when undertaking these assessments.

2.3 RAPID BUILDING ASSESSMENT

Rapid building assessments are a brief evaluation of individual buildings and their immediate surrounds for damage, usability and hazards exposure. The goal is to assess immediate risk to public safety. When carrying out these assessments, actual or potential land instability and geotechnical hazards also need to be considered. The RBA is undertaken under the control of the local Territorial Authority under the Building Act.

Further information can be found on the MBIE website (link provided at the end of this Part of the guidance). It is essential that all geoprofessionals working in a Rapid Building Assessment team are familiar with this document, the process and the relative roles of engineers and geoprofessionals.

There are two assessment levels:

- Level 1 Raid Assessments – these begin within hours of the event and mainly involve external building inspections. They are likely to take about 15 to 20 minutes per building.
- Level 2 Rapid Assessments – these may be completed at the same time as Level 1 assessments or later. They involve internal and external inspection, and can take two to four hours per building.

Rapid building assessors issue placards for each building they assess. Placards must be issued under the Building Act for assessments taking place in areas that have been designated for building emergency management. Placards are issued under the CDEM Act if there is no designation in place but a state of emergency or transition period is in effect.

Full details of how to undertake a Rapid Building Assessment for geotechnical failures are given in the MBIE *Field Guide: Rapid Post Disaster Building Usability Assessment – Geotechnical* (link provided at the end of this Part of the guidance).

Any person properly authorised to undertake an assessment under the Civil Defence Emergency Management Act 2002 or the Building Act (as relevant) may undertake a rapid building assessment. However, MBIE recommends:

- only trained Rapid Building Assessors should complete rapid building assessments and placard properties and buildings (which may be on the advice of the engineer).
- any building officials and engineers who are not trained on rapid building assessments but who are supporting the emergency response form part of a team with a trained assessor.

3 GEOPROFESSIONAL'S ROLE DURING EMERGENCIES

The role of the geoprofessional in an emergency response inspection may vary but could include:

- Identifying further failure scenarios and any immediate risk
- Establishing the risk zone based on the probable scenarios
- Participating in, or advising, a Rapid Building Assessment team
- Warning the property owner and any other affected properties
- Requesting shut down of services and ensuring feedback from the service provider has been obtained
- Informing the territorial authority (if the work is being undertaken for insurers or the property owner only)
- Installing preliminary measuring and monitoring systems (e.g. putting pegs in the ground or markers across cracks etc)
- Identifying options to reduce the risk of injury.

3.1 UNDERSTANDING THE ROLE

The geoprofessional should always ensure before starting any emergency response that they:

1. Understand which stage of response they are involved in
2. Know their role (and its limitations) within the response
3. Understand the legal framework under which they are operating
4. Know who the decision-maker is
5. Have their responsibilities documented and understand the liability they are taking on.

Inspections undertaken on behalf of any party other than a territorial authority under the Rapid Building Assessment process (see below) do not permit the geoprofessional to legally enforce evacuation of the property. If the geoprofessional considers that there is a high risk of landslide activation or reactivation that could cause injury or death, and/or damage to buildings then this should be immediately referred to the Territorial Authority or National Emergency Management Agency after informing any persons on the property (residents, assessment teams, etc).

Emergency situations can lead to engineers operating beyond the bounds of their competence. Geoprofessionals should regularly consider if the work they are undertaking, or the advice they are giving, is appropriate.

In most cases the geoprofessional will be an advisor, not the final decision maker. They should be aware of their role within the system and understand how decisions are made. The Coordinated Incident Management System (CIMS) framework is used during emergency responses across New Zealand. More information on this can be found on the National Emergency Management Agency website.

3.2 RAPID BUILDING ASSESSMENT

When a rapid building assessment is conducted, the following steps should be followed by the geoprofessional:

1. **Identify the hazards**
2. **Assess building and its surrounds**
3. **Note other hazards**
4. **Record details of assessment**
5. **Assist the Building Response Manager (BRM) to assign the appropriate placard to the building.** The placard will be one of the following:
   a. Red (entry prohibited because of either land risk or damage to building).
   b. Yellow (restricted access by supervised personnel, or for a short period of time)
   c. White (can be used)
6. **Advise the BRM on access restrictions** to the building where necessary
7. **Advise the BRM on information** that should be provided to the building owner
8. **Advise the BRM Lead on any further inspections,** if necessary. These may include:
   a. Rapid Building Assessment – further technical support from a Tier 1 technical lead
   b. Interim Use Evaluation – assesses the impact of damage on the continued use of a building or adjacent property, with an emphasis on public safety. This is generally done by a CPEng Engineer (structural) contracted by the building owner/user. This is the responsibility of the building owner and would be outside of the scope of the RBA.
   c. Detailed Damage Evaluation – determines the full scope of damage and required repairs and resources. This is done by structural and geotechnical engineers and is the responsibility of the building owner. This evaluation is outside the scope of the RBA

3.3 RECORD KEEPING

Taking notes is commonly forgotten in emergency situations. It is essential that the geoprofessional keeps concise and frequent notes of their observations and any decisions made, along with who made them and the justifications for them. The Geotechnical Rapid Assessment Form provided in Section 7 of the MBIE Building Usability Assessment guide provides a useful checklist for record keeping. The items considered on this form can be adapted to emergency situations which do not involve buildings.

It is likely that decisions will be questioned later, as they will have material impact on people's safety and on their most valuable assets. Homeowners, coroners and courts may demand copies of these notes. Robust records are essential to avoid serious legal consequences.

3.4 SAFETY DURING AN INSPECTION

The safety of the geoprofessional and others is of highest importance when undertaking an emergency response inspection. For advice about managing your safety in geotechnical emergency situations, see the *Field Guide: Rapid Post Disaster Building Usability Assessment – Geotechnical* (link provided at the end of this part of the guidance).

Key items to consider:

- If the property has already been placarded by the territorial authority and the inspection is on behalf of an insurer or the property owner, then the obligations of the placard need to be met. Red and yellow placards prohibit or restrict access respectively into the placarded buildings on the property, and access will need to be agreed with the territorial authority.
- Geoprofessionals should work in teams of two, carefully and safely inspect the property and use engineering geological judgement based on their experience to determine whether there is an immediate risk to the building or life. For subtle or incipient landslides, the geoprofessional should look for surface features described in Part 3 of this guidance document - Recognition and identification of Landslides.
- The inspection should be undertaken from safe ground off the landslide or inferred landslide as much as possible. Inspecting confined or tight spaces must be avoided. Keep out in the open and if the ground starts to move, then evacuate to solid ground away from the direction of movement. Know your escape route. The geoprofessional should not stay in the landslide area longer than necessary.
- Where the landslide is well developed (Figure 11.1 for example), there should be no need to enter the landslide area. If there is a good reason the geoprofessional needs to walk onto the landslide then they should watch for areas where there is visible soil or rock movement and to listen out for cracking, rumbling or crunching sounds. They should also be vigilant for steep unstable slopes, falling, sharp objects in the debris, soft debris that could be sunk into, severed live services, cavities, contamination or exposure of hazardous material.
- While you will be likely dealing with people under stress and suffering trauma, no geoprofessional should tolerate any verbal or physical abuse. You are there to do a job and if you feel that you cannot do that job safely, you should leave immediately.

Figure 11.1 A very unsafe situation. Vista Grove Landslide, Lower Hutt, August 2006

3.5 THE IMPORTANCE OF EMPATHY

See Section 12.2 of the Field Guide: Rapid Post Disaster Building Usability Assessment – Geotechnical for more information.

A landslide can cause damage to treasured property or possessions, affect health, cause injury or harm to persons, animals or pets, and have significant financial impacts.

Property owners may be dealing with a whole range of emotions following the landslide. Whether it be anger ("its unbelievable how badly we have been treated"), fear ("will it ever be safe to go back home?") or irritation ("the engineer didn't turn up when they said they would") the geoprofessional needs to be empathetic and understanding but to remain vigilant of their own and others safety while still collecting all the information they need to make an informed decision on the landslide risks.

Empathy usually refers to understanding and entering into another's feelings by *'putting yourself in someone else's shoes'*. It is important for the geoprofessional to show empathy when inspecting landslides by listening to the property owner, showing them respect and looking at the landslide from their perspective as this is often a unique situation for them while for the geoprofessional it is one property of many dozens or even hundreds that is being assessed.

As a geoprofessional, you are on site for a specific purpose, and while you should listen (with empathy), you cannot solve all of the problems that the landowner might have. Instead, you will need to refer people to the correct agencies that can help. Taking additional information with you, for example sheets with phone numbers and contact details for different agencies, that you can hand to the property/homeowner is very useful.

Geoprofessionals who may have to deal with challenging situations are recommended to undertake a Psychological First Aid course (such as the one available from Red Cross).

4 RECOVERY (POST EMERGENCY PHASE)

In the Recovery Phase the geoprofessional may be involved in tasks that assist the territorial authorities, insurers and/or property owners to make decisions in relation to the future occupation and use of a home or commercial building. In all cases, the geoprofessional is an adviser, not a decision-maker. Even if the decision seems obvious, be aware that you must be very careful about what you say, particularly to the owners.

4.1 RBA PLACARD RE-ASSESSMENTS

Placard reassessments are undertaken under the control of the local Territorial Authority under the Building Act, where a Designation is in place, or under the Civil Defence and Emergency Management Act where a Building Act Designation has not been made but a State of Emergency has been declared. It is important to understand the legislation under which you are operating. If in doubt, request advice from the relevant Territorial Authority.

Normally placard reassessment will occur once remedial works have been undertaken to mitigate the risk. It may also occur where the risk has changed naturally, or where new information allows a re-evaluation of the risk.

If the risk remains, but the Declaration and/or State of Emergency has come to an end, the placard may need to be replaced by a notice issued under Section 124 of the Building Act 2004. This will be issued by the Territorial Authority.

4.2 INTERIM USE EVALUATIONS

These are undertaken on behalf of building owners to assesses the impact of damage or natural hazard risk on the continued use of a building or adjacent property, with an emphasis on public safety.

4.3 DETAILED DAMAGE EVALUATIONS

These are undertaken on behalf of building owners (or sometimes their insurer) to determine what remedial works are required to manage the risk. These may include identification of works that are required to enable the territorial authority to remove the placard placed as part of the Rapid Building Assessment, and other works required for longer-term stability and safety.

4.4 INSURANCE ASSESSMENTS

These are often undertaken on behalf of private insurers to evaluate an insurance claim, under the Earthquake Commission Act (1993) (to be replaced with the Natural Hazards Insurance Act from July 2024) and taking into consideration the terms of the policy. It is essential that the geoprofessional understands the scope of their assessment, and the limitations that the Acts and any policy limitations may impose on their assessment.

The requirements for assessment under the Acts are very specific, and geoprofessionals will need to undertake training as directed by loss adjusters or insurers acting on behalf of Toka Tū Ake (the organisation formerly known as the Earthquake Commission, EQC).

Regardless of these limitations, the geoprofessional should always act in accordance with the Engineering New Zealand code of ethics. Of particular relevance are the requirements to report adverse consequences, to inform others of the consequences of not following advice, and to take reasonable steps to safeguard health and safety.

4.5 PRIVATE REMEDIAL ASSESSMENTS

These are generally equivalent to a Detailed Damage Evaluation undertaken on behalf of building owners (or sometimes their insurer) to determine what remedial works are required. These may include identifying or designing works that are required to remove the placard placed as part of the Rapid Building Assessment, and other works required for longer-term stability.

5 REFERENCES AND KEY INFORMATION SOURCES

The following document should be read and understood:

- Engineering New Zealand, 2016. Code of Ethical Conduct. https://www.engineeringnz.org/engineer-tools/ethics-rules-standards/code-ethical-conduct/
- Engineering New Zealand, 2022. Understanding the bounds of your competence. https://d2rjvl4n5h2b61.cloudfront.net/media/documents/Bounds_of_competence.pdf
- MBIE, 2018. Field Guide: Rapid Post Disaster Building Usability Assessment – Geotechnical. https://www.building.govt.nz/assets/Uploads/managing-buildings/post-emergency-building-assessment/building-usability-assessment-geotechnical.pdf
- MBIE, Introduction to the Rapid Building Assessment System (website, accessed July 2023): https://www.building.govt.nz/managing-buildings/managing-buildings-in-an-emergency/rapid-building-assessment-system/
- National Emergency Management Agency, Coordinated Incident Management System (website, accessed July 2023). https://www.civildefence.govt.nz/resources/coordinated-incident-management-system-cims-third-edition/

Geoprofessionals should also be aware of the following aspects of legislation should also be known:

- Building Act 2004, especially:
  - Section 71 to 74
  - Section 121 to 129
  - Section 133BA to 133BY
- Civil Defence Emergency Management Act 2002, especially:
  - Section 91 and 92
- High Court Rules 2016, Schedule 4, Code of conduct for expert witnesses
- Earthquake Commission Act 1993 https://www.legislation.govt.nz/act/public/1993/0084/latest/DLM305968.html
- Natural Hazard Insurance Bill (coming into effect on 1 July 2024) https://www.legislation.govt.nz/bill/government/2022/0113/latest/LMS509581.html

APPENDIX PART 7

SLOPE STABILITY MODELLING

APPENDIX PART 7
SLOPE STABILITY MODELLING

A1 SOIL SHEAR STRENGTH

A soil's shear strength is the maximum shear stress it can withstand before failure occurs. Resistance to shear is provided by the soil's interparticle contacts and therefore effective stress governs the shear strength of soil regardless of whether failure happens under drained or undrained conditions. The Mohr-Coulomb failure envelope (shown in Figure A1) is the relationship between the soil's shear strength and effective stress and can be expressed as:

$$\tau = c' + \sigma'_{ff}\tan\phi'$$

Where:

τ = soil shear strength – the effective shear stress on the shearing surface at failure
c' = effective stress cohesion
σ'_{ff} = effective stress on the failure plane at failure
ϕ' = effective stress internal friction angle

Undrained loading of saturated soils results in a horizontal total stress strength envelope where shear strength is constant and independent of the total stress. This occurs because a change in normal stress causes an equal change in pore pressure but no change in effective stress, and hence no change in strength. Figure A1 shows the total and effective strength envelope of a saturated clay. The total stress strength envelope is defined by:

$$\phi = 0° \text{ therefore, } \tau = c = Su$$

Where:

c = total stress cohesion (kPa)
S_u = undrained shear strength (kPa)

Table A1 provides a summary of advice and considerations when estimating soil shear strength.

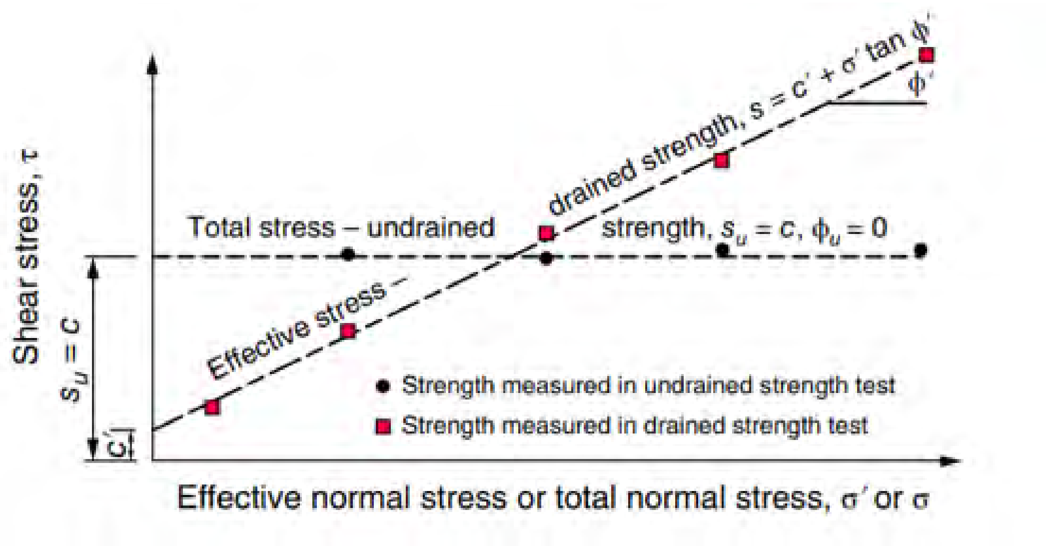

FIGURE A1: Drained and Undrained Strength Envelopes for Saturated Clay (Duncan et al 2014, Fig. 3.5)

TABLE A1: Summary of Considerations when Estimating Soil Shear Strength

SOIL TYPE	STRENGTH PARAMETERS	METHODS FOR EVALUATING STRENGTH PARAMETER/ COMMENTS
Granular Soils - Sands and Gravels	**Drained, ϕ'**	• Granular soil strength envelopes exhibit a roughly linear relationship between shear stress and the effective overburden at low values of overburden, but at higher normal stresses the failure criterion becomes increasingly curved due to particle breakage effects. A linear envelope assumption at high overburden can be unconservative. Further discussion of this phenomena can be found in Duncan et al (2014). • Laboratory testing of granular soils can be problematic due to the difficulty in obtaining "undisturbed1" samples for laboratory testing, and limitations on grain size that can be accommodated by the laboratory equipment. • Strength of granular soils is often best estimated based on: – correlations with material properties (gradation, relative density, and confining pressure) or – results of in situ tests (SPT, CPT, dilatometer). • Estimating the undrained strength of liquefied sands in an earthquake event is discussed in Module 3 of the MBIE Earthquake Engineering Series
Silts	**Drained and Undrained, Su, ϕ' and c'**	Silts are less well-understood than granular soils and clays and their behaviour can range from clay-like to sand-like. Some special considerations when estimating silt shear strength are: • It is difficult to obtain "undisturbed" samples in low plasticity silts. • Low plasticity silts, even when normally consolidated, dilate when sheared. In undrained laboratory tests, dilation causes a decrease in pore pressures which, when negative, form bubbles (cavitation) within the sample, affecting the behaviour. This tendency to dilate can result in uncertainties in the reliability of undrained laboratory test results. • The range in drainage rate of silts makes it difficult to determine if a silt deposit will be in a drained or undrained condition. If this is the case, both conditions should be considered. • In-situ testing can provide useful estimates of undrained shear strength for plastic silts but Duncan et al (2014) indicates that for non to low plasticity silts correlations with in-situ tests (CPT, DMT) are not as reliable. • It is common practice in New Zealand to use a shear vane to measure the strength of silts. Some researchers, such as Duncan et al (2014), have criticised this because of uncertainty as to whether a shear vane test is genuinely undrained for a low plasticity silt. However, such a test in silt does, at least, provide a measure of the shear strength of the silt under fast loading conditions, whether or not it is truly undrained. So the measure still has value to the geoprofessional to model loading conditions of similar shear duration (i.e. in the order of a minute), so long as the overburden stress when measured is similar to the overburden stress in the model.

1 No sample is ever truly undisturbed.

TABLE A1: Summary of Considerations when Estimating Soil Shear Strength (continued)

SOIL TYPE	STRENGTH PARAMETERS	METHODS FOR EVALUATING STRENGTH PARAMETER/ COMMENTS	
Clay	**Undrained, S_u or S_u/σ'**	Laboratory Tests	Aim to gather a sample as undisturbed as possible by (1) Use thin-walled tube piston samplers, (2) Seal tubes upon retrieval to prevent water loss, and (3) store and transport carefully. The SHANSEP procedure (Ladd and Foott, 1974) can be used to address sample disturbance – this procedure involves consolidating samples beyond in situ stresses to reduce disturbance effects. Commonly used laboratory testing includes: • Unconfined Compressive Test (UCT)- often the cheapest, <u>least reliable</u>, and gives the lowest undrained shear strengths as most sensitive to sample disturbance. Other tests/methods are preferred. • Unconsolidated-Undrained Triaxial Test (UU)- quick and more reliable than UCT, a 3-point test should be carried out. Some researchers have cautioned against the use of UU tests as accurate results rely on the incidental cancelation of inherent errors (fast rate of shearing increases S_u, ignoring anisotropy increases S_u, and sample disturbance decreases S_u). Ladd and DeGroot (2003) suggest less expensive field and laboratory shear vane testing as a better alternative with any cost savings spent on Atterberg Limits testing and consolidation testing. • Laboratory Miniature Vane Shear Test- good for soft saturated clays, simplest and easiest to perform. • Isotropically Consolidated-Undrained Triaxial Test (CU)- soil is isotropically consolidated to a set pressure (σ'_3) then sheared. • Direct Simple Shear (DSS)- provides better estimates of S_u than UU but not as good as CU. • Ko Consolidated Undrained Triaxial Test (CKoU)- sample is anisotropically consolidated before shearing. The undrained shear strengths from CU and CKoU tests are higher than from other laboratory tests and are unconservative for many applications (Duncan et al 2014). • Residual strength parameters can be measured in the laboratory with a ring shear apparatus or CCD, providing they are tested to sufficient strains.
		Insitu Tests	• Hand held Field Shear Vane- good/cheap way of estimating S_u in saturated soils at shallow depths (refer to NZGS Guide for Hand Held Shear Vane, Aug. 2001). • Geonor Field Vane- push-in type vane operated from a drill rig for measuring soil shear strengths at depth, typically in very soft to firm soils. • Cone Penetration Test (CPT)- S_u is related to the CPT tip resistance and overburden using a N_{kt} factor. N_{kt} typically varies between 10 to 18 (Robertson and Cabal, 2015). Holtrigter et al (2017) provides typical values of N_{kt} in Auckland clays. Given the significant range that can occur, the relationship should ideally be based on calibration with field shear vane tests in boreholes. Remoulded undrained shear strengths can be estimated to be equal to the CPT sleeve friction, fs, but due to inherent difficulties in accuracy the estimate should be viewed as a guide only (Robertson and Cabal, 2015). Remoulded shear strengths on this basis ideally should be calibrated with ring shear or long strain direct shear laboratory testing and/or correlation with accepted relationships (e.g.,Skempton and Northey 1952) • Dilatometer (DMT)- S_u can be estimated based on relationships with Kd. The relationship can be improved with calibration based on field shear vane tests. • Standard Penetration Tests (SPT)- relationships of S_u with SPT blow count provide a very crude estimation of undrained shear strength and are not recommended, especially for SPT 'N' values below 10. Other in situ methods are strongly recommended.
		Correlations	A number of correlations exist relating plasticity index and OCR with undrained shear strength. However, there is much scatter in the data used to develop these correlations and their usefulness has been debated (Ladd et al, 1977). A few commonly referenced relationships are included but reliance on these correlations in isolation should be avoided for all but preliminary estimates of strength. Soil strength based on, or calibrated with field and laboratory strength testing is strongly recommended. • Skempton and Bjerrum (1957) - Su = (0.11+0.0037 PI) σ' v (where PI=plasticity index) • Skempton and Northey (1952) – remoulded Su = n*102(1-LL) (where LL= liquid limit) and n is a constant equal to either one or two. • Jamiliokowski et al (1985) - Su / σ' v = 0.23 (OCR)0.8 • Mesri (1989) - Su / σ'v = 0.22 OCR • SHANSEP General Form (Ladd and Foott, 1974) – Su /σ' v = (Su / σ' v)nc (OCR)m - this procedure allows for development of a site-specific correlation relating undrained strength to overconsolidation ratio. It is used in conjunction with field and laboratory testing and addresses the effects of disturbance, strain rate, and anisotropy. It is necessary to know the stress profile (preconsolidation) of the clay. This procedure may not be suitable for highly sensitive or structured clays.
	Drained, ϕ' and c'	Normally consolidated clays have an effective cohesion of zero (c'=0). Effective stress strength parameters (c', ϕ') can be measured in • Consolidated-Drained Direct Shear Tests (CDD), • Consolidated -Undrained Triaxial Tests (CUD) with measured pore pressures, and • Consolidated Drained Triaxial Tests (CD). CD tests are uncommon because they take a long time.	

A2 PORE WATER PRESSURES

When approximating pore pressures based on the piezometric or phreatic surface pore pressures may be estimated as the height of the water column times the specific weight of water (i.e. $u_w = \Upsilon_w * h_w$). The height of the water column is the height to the phreatic surface (where pore pressures equal atmospheric pressure) in unconfined aquifers, or to the piezometric line representing the hydraulic head in confined aquifers. Where the phreatic surface is not horizontal, h_w can be estimated as z* Cosβ (β = slope of the phreatic surface, and z =vertical depth below the phreatic surface). This concept may be useful for hand calculations and is handled directly by stability analysis software.

Other ways of modelling more complex pore pressure distributions in slope stability software include:

- Multiple piezometric lines (applied to different soil units).
- Ru and B-bar coefficients – coefficients relating the pore water pressure with overburden and major principal stress respectively.
- Definition of pore water pressure at discrete points (pressures interpolated between these points)
- Negative pore pressures –negative pore pressures can be specified through the use of a matrix suction friction angle (ϕ_b) which defines the additional component of strength due to matrix suction. As negative pore pressure increases soil strength, it is commonly conservatively ignored.
- Excess pore pressures – excess pore pressures can be generated from applied loads in some software packages.

A3 SEISMIC SLOPE STABILITY

As discussed in Part 7 Section 8, there are a number of key components to simplified seismic slope stability assessments:

- Seismic Demand
- Dynamic Soil Strengths
- Seismic Analyses (Post-Seismic Check, Pseudo-Static Analyses, Newmark Deformation Analyses)

Details of these components are discussed in the following Sections.

A3.1 SEISMIC DEMAND

The seismic demand, defined by either a Peak Ground Acceleration (PGA) or a spectral acceleration at a period of interest (Sa(T)), and the moment magnitude (M_w) of the earthquake are critical inputs to simplified seismic slope stability analyses. These seismic demand parameters can be derived in multiple ways as outlined in Table A2.

The return period selected for analysis will largely depend on the type of asset the slope is related to and its Importance Level. For slopes related to roading, NZTA/Waka Kotahi has defined return periods related to performance load cases (e.g., Damage Control Limit State, DCLS) and these are described in the NZTA/Waka Kotahi Bridge Manual (2022). Guidance for dams is provided in NZSOLD (2015). For slopes related to structures, NZS:1170 (2004) specifies two load cases, serviceability limit state (SLS) and ultimate limit state (ULS) and their corresponding return periods (which are dependent on importance level and the design life of the structure).

TABLE A2: Methods for Deriving Seismic Demand Parameters

DERIVATION REFERENCE/ METHOD	PARAMETERS PROVIDED	DESCRIPTION
MBIE Module 1 (2021) – Appendix A[2]	PGA, M_w	Module 1 of the MBIE Earthquake Engineering Series provides estimates of PGA and M_w based on generic PSHA[1] run for multiple New Zealand locations (Cubrinovski et al, 2022). This method improves upon the generic PSHA presented in NZS 1170.5 and provides interim guidance for routine projects until updates to the National Seismic Hazard Model, NSHM (GNS, 2022) are incorporated into NZS 1170.5.
National Seismic Hazard Model, NSHM (GNS 2022) Webtool[2]	PGA, Sa(T), M_w	New Zealand wide generic PSHA has been completed using the recently updated NSHM (GNS, 2022) and a webtool developed to provide results for a range of input parameters (location, return period, V_{s30}). NZ NSHM (gns.cri.nz)
Local Generic PSHA	PGA, Sa(T), M_w	Some city and regional councils may commission generic PSHA specific to a region or urban centre (e.g. Tauranga City Council – Bradley, 2019).
Site-specific PSHA with or without site response analysis	PGA, Sa(T), M_w	These studies provide seismic demands for a location of interest incorporating specific site characteristics and can account for the most recent studies/data. They should only be carried out by experienced specialists and should be subject to rigorous peer review. These methods are discussed further in Module 1 of the MBIE Earthquake Engineering Series.

[1]Probablistic Seismic Hazard Analysis
[2]These methods may be superseded when the updates to the NSHM are incorporated into NZS 1170.5.

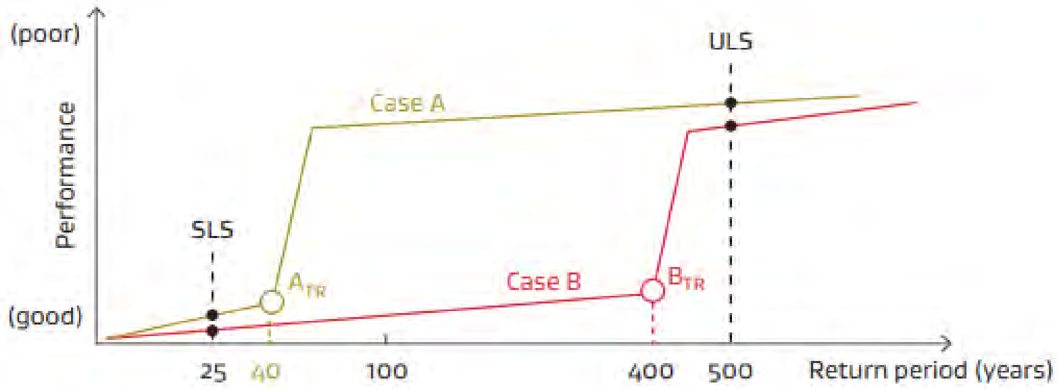

FIGURE A2: Step change in performance reproduced from Module 1 of the Earthquake Engineering Series.

While only discrete earthquake load cases are specified for assessment by NZS:1170, where movement of a slope is triggered between those load cases, it is strongly recommended to assess stability for intermediate return period earthquakes to determine where failure may trigger and identify any step-change behaviour. Figure A2 illustrates how two sites can have similar performance at SLS and ULS return periods, but different risk due to variation in the return period at which the step change in performance occurs. A performance-based approach, where slope displacements are calculated for a range of earthquake return periods to characterise the seismic stability hazard, is helpful in understanding seismic slope stability risk. Additional discussion of performance-based seismic assessment can be found in Bray and Macedo (2023).

Topographic amplification of the ground motion must be considered, based on the specific topography of the site. Procedures for estimating topographic amplification are included in Brabhaharan et al (2018) (integrated into NZTA/Waka Kotahi Bridge Manual,2022), Module 6 of the MBIE Earthquake Engineering Series, and EC8 (European Committee for Standardization 2004).

A3.2 DYNAMIC SOIL STRENGTHS

A3.2.1 Granular Soils

The dynamic shear strength of granular soils depends on the potential for pore water pressure build-up or full liquefaction as outlined below:

- No Liquefaction (factor of safety against liquefaction (FoS_{liq}) > 1.4) - The dynamic strength of granular soils can be represented by effective stress drained strength parameters.
- Partial Pressure Build-Up (FoS_{liq} = 1.1 to 1.4) – reduced shear strength due to decrease in effective stress from pore water pressure build up. Figure A3 can be used to estimate excess pore water pressure and reduced effective stress. Research on pore pressure build-up during seismic shaking is ongoing and additional methods for estimating excess pore pressures are likely to be developed.

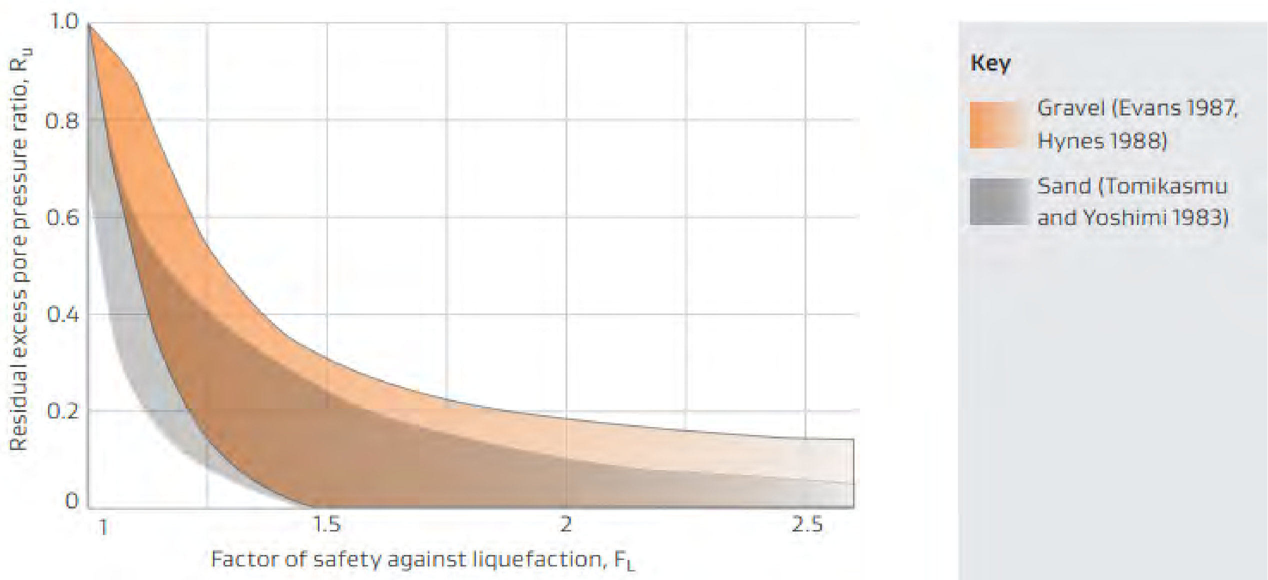

FIGURE A3: Excess pore pressure generation vs Liquefaction FoS (Marcuson et al 1990).

- Liquefaction (FoS_{liq} < 1.1) – Use liquefied residual undrained shear strengths. Residual strengths of liquefied soils and methods for their estimation are discussed in Module 3 of the MBIE of the Earthquake Engineering Series.

A3.2.2 Cohesive Soils

The use of peak or residual dynamic strengths will depend on the level of deformation anticipated.

Chen et al (2006) provides insights on the peak dynamic strength of clays. The peak dynamic undrained shear strength of clay is related to the peak static strength adjusted for dynamic effects:

$$S_{u\ dynamic,\ peak} = S_{u\ static,\ peak}\ (C_{rate})(C_{cyc})(C_{prog})(C_{def})$$

where rate of loading (C_{rate}) >1, cyclic degradation (C_{cyc}) <1, progressive failure (C_{prog})<1, and distributed deformation (C_{def})<1.

It is common for these factors to result in $S_{u\ dynamic,\ peak} \sim S_{u\ static,\ peak}$, but this varies based on the earthquake motion, with long duration shaking resulting in a decreased dynamic strength.

Dynamic residual shear strengths may be appropriate for strain-softening (brittle) soils if deformation is expected. Dynamic residual shear strengths can be assumed to be the static residual shear strength.

A3.3 POST-EARTHQUAKE STABILITY ANALYSIS

The first step in seismic stability analysis is to evaluate whether soils within or below the slope could lose significant strength due to cyclic loading (i.e., liquefaction). If they can, a post-earthquake stability analysis (often termed a "flow failure check") should be carried out to assess static stability following soil strength loss during shaking.

The steps for this analysis are outlined in Figure A4.

A3.4 PSEUDO-STATIC ANALYSIS

Where severe soil strength loss is not anticipated, pseudo-static analyses can be used as a screening tool for seismic slope stability. Pseudo-static analysis is a simplified procedure for evaluating seismic stability. Earthquake loading is represented as a static horizontal force equal to the soil weight times a seismic coefficient, ks. The seismic coefficient is integrated into commercial slope stability software for use in conventional equilibrium analyses.

The key component to pseudo-static analysis is the estimation of the seismic coefficient, ks. The seismic coefficient, ks should be selected based on the tolerable displacement for the given earthquake event. The following methods can be used to estimate deformation-based ks. Steps for completing the assessment are shown in Figure A6.

Estimate dynamic soil strengths. Use residual liquefied undrained shear strength associated with those layers susceptible to liquefaction.

↓

Perform stability analyses with a horizontal seismic coefficient of zero, using residual liquefied shear strengths. Where FoS is below or near one (less than ~1.1), large displacement failure is possible i.e., flow failure. This will be the primary issue for the geoprofessional to consider and either mitigation or additional investigation and more complex analysis will be required. In some cases, lateral spreading procedures may be more appropriate to estimate deformations (Module 3 of the MBIE of the Earthquake Engineering Series).

↓

Where the FoS is greater than the target, it may be desirable to estimate slope displacements as discussed in Section A3.5. However, as noted by Bray and Macedo (2023), Newmark procedures should be applied with caution for cases involving soil that liquefies (See Newmark Methods and Liquefaction in Section A3.5).

FIGURE A4: Post-seismic seismic analysis procedure

- Bray and Macedo (2019). The Bray and Macedo procedure allows the user to define the tolerable deformation. The method is suitable for crustal earthquakes and consideration of, and relationships for, subduction zone earthquakes have been provided by the authors in subsequent papers (Macedo et al. 2023). Spreadsheet implementation of this method can be found on Professor Jonathan Bray's UC Berkeley webpage (Jonathan D. Bray | Civil and Environmental Engineering (berkeley.edu)).
- Anderson et al. (2008)/ FHWA (2011). This method provides a pseudostatic coefficient where tolerable displacements are 50 mm or less.
- Guidance for the seismic design of retaining walls (Module 6 of the MBIE Earthquake Engineering Series) introduces a wall displacement factor W_d, that is used to reduce the PGA to provide a horizontal seismic coefficient. The level of reduction depends on the sensitivity of the situation to movement of a retaining structure instead of being directly related to a specific deformation. A W_d factor is provided for six different scenarios and the geoprofessional selects which case best applies. This approach to determining ks may be appropriate for slopes that fit into the defined cases, and where the level of deformation does not need to be determined explicitly.

ASPECTS OF THE SEISMIC COEFFICIENT, KS

- ks is not the PGA. ks is typically less than the PGA to account for incoherence of the motion in the sliding mass and the allowance of some deformation.
- The value of the seismic coefficient (ks) corresponds to a specific factor of safety and deformation. Different combinations of ks and FoS can describe an equivalent performance as shown in Figure A4.
- In most cases, some deformation following an earthquake is tolerable and selection of ks should account for this.
- The Maximum average Horizontal Acceleration (MHA) is the maximum value of ks and is the value of ks associated with no displacement. The MHA accounts for the cumulative effects of incoherent motion in a deformable sliding mass. The MHA is generally less than the PGA as the PGA occurs at one point in the soil mass at only one time during the earthquake and the MHA is an average value over the entire mass. MHA is also referred to as k_{max}.
- ks is typically less than the PGA but in some situations such as for a shallow failure near the crest of a slope with little allowable deformation, ks is about equal to the PGA. It is also sometimes assumed that ks is equal to PGA where soils susceptible to progressive failure (heavily overconsolidated, brittle soils) are present and their peak strengths are used in the assessment. This is not required where strength loss associated with the expected deformations (i.e. typically residual strengths) have been assumed.
- It is typical to ignore vertical acceleration.

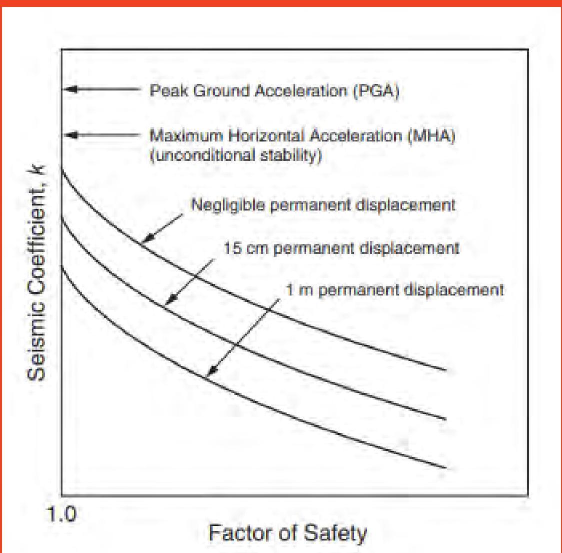

FIGURE A5: Relationship between seismic coefficient, deformation, and FoS

Determine the seismic demand parameters for the return period of interest as outlined in Table A2.

↓

If soils can lose significant strength at the design level of shaking, first carry out a post-seismic check as detailed in Section A3.3. If an adequate FoS exists in this case, continue to next step.

↓

Determine dynamic soil strengths compatible with the anticipated deformations (Section A3.2).

↓

Determine the allowable threshold displacement – based on input from stakeholders/design team.

↓

Calculate the seismic coefficient, ks in accordance with the selected method(s) (Section A3.3).

↓

Perform stability analysis with the calculated ks. If FoS > 1, then displacements are assessed to be less than the selected threshold. If FoS < 1, displacements will be greater and deformation analysis (Section A3.5) can be undertaken.

FIGURE A6: Pseudo-static seismic slope stability screening procedure

A3.5 ESTIMATING EARTHQUAKE INDUCED DISPLACEMENTS

Earthquake-induced slope displacements can be estimated using relatively straightforward Newmark-block type procedures, or more complex non-linear numerical methods. In many cases a simplified approach is sufficient, and for higher risk projects, the simplified approach provides an initial indication of performance that can be used to determine if more complex analyses are warranted. Owing to the complexities of dynamic slope performance, displacements estimated using these procedures are approximate and should be considered only as indicators of likely seismic performance.

The Newmark (1965) sliding block method assumes that a rigid slope mass moves during an earthquake if the induced acceleration exceeds a critical value known as the yield coefficient (k_y). This coefficient, applied as horizontal acceleration, brings the block to limit equilibrium with a FoS = 1. The method calculates total displacement by

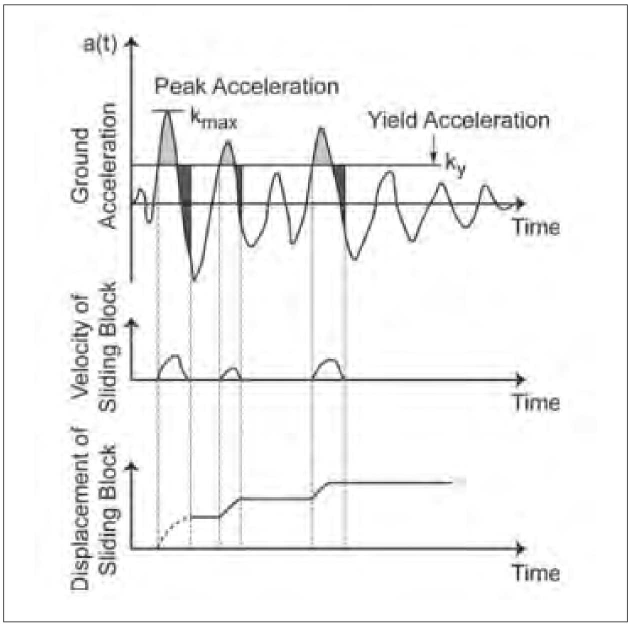

FIGURE A7 – Illustration of the Newmark sliding block method

double integrating portions of the earthquake record where acceleration exceeds the critical value. The Newmark sliding block method is illustrated in Figure A7.

The Newmark sliding block method requires design acceleration time history inputs. These time histories can be determined from ground motion studies where measured time histories from past earthquakes have been selected and scaled to match a design earthquake spectra. The selection and scaling of ground motions requires experienced specialists and is not typically carried out for landslide projects.

Conveniently, researchers have analysed suites of earthquake time histories using Newmark methods to provide simplified procedures for estimating seismic displacement. A minimum of three procedures should ideally be used to estimate deformations to get a sense of the uncertainty in the estimates. The range of estimates can be used for a scenario analysis, or either average or upper-bound deformation estimates selected for design.

Assessing deformations for a range of k_y values can indicate how sensitive the estimation of performance is to the ground model for a specific seismic demand. This can be used to determine if more investigation or complex analyses are warranted.

A selection of procedures for estimating seismic slope displacements is outlined in Table A3, and the general procedure for estimating slope displacements is shown in Figure A8.

Further commentary on the methods in Table A3 will be provided in Unit 4.

TABLE A3: Simplified slope displacement procedures

METHOD	APPLICABLE TECTONIC REGION TYPE[1]	APPLICABLE SLIDING MASS TYPE[2]
Jibson (2007)	Shallow Crustal	Rigid
Anderson et al. (2008)	Shallow Crustal	Rigid
Rathje and Antonakos (2011)	Shallow Crustal	Flexible and Rigid
Bray and Macedo (2019)	Shallow Crustal	Flexible and Rigid
Macedo et al (2023)	Subduction Zone	Flexible and Rigid

[1]New Zealand contains both shallow crustal and subduction zone regions. Displacement procedures have been developed for both regions. The 2022 NSHM disaggregation webtool can be used to determine the region type that dominates the hazard for a particular location and return period.
[2]Some displacement procedures assume that the sliding mass is rigid. This is a reasonable assumption for relatively shallow sliding of stiff soils. Some newer methods incorporate the response of flexible sliding masses.

Determine the seismic demand parameters for the return period of interest as outlined in Table A2.

↓

If soils can lose significant strength at the design level of shaking, first carry out a post-seismic check as detailed in Section A3.3. If an adequate FoS exists in this case, continue to next step

↓

Determine dynamic soil strengths compatible with the anticipated deformations (Section A3.2).

↓

Perform stability analyses, varying the value of horizontal static force applied until the FoS against a critical slope failure is equal to one. This is the yield coefficient, ky.

↓

Calculate displacement using simplified slope displacement procedures outlined in Table A3.

FIGURE A8: Procedure for Estimating Slope Displacements

NEWMARK METHODS AND LIQUEFACTION

It is important to appreciate that the underlying assumptions for Newmark procedures may not be compatible with the distributed deformation of liquefied soils (MBIE Module 3, 2021). These methods should not be used for cases where liquefiable soils are present without acknowledgement of the large uncertainties that may exist in estimated deformations.

Where the post seismic FoS is < 1 and liquefaction-induced lateral spreading type movement is expected to occur, lateral spreading methods outlined in Module 3 of the MBIE Earthquake Engineering Series are likely more appropriate than Newmark procedures for estimating displacement. Lateral spreading can occur after ground shaking (i.e., under no inertial loading), and Newmark procedures are generally not appropriate in these cases.

ESTIMATING THE SEISMIC YIELD COEFFICIENT

The seismic yield coefficient, k_y depends on the specific failure surface being assessed. There are three surfaces that may be considered:

A) The surface with the lowest static FoS

B) The surface that produces the minimum k_y

C) The surface that produces the lowest k_y/MHA

The surface with (A) the lowest static FoS is often not the one that produces the most critical yield acceleration. In the authors' experience the slip surface corresponding to (B) the minimum ky can become unrealistically deep and long. Selecting the surface with (C) the minimum k_y/MHA can help to reduce this tendency as some methods (e.g. Anderson 2008, and Bray and Macedo 2019), account for the change in MHA with increase in the failure mass height. Other ways of managing this tendency include:

(1) focussing on correctly modelling the lateral changes in ground conditions. It is common to simplify the modelled ground conditions by assuming soil stratigraphy extends laterally away from the slope. In reality, soil layers are rarely horizontal and these changes in ground conditions laterally can limit the size of the sliding mass.

(2) reducing the seismic demand to account for lateral incoherence in the ground motion. Some research has been done in this area and will be further discussed in Unit 4.

Further discussion on selecting a realistic yield failure surface will be included in Unit 4.

A3.6 DYNAMIC ANALYSIS OF SLOPES

Most numerical analysis (Finite Element/Finite Difference) software for slope stability provides the user with the means to examine more closely the likely response of slopes to strong earthquake ground motions. They enable study of:

a) Increased porewater pressures that could lead to full liquefaction or the partial degradation of shear strength in silts and clays.

b) The dynamic response of soils including the effects of amplification.

c) The likely displacements the slope may undergo during an earthquake for comparison with acceptable levels of slope movement and the impact on structures or infrastructure.

d) The inclusion of acceleration time history inputs that reflect the frequency content, PGAs, and magnitude of more detailed studies of the site's earthquake hazard.

e) Interaction of the slope with structures founded on, within, or near the slope. Examples include wharf structures impacted by movements of the adjacent slope and bridges with embankments on liquefiable soils that may experience lateral spreading.

As previously described in Section 2.5.2, to generate realistic assessments of earthquake slope performance it is important that adequate investigation of the input parameters has been carried out and that those performing the analyses have appropriate knowledge and experience. Incorrectly modelled numerical analyses can lead to spurious results. Results of complex numerical analysis should always be checked against results from simpler analyses.

A4 BACK-ANALYSIS AND MITIGATION ASSESSMENT

Some considerations when undertaking a back-analysis are:

- Use all the known information in the back analysis to reduce uncertainty and establish a complete model of the slope at the time of failure. The model of the site should be calibrated with the site observations and monitoring of groundwater and ground (surface and subsurface) movements. This may involve refining failure surface searches to match observed signs of movement indicating the extent of the instability. As with any slope stability analysis, the quality of the output of a back-analysis depends on the quality of the input ground model.
- Only one strength parameter can be calculated by back-analysis. In some cases, the location of the failure surface has been used to calculate both cohesion and friction angle, but this has had mixed success. Better results may be gained by using other information to establish one shear strength

parameter (say ϕ'), and back calculate the other (c') (Duncan et al, 2014).
- Back analysis may not provide reliable results where progressive failure has occurred.
- It is critical to understand whether shear strength should be represented by drained or undrained strength parameters.
- Strength parameters determined using back analysis should be consistent with what is known of the materials within the slope based on investigation data and site observations. Where results of back-analysis are inconsistent with what is known, other assumptions of the ground model, such as groundwater depth may being incorrect and a re-evaluation of the ground model is required.

Some considerations for analysis of remediation options include:
- For Limit Equilibrium analysis use a consistent stability algorithm method for both analysis of the stability problem (or back-analysis of the failure) and design of the proposed mitigation. Changing stability algorithms may introduce errors associated with the difference in formulation of the algorithm assumptions.
- The reduction or control of high groundwater pressures is often an effective and cost-efficient method of improving slope stability.
- A buttress placed on top of a shear surface with a low effective friction angle will be inefficient in improving slope stability compared to one placed on a shear surface with a high effective friction angle. If possible, it would be preferable to remove a weak shear surface during construction of a buttress to improve the effectiveness of the buttress at improving stability.

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