NEW ZEALAND GEOTECHNICAL SOCIETY INC
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SLOPE STABILITY GEOTECHNICAL GUIDANCE SERIES

UNIT 3

SLOPE STABILITY ANALYSIS

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

MARCH 2025

ACKNOWLEDGEMENTS

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DOCUMENT STATUS

Issue Date: March 2025
Issue Status: Draft for feedback

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PREFACE

There will always be an unknown but finite risk of failure of any steep slope. This may be difficult to accept, for it goes against the grain to regard a design problem as indeterminate, but it must be accepted and the implications faced before any real advance can be made. By quantifying both the failure probability and the also the consequences of a failure the problem can be re-cast into the form of a benefit-risk analysis of the whole project in which the slopes are involved, and the most useful service that the soil engineer can give is to assess these probabilities and consequences realistically and impartially. It is never perfect safety which is really being sought but an answer to the question "How safe is safe enough" and the answer must always be that this depends on the consequences.
Peter Lumb, 1975

Slope Stability is an enormous subject. Unit 3 endeavours to provide New Zealand geotechnical engineers and engineering geologists with guidance on just one subject – the modelling, the calculations, the analysis, the mathematics, call it what you will, of slope stability problems.

There are several kinds of slopes – natural slopes, cut slopes, fill slopes, slopes that have a mixture of all three. Unit 3 is relevant to all of them.

There are slopes that already have landslides on them, and slopes where landslides have not occurred. Unit 3 is about both.

There are slopes for residential areas, slopes near roads, slopes that form dams, and slopes that form quarries. If you are a geoprofessional working on a slope, and that slope is made of soil or rock, then Unit 3 will have something to interest you.

How steep does the ground need to be for it to be a slope? It depends on the geology, of course – in Northland Allochthon it could be as little as 7 to 10 degrees.

If you've done the appropriate field mapping, and done an appropriate investigation, then Unit 3 provides you with the principles that will allow you to express the margin against instability in mathematical terms. And how to assess stability, after any mitigations have been constructed, and in future events such as storms and earthquakes.

Issues that Unit 3 doesn't cover include:

- How to do a regional or area-wide stability assessment or how to write rules about avoiding natural hazards.
- How to recognise a landslide in the field (see Unit 2 for this).
- What investigations you should do for what types of slopes or landslides (see Unit 2 for this).
- How to mitigate unstable slopes (see Unit 4 for this). Although Unit 3 will tell you how to consider any structural elements, like anchors or piles, if you choose to put them into your analysis.
- Unusual ground conditions.
- Appropriate levels of monitoring during or after construction.
- Dams and mines. If you are operating in a slope stability field with clear authority and guidelines – such as dams or mines – then stick to that authority. But still read this.

If you have a slope and an investigation and a reasonable level of understanding of how that slope has performed so far, Unit 3 will help you answer the question "how stable is my slope?" and it will help you with the next question: "how stable is stable enough?"

New Zealand doesn't have any geotechnical design or analysis standards and until recently, few guidelines. Practitioners rely on publications from overseas and amongst these, there is nothing that constitutes a slope stability analysis guideline, at least not the way we have formulated it here. In this vacuum, a variety of approaches have developed. Some geoprofessionals may be operating at something resembling international best practice, whatever that is, while others may still be assessing slope stability for subdivisions using a handful of Scala probes and their own experience. Good information is disparate and scattered. We have sought to bring together the best information from the publications of authorities and the findings and opinions of researchers.

We have one piece of original research to share. Drawing on the prior work of Silva et al, we have created a mathematically derived basis for Factor of Safety selection under Long-term Static conditions and High Ground Water conditions.

Unit 3 concentrates on simplified methods. These methods should allow you to make useful conclusions on the margin against instability for most slopes. But sometimes you may have a difficult problem, and you will need to research elsewhere for the answer. There is a long reference list to help you there.

Your authors are five geoprofessionals, from different backgrounds and countries, united by our years of demanding experience, propelled by our desire to share the best of our knowledge with you, and galvanised by our enthusiastic belief that this document might someday exist. We hope you read it carefully, use it prudently, and tell us what you think.

We apologise for the lack of conciseness of this document. Slope stability analysis turns out not to be a concise subject.

GLOSSARY

Explanatory Note on NZGS and MBIE Referencing:

This document refers to the Ministry of Business, Innovation and Employment (MBIE) Earthquake Engineering Guidance Series (2021). The modules are referenced in the text as Module 1, Module 2 etc.

This document also refers to the New Zealand Geotechnical Society (NZGS) Slope Stability Guidance Series (of which this report is Unit 3). The units are referenced in the text by their unit number, e.g. Unit 1, Unit 2 and so on.

The following abbreviations are used in this document:

- AEP: Annual Exceedance Probability
- AGS: Australian Geomechanics Society
- AS/NZS: Australian/New Zealand Standard
- ATV: Acoustic televiewer method
- CD: Consolidated Drained Triaxial Compression Test
- COV: Coefficient of Variation
- CPT: Cone Penetration Test
- CU: Consolidated Undrained Triaxial Compression Test
- DDS: Drained Direct Shear test.
- DEM: Discrete Element Method
- DMT: Dilatometer Test
- EGM: Engineering Geological Model
- FDM: Finite Difference Method
- FEM: Finite Element Method
- FoS: Factor of Safety
- GNS: GNS (Geological & Nuclear Sciences) Science
- GSI: Geological Strength Index
- GIR: Geotechnical Investigation Report
- H-B: Hoek-Brown
- HCV: Highest Conceivable Value
- JCS: Joint wall Compressive Strength
- JRC: Joint Roughness Coefficient
- LCV: Lowest Conceivable Value
- LEM: Limit Equilibrium Method
- LL: Liquid Limit
- LoE: Level of Engineering
- MBIE: Ministry of Business, Innovation and Employment
- M-C: Mohr-Coulomb
- MHA: Maximum average Horizontal Acceleration
- NC: Normally Consolidated
- NSHM: National Seismic Hazard Model
- NTH: Norwegian Institute of Technology
- NZGS: New Zealand Geotechnical Society
- OC: Over Consolidated
- OCR: Over Consolidation Ratio
- OTV: Optical televiewer method
- PDF: Probability Density Function
- PGA: Peak Ground Acceleration
- PI: Plasticity Index
- PLT: Point Load Test
- PoE: Probability of Exceedance
- PSD: Particle Size Distribution
- PSHA: Probabilistic Seismic Hazard Analysis
- Q: The Unconsolidated Undrained Triaxial Test is sometimes called the Q test. Separately, Q is also the quality of the rock mass based on the Q-system rock mass classification. And Q is used as the symbol for surcharge in NZS1170.
- RMR: Rock Mass Rating
- RQD: Rock Quality Designation
- SHANSEP: Stress History and Normalized Soil Engineering Properties
- SPT: Standard Penetration Test
- SWCC: Soil Water Characteristic Curve
- TAF: Topographical Amplification Factor
- UCS: Uniaxial Compressive Strength
- UCT: Unconfined Compression Test
- UHS: Uniform Hazard Spectrum
- UU: Unconsolidated Undrained Triaxial Compression Test

1 INTRODUCTION

1.1 THE FOCUS

The subject of slope stability analysis is vast, and as such this document is focussed on specific aspects.

Unit 3 includes and builds on guidance presented in Part 7 of Unit 1. It focusses on the assessment of landslide triggering in soil and rock slopes using limit equilibrium methods (LEM). For most routine projects, limit equilibrium methods provide adequate means for assessing stability and these are the most common methods used in New Zealand and globally. "*Their validity has been demonstrated by back-analysis of actual cases and models, as well as by long experience in practical applications*" (Fell et al., 2000).

Unit 3 provides an overview of foundational topics such as rock and soil mechanics principles, types of limit equilibrium methods and the mechanics of these procedures. These topics are covered extensively in other texts (Abramson et al., 2002; Wyllie & Mah, 2004; Duncan et al., 2014; Turner & Schuster, 1996). This document instead focusses on the practical application of LEM, with an emphasis on scenarios the authors have found difficult to address or issues that are commonly misunderstood and mishandled in analysis. The main topics covered in this Unit include:

- Rock strength and stability assessment
- Estimating soil strength
- Partial saturation, modelling pore pressure distributions and high rainfall scenarios
- Selection of target Factor of Safety (FoS) taking into account uncertainty and consequence
- Seismic slope stability analysis and deformations
- Handling of uncertainty

1.2 BEFORE ANY ANALYSIS

Many key aspects of slope stability assessment happen before analysis begins. The geoprofessional should put emphasis on establishing a robust ground model, which will form the basis for subsequent stability analyses and mitigation design. This model should include:

- Thorough engineering geological appraisal. Has the slope failed or have other slope failures occurred in the vicinity? What triggered them? What are the anticipated modes of failure and the post-failure behaviour based on failures in the vicinity or in similar geology/ground conditions? This is often the most important part of the process. Units 1 and 2 provide discussion on identifying landslides and developing the engineering geological model.

LIMITATIONS OF UNIT 3

Complex Numerical Analysis

Numerical modelling methods are only briefly introduced. These more complex techniques can provide insight into mechanisms of slope failure and deformations not possible with LEM. These techniques are enjoying more widespread use due to technological advances and implementation in user-friendly software. However, in most cases, the quality of the slope stability assessment and its predictive power is more strongly related to the inputs than to the complexity of the analysis. Limit equilibrium analysis coupled with a robust ground model, calibrated against observed performance, provides adequate understanding of stability for most projects. Notwithstanding, we envisage that the use of numerical modelling techniques will become more prevalent over time and expect guidance on these analysis techniques in slope stability assessment may be provided in future revisions of this document.

Area-Wide Landslide Studies

This document focusses on assessment of individual slopes (i.e. an embankment or cut). Area-wide landslide studies provide an evaluation of landslide hazards over large geographic areas and are typically carried out to inform urban planning, infrastructure development and disaster risk management. These types of assessments use information on past landsliding, topography, geology, and potential triggering events to assess the susceptibility, frequency, and consequence of landsliding over the study area. These types of assessments are discussed in GNS Science's Landslide Planning Guidance (de Vilder et al., 2024), and are outside the scope of this document.

Assessment of Post-Failure Behaviour

The detailed assessment of post-failure behaviour is largely outside the scope of this document but should always be considered as part of the stability assessment. This informs understanding of the overall risk and is discussed in Section 13.8. Detailed discussion on assessment methods for post-failure behaviour for rockfall can be found in Unit 1 Part 8, and for debris flow in Unit 1 Part 9. Fell et al (2000) provide recommendations for relatively straightforward ways of estimating travel distance and velocity for a range of slope conditions (natural slope, cut slope, fill slope etc) and is recommended reading.

- Adequate geotechnical investigation and testing to:
  - Define the geological materials comprising the slope and their distribution within it.
  - Define the shear strength properties of the materials within the slope, and
  - Define the pore water pressure distribution within the slope and below the failure surface, and how they vary with time (e.g. seasonally, or extreme events).

Subsurface investigations are discussed in Unit 2, rock strength is discussed in Section 4, and soil strength in Section 10.

Prior to analysis the geoprofessional should also:
- Develop an understanding of environmental factors that may affect the slope over the design life i.e. rainfall, human modification, loading on the slope, earthquakes etc, and
- Develop the target performance criteria (Section 13.6 discusses selection of target FoS and Section 17.7 discusses seismic deformation thresholds).

Based on the ground model the geoprofessional should understand the likely slope performance and compare analysis results to what is anticipated. Where analysis results do not reflect expected performance, the ground model or the analysis type needs to be revisited and refined.

"*Computer programs are only tools that aid in the design. The answers are only as good as the input data. Don't get carried away with plugging in the numbers and examining the results. You may learn the "garbage in - garbage out" principle the hard way*" (Samtani & Nowatzki, 2006).

1.3 HIERARCHY OF AUTHORITY

In some cases, aspects of the slope stability analysis (e.g. target FoS, methods of analysis) are prescribed by a Crown entity, stakeholder, or local authority, such as NZSOLD (2023) for dams or NZTA Waka Kotahi (2022) for highway slopes. If a clear authority is present, then the recommendations of those authorities should be followed.

We suggest that the Unit 3 recommendations are appropriate for slopes where there is no clear other authority present.

2 NATURAL VERSUS CONSTRUCTED SLOPES

Natural and cut slopes differ from constructed fill slopes in several important ways, and the approach to their assessment may also. The geological materials of natural and cut slopes tend to be highly variable and groundwater is often present. In contrast, fill slopes may have more uniform soil conditions, although the materials used can range from sands and gravels to cohesive soils with a high fines content.

2.1 NATURAL AND CUT SLOPES

Natural and cut slopes present significant challenges due to their varied geological conditions. These slopes often consist of different geological strata and materials with varying degrees of weathering and discontinuities. Groundwater is frequently present within the slope, sometimes perched on less permeable layers, and near surface materials are often partially saturated with their degree of saturation changing with variation in rainfall and influenced by vegetation. Evaluating natural and cut slopes can be difficult due to their variability and the challenges in accessing steep or high slopes for site exploration. The variability in natural deposits makes it hard to locate or model critical soil layers accurately, which is essential for assessing slope stability. As a result of these complexities, the stability of natural and cut slopes may often be better assessed by the observation of the long-term stability of nearby slopes than by limit equilibrium methods (LEM), where the models' inputs have high uncertainty.

The assessment of natural and cut slope stability through either LEM or observational methods is aided by an understanding of failures in similar geologies, topographies and in the vicinity. The NZ Landslide Database was recently developed to provide a consolidated, consistent landslide inventory for New Zealand (Roberts, 2023). It is a valuable tool in understanding the likely modes of failure and triggering events for slopes in areas where landsliding has occurred. When assessing the stability of natural and cut slopes, inventories like this should be consulted as part of the engineering geological assessment.

2.2 CONSTRUCTED FILL SLOPES

Where new slopes are being constructed, these slopes are often easier to evaluate because the fill material is commonly well-defined, making the determination of material properties straightforward. Groundwater is typically located below the base of the fill. Geotechnical investigations for fill designs generally focus on the foundation. If the embankment fill is placed on soft or liquefiable foundation material, determining the foundation's strength under static and seismic loading becomes crucial. Conditions in constructed slopes can generally be well modelled using LEM.

Historic fill slopes are less straightforward and understanding past practice used to construct the slope can be helpful in determining critical failure mechanisms (e.g. loosely placed, side-cast road embankment fill, over decomposed vegetation. See Part 3, Unit 1).

3 ROCK ENGINEERING PRINCIPLES

The engineering behaviour of rock materials is different to that of soil materials, in terms of strength and deformability of the materials and the possible modes of failure. Due to the discontinuous nature of rock masses, the methods of continuum mechanics and soil mechanics are generally inadequate for analysing rock engineering problems (Muller, 1964). Therefore, the principles and philosophy of analysing the stability of a rock slope are different to that of a soil slope.

3.1 COMPONENTS OF THE ROCK MASS

Rock masses consist of the intact rock material and individual discontinuity surfaces and are often characterised by anisotropy, depending on the presence, persistence and directional characteristics of the rock discontinuities. To understand the engineering behaviour and properties of the rock mass and how to interpret them for a rock slope stability problem, it is important to understand the basic components of the rock as outlined in the below sections.

Intact rock: is the unfractured block of rock in-between the discontinuities. The size of the blocks may range between a few millimetres for very fractured rocks, to several metres in massive rocks.

Rock discontinuities (defects): are the planes of structural weaknesses or mechanical breaks in the rock mass, with negligible tensile strength (Priest, 1993). Discontinuities can be geologic in origin, formed during the geological history of the rock, or anthropogenic in origin, formed due to blasting, drilling, hydraulic fracturing etc. Discontinuities may be systematically or randomly oriented, parallel or intersecting, and divide the rock into discrete blocks of various shapes and sizes (Figure 1).

Rock mass: is the matrix containing the intact rock and rock discontinuities. The engineering properties of the rock mass are governed by the strength of the intact rock and the frequency, geometry and the physical and mechanical properties of discontinuities.

Rock strength: is the maximum stress level the intact rock, the rock mass or the rock discontinuity surface can carry. The critical rock strength parameter, in the context of a rock slope stability analysis, will depend on the mode of slope failure examined. For example, when analysing discontinuity-controlled failures, the shear strength of the weakness plane will be the critical parameter. When analysing the stability of slope, consisting of a highly and randomly fractured rock mass that could be considered homogeneous and isotropic

(a)

(b)

FIGURE 1: (a) Intact rock sample and (b) samples with discontinuities (shears)

and treated as an equivalent continuum, then the rock mass strength is the governing parameter.

Rock Deformability: Is the ability of rocks and rock masses to deform (change shape, size or volume) when subject to stress. Rock deformability can be instant or time dependent. Rock mass deformability depends on the deformability of both intact rock and the discontinuities. Hard indurated rocks such as greywacke have different deformability characteristics compared to weak sedimentary rocks, which govern the ability of the rocks to sustain elastic or plastic deformations without failure.

3.2 KEY CONSIDERATIONS FOR ROCK SLOPE STABILITY ANALYSIS

In a rock slope stability analysis, the most important factors to be considered are:

- the geometry of the discontinuities behind the slope face that could generate single or multiple-plane translational failure surfaces,
- the possibility of failure through the rock mass of insufficient strength or highly fractured rock with randomly oriented discontinuities that can be considered as a continuum, that can be potentially approximated with an equivalent soil slope circular failure,
- the possibility of failure along a complex, composite surface made up of both discontinuities and rock mass,
- the appropriate selection of the shear strength parameters for the potential sliding surface, as described above.

The sliding surface in a rock slope may consist of a single plane continuous over the full area of the surface, a complex stepped surface controlled by discontinuities, or a complex surface made up of both discontinuities and new fractures through intact rock (rock bridges).

The term ‘rock bridge’ is used to describe the intact rock separating discontinuities. When failure of the rock mass requires failure through the intact rock bridge, which can be an order of magnitude stronger than the rock mass, both the mode and rate of failure are affected.

The selection of an appropriate mode of slope failure and shear strength of the failure plane depends primarily on the relative scale between the sliding surface and structural geology. For example, in the slope shown in Figure 2, the dimensions of the overall slope are much greater than the discontinuity length, so any failure surface is likely to pass through the jointed rock mass and the appropriate rock strength to use in design of the slope is that of the rock mass. In contrast, the bench height is about equal to the joint length so stability could be controlled by a single joint, and the appropriate rock strength to use in design of the benches is that of the joints set that dips out of the face. Finally, at a scale of less than the joint spacing, blocks of intact rock occur and the appropriate rock strength to use in the assessment of drilling and blasting methods, for example, would be primarily that of the intact rock (D. C. Wyllie & Mah, 2004).

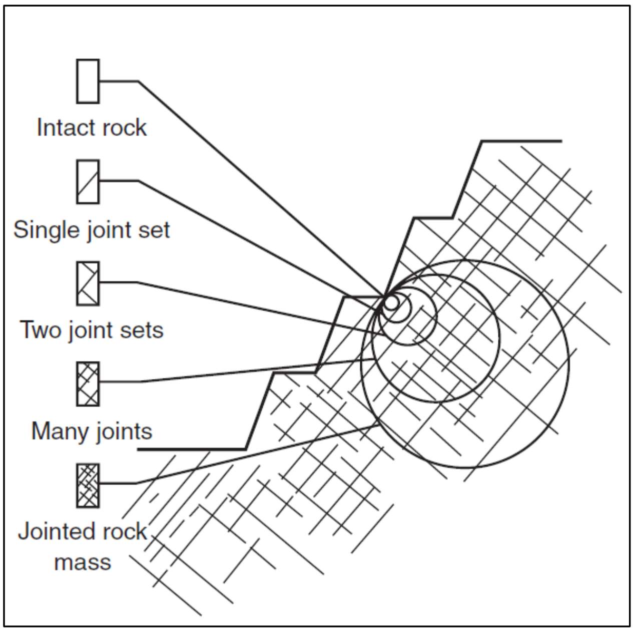


FIGURE 2: Transition from intact rock to jointed rock with increased size of sample (Wyllie & Mah, 2004).

4 ESTIMATING THE STRENGTH AND DEFORMABILITY OF ROCK

4.1 INTACT ROCK STRENGTH

The strength of intact rock should be determined from one of the following tests, in order of preference, starting with the most preferred:

- Triaxial test
- Uniaxial Compressive Strength test
- Point Load test

Each of these tests is discussed below.

The most reliable test to assess the strength of intact rock is the triaxial test. The triaxial test in rock is carried out in a specifically designed cell, see Figure 3, where the specimen is axially loaded under constant confining pressure. The confining stresses used to establish the strength of intact rock should consider the confining stresses anticipated in the problem being analysed. In a slope stability problem, slope failures are expected to be relatively shallow and confining stresses in the failed region or along the failure surface are much less than the vertical stress at the bottom of the slope, especially when the slope is of substantial height. After a sequence of at least three triaxial tests, failure envelopes of the rock samples are derived using the most common failure criteria applied in rock mechanics:

- The Mohr-Coulomb (M-C) Failure Criterion, which provides cohesion and friction angle parameters of the rock specimen.
- The Hoek-Brown (H-B) Failure Criterion, which provides the material constant mi and the uniaxial compressive strength of the intact rock σ_{ci} to be used in the application of the H-B criterion for the rock mass.

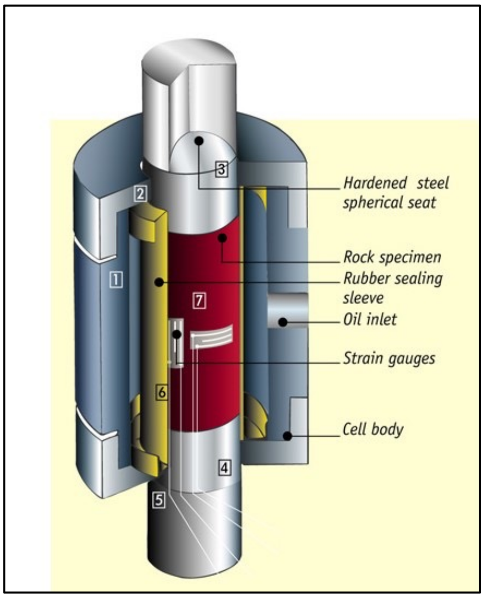


FIGURE 3: Hoek cell for triaxial tests (Hoek, 2023).

It is possible to make deformation measurements during a triaxial test to measure the stress-strain response of the sample and estimate the Modulus of Deformation for the intact rock (Hoek, 2023). Saturation or pore pressure build-up is not a critical issue for rocks as the porosity of rocks is much lower than that of soils, thus testing a dry or a saturated sample would not significantly affect the results. Pore pressures may become an important factor for weak and / or highly porous rocks, and then a specialised type of test allowing for saturation of the rock specimen should be considered. Triaxial tests on saturated samples of Melbourne mudstone are described in Chu et al (1983).

The triaxial test however is rarely carried out in rocks in New Zealand, usually due to time or budget constraints. The most common test for strength of intact rock in New Zealand is the Uniaxial Compressive Strength (UCS) test, which is carried out on borehole core samples 50 mm or 100 mm in length. One of the problems of the uniaxial test for compressive strength testing is the reliability, as the test is carried out with zero confining pressure. Failure occurs at the transition between compressive shear failure and tensile splitting failure of the intact rock. Chakraborty et al (2019) describe the various failure modes that are observed in uniaxial testing, and these are illustrated in Figure 4. The Hoek-Brown criterion is only applicable to shear failure, and it is important to interpret the results of uniaxial tests with caution.

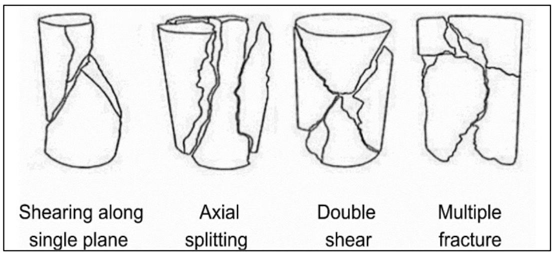


FIGURE 4: Shear failure, axial splitting and more complex failure modes observed in uniaxial compression testing.

A key limitation of both triaxial and UCS intact rock tests is finding borehole core samples that truly represent intact blocks, especially in closely jointed and heavily fractured rock masses which are very common in New Zealand. In most cases the strength of the intact rock measured with these tests is influenced by the presence of hairline or incipient discontinuities which are not easily visible, and this should be taken into account when assessing design parameters. Measured UCS tests should be calibrated with typical values available in the literature.

To overcome some of the limitations associated with finding suitable samples for testing, the Point Load Test (PLT) can be used, ideally supplementary to the UCS test (Broch & Franklin, 1972). For this test, irregularly shaped rock samples can be tested if they have a length to diameter ratio of at least 1.4. The advantages of the test are that it is quick and low cost, it can be carried out in samples of irregular shape, retrieved from the field and not necessarily from borehole cores.

The results of the PLT are usually presented in terms of a reference diameter equal to 50 mm (I_{s50}). The UCS strength is related to the point load index with 50 mm cores by multiplying the I_{s50} value by a factor between 16 and 24, depending on the specimen diameter and rock type (ASTM , 2016; Rusnak & Mark, 2000). Strength anisotropy can also be assessed with the PLT by carrying out diametral and axial tests on the same rock types (Broch, 1983).

The UCS strength of the rock can be indirectly assessed with simplified field tests on rock exposures as well as borehole core samples. When these tests are conducted, the weathering grade of the rock should be recorded for each test, as well the potential influence of discontinuities.

Tests that could be used for the direct or indirect determination of the strength of intact rock are provided in Table 1. Guidelines for the design of a rock specimen laboratory program are provided in Hoek (2023).

4.2 ROCK MASS STRENGTH

The stability assessment of a rock slope is not possible without reliable estimates of the strength characteristics of the rock mass. The strength of a rock mass is controlled by the intact rock strength and the strength of the discontinuities.

An intact rock specimen contains very few or no discontinuities and is homogeneous, which in general results in much higher strength than the rock mass itself. The laboratory tested rock specimen therefore does not represent the strength and deformability properties of the rock mass.

The rock mass strength and the rock mass deformation modulus are difficult to estimate directly in the field or by laboratory testing. Bieniawaski (1989), Hoek et al. (2002) and Barton (2002) have suggested empirical equations for the estimation of both rock mass strength and rock mass deformation modulus. These empirical equations are linked to rock mass classification systems – Rock Mass Rating (RMR), Geological Strength Index (GSI) and Q-system, respectively. These rock mass classification systems include some level of subjectivity, therefore it is best that the estimation of rock mass strength and rock mass deformation modulus are directly linked with laboratory test results.

However, in most cases it is practically impossible to carry out triaxial or shear tests on rock masses at a scale which will provide useful information for design. Consequently, the ability to predict the strength of the rock mass based on direct tests is limited (Hoek, et al., 1995). To overcome this problem empirical criteria have been developed to enable the estimation of the rock mass parameters to be used in design.

4.2.1 The Hoek Brown failure criterion

The most widely used failure criterion for rock is the Hoek Brown (H-B), which was developed to express the non-linear peak strength envelopes of rocks and rock masses in terms of the major and minor principal stresses at peak strength (Brown, 2008). The original expression of the empirical, non-linear, isotropic peak strength criterion for rock masses was given in total stress terms as:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left[m\left({}^{\sigma_3}/_{\sigma_{ci}}\right) + s\right]^{0.5} \qquad \text{Equation 1}$$

where σ_1 and σ_3 are the major and minor principal compressive stresses at peak strength (assuming $\sigma_2=\sigma_3$ or $\sigma_2=\sigma_1$), σ_{ci} is the uniaxial compressive strength of 50 mm diameter samples of the intact rock, and m and s are material parameters.

Table 1: Laboratory and field tests for determining the strength of intact rock

Test	Type	Where	Reference
Geological hammer	Indirect	Field and Laboratory	New Zealand Geotechnical Society, 2005
Schmidt Hammer Rebound Hardness	Index -Indirect	Field and Laboratory	ISRM, 2014
Point Load Test	Index -Indirect	Field and Laboratory	ISRM, 2007
Uniaxial Compressive Strength and Deformability **Complete Stress-Strain Curve for Intact Rock in Uniaxial Compression**	Direct	Laboratory	ISRM, 2007
Triaxial Test	Direct	Laboratory	ISRM, 2007 ISRM, 2019

For intact rock, m and s take their maximum values of m_i and 1.0, respectively, with m_i being a petrographic constant determined from triaxial testing. In the original and subsequent publications of the criterion, tables were given of values of m_i for a range of rock types (Hoek & Brown, 1980a, 1980b; Marinos & Hoek, 2000). From the early stages of development, the H-B criterion attempted to determine rock mass deformability properties as well.

Because of the lack of suitable alternatives when it was first introduced, the criterion was soon adopted by the rock mechanics community and developed over the years to be applied for a range of cases. A detailed history of the developments of the H-B criterion is provided in various publications (Hoek & Marinos, 2007; ISRM, 2014).

The last major revision to the H-B criterion was presented in the 2002 edition (Hoek et al., 2002), where the relationships between the GSI and m_b, s and a were modified, and a new factor D was introduced to account for near surface blast damage and stress relaxation. The latest equation for the generalised H-B criterion is provided below:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left[m_b\frac{\sigma'_3}{\sigma_{ci}} + s\right]^a$$

Equation 2

In the 2002 edition of the H-B criterion, m_i is a curve fitting parameter derived from triaxial testing of intact rock. The parameter m_b is a reduced value of m_i, which accounts for the strength reducing effects of the rock mass conditions defined by GSI (see Figure 8 and Equation 3). Adjustments of s and a are also required as a function of GSI (Equation 4 and Equation 5).

$$m_b = m_i exp\left|\frac{GSI-100}{28-14D}\right|$$

Equation 3

$$s = exp\left|\frac{GSI-100}{9-3D}\right|$$

Equation 4

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

Equation 5

The uniaxial compressive strength of the rock mass is provided by setting σ'_3=0 in Equation 2.

The tensile strength is

$$\sigma_t = -\frac{s\sigma_{ci}}{m_b}$$

Equation 6

The same publication presents the equations for determining equivalent angles of friction and cohesive strengths for each rock mass and stress range and for determining the appropriate value of σ_{3max} for every specific application, including slopes, as follows:

$$\frac{\sigma'_{3max}}{\sigma'_{cm}} = 0.72\left[\frac{\sigma'_{cm}}{\gamma H}\right]$$

Equation 7

The publication provides an updated methodology for predicting rock mass deformability:

For σ_{ci}≤100 MPa

$$E_m\,(GPa) = \left[1 - \frac{D}{2}\right]\sqrt{\frac{\sigma_{ci}}{100}}\;10^{\left(\frac{GSI-10}{40}\right)}$$

Equation 8

For σ_{ci}>100 MPa

$$E_m\,(GPa) = \left[1 - \frac{D}{2}\right]10^{\left(\frac{GSI-10}{40}\right)}$$

Equation 9

GSI is estimated from the charts included in Figure 5 and Figure 6 (Hoek & Marinos, 2007). Scaling of the Hoek Brown failure envelope from the intact rock strength to that of the rock mass using the GSI classification is graphically represented in Figure 7. A method of quantifying the assumptions and inputs for the GSI classification based on Rock Quality Designation (RQD) and Joint Condition ($JCon_{89}$) is provided in Hoek et al (2013). Relationships exist to convert RMR_{89} and Q to GSI (Hoek et al., 1995), where Q is the quality of the rock mass based on the Q-system rock mass classification (Barton et al., 1974). Although these methods of quantification of the GSI value have been developed to make the index friendlier to engineers, Hoek (2007) recommends that GSI be estimated directly by means of the charts published on its use, applied by appropriately qualified engineering geologists (Hoek & Brown, 1997).

Relatively few fundamental changes to the H-B criterion 2002 edition were introduced by Hoek & Brown (2019). The same publication discusses many issues of utilisation and presents case histories to demonstrate practical applications of the criterion and the GSI system.

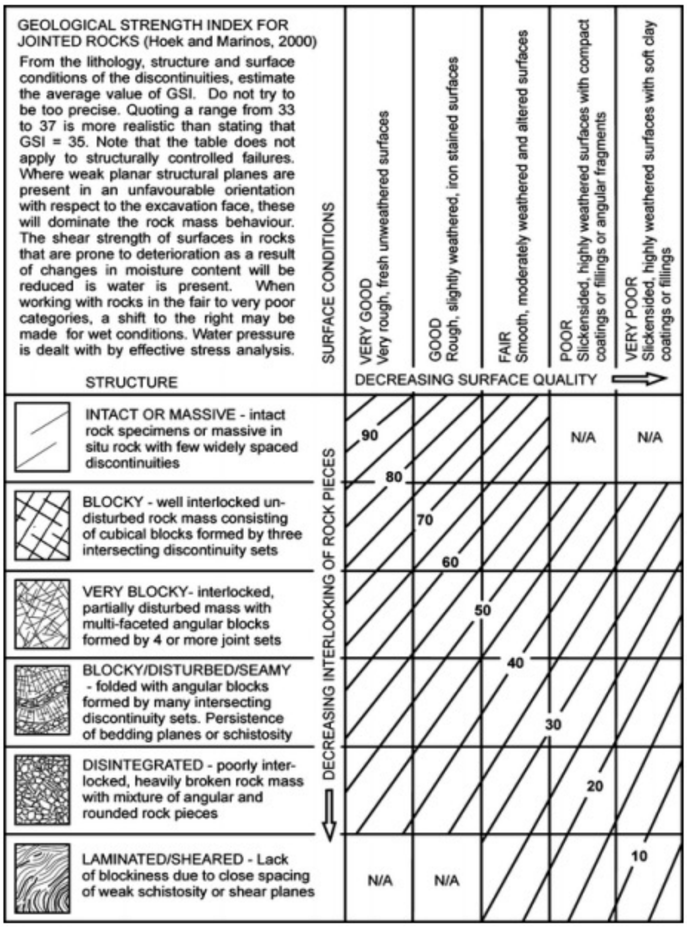
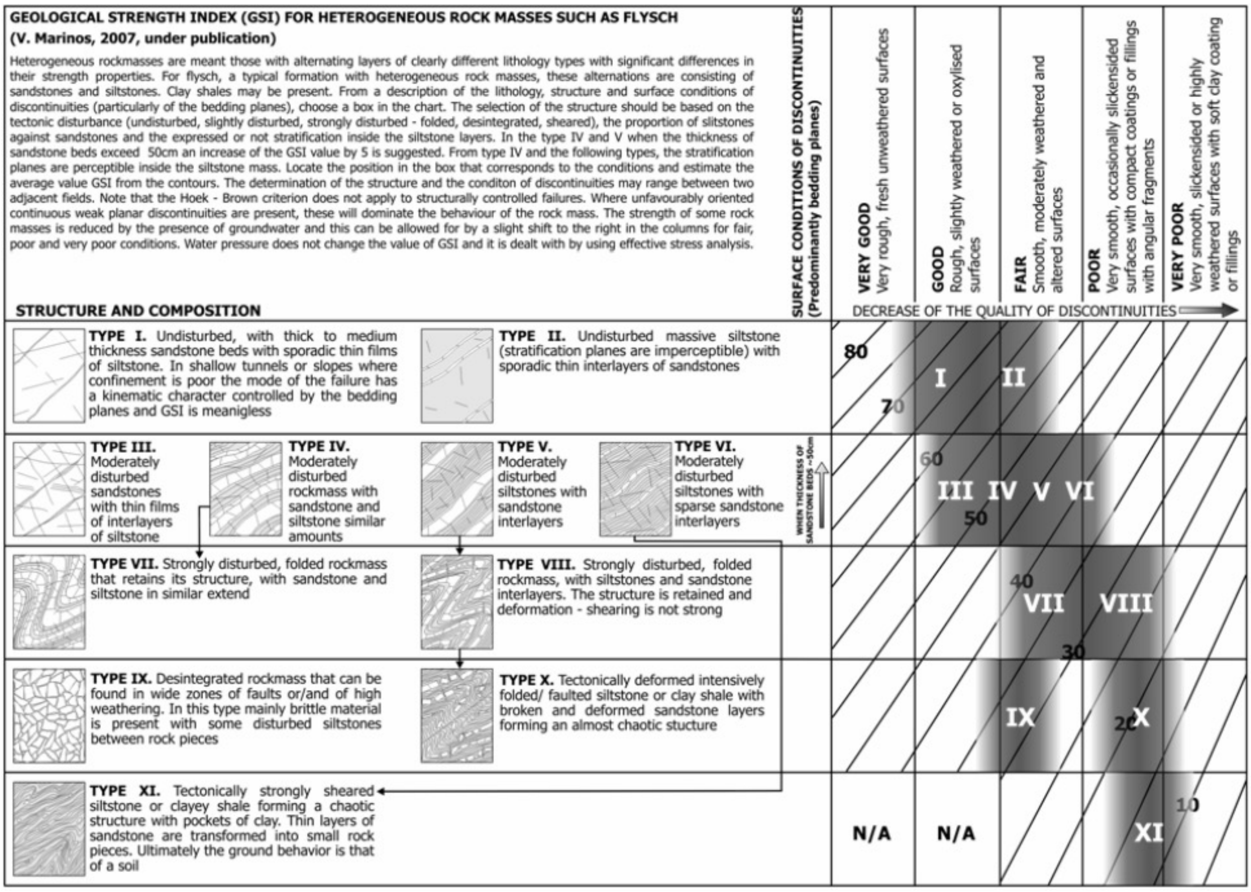


FIGURE 5: General chart for GSI estimates from geological observations (Hoek & Marinos, 2007).

FIGURE 6: GSI for heterogeneous rocks such as flysch (Hoek & Marinos, 2007). In New Zealand, this chart could be used for rock masses such as the East Coast Bay Formation and Wellington Greywacke if 1) clear alternating layers of different lithology can be identified or 2) when complex failure modes along paths of least resistance are possible and the H-B criterion is considered applicable.



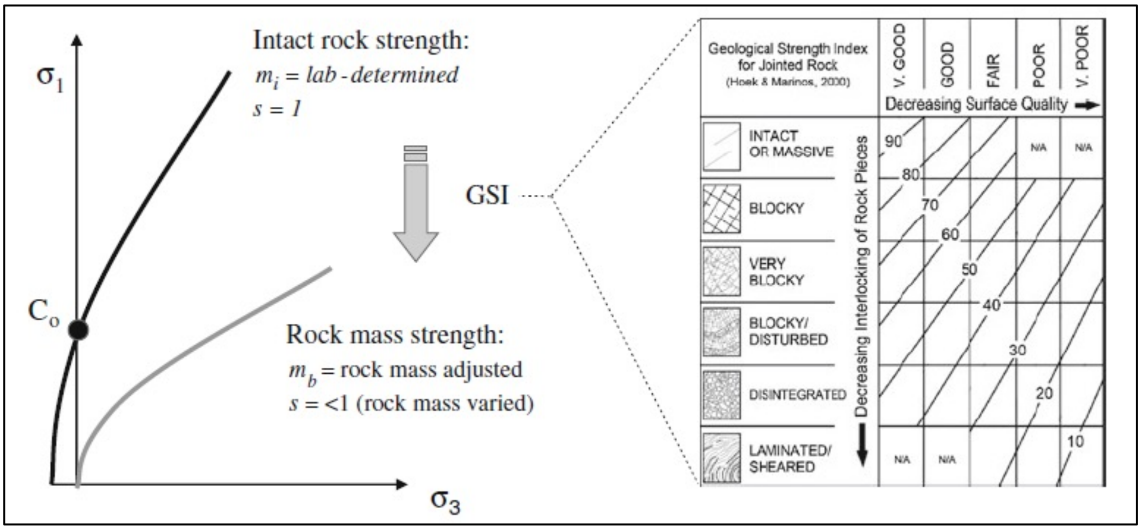


FIGURE 7: Scaling of Hoek Brown failure envelope for intact rock to that for rock mass strength using the GSI classification (ISRM, 2014).

A flow chart of data input and application of the H-B criterion for estimating rock mass parameters is provided in Figure 8.

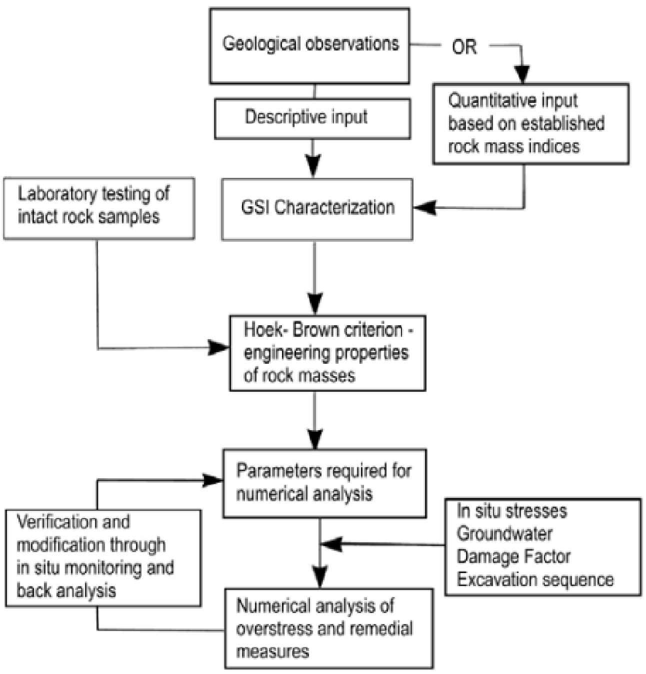


FIGURE 8: Flow chart of data input for the application of the GSI/Hoek-Brown method for estimating rock mass parameters (Hoek et al., 2013).

4.2.2 Applications and limitations of the H-B criterion

The diagram shown in Figure 9 has been used to explain the range of applicability of the H-B criterion. Figure 9 indicates that the H-B criterion should not be used in cases of massive rocks and where there are only one or two sets of discontinuities. In this case, or when the discontinuity spacing is larger than the scale of the problem (e.g. length of slope examined), the form of the criterion for small samples of intact rock and fracture propagation principles for massive rock should be used. In the case of massive rock masses with one or two sets of pre-existing discontinuities and possible failure structurally and gravitationally controlled by one or the combination of these discontinuities, then stability is governed by the shear strength of the discontinuity surfaces, as opposed to or in addition to rock mass failure. The strength of discontinuity surfaces is discussed in Section 4.3.

Similarly, the GSI classification system assumes that the rock mass contains enough "randomly" oriented discontinuities that it behaves as a homogeneous isotropic mass. In other words, the behaviour of the rock mass is independent of the direction of the applied loads. An example of such a rock mass in New Zealand is Wellington Greywacke. The GSI system should not be applied to rock masses in which there is a clearly defined dominant structural orientation or structurally dependent gravitational instability.

The H-B criterion and GSI method should therefore be used with caution in the case of anisotropic rocks, such as Otago Schist. When failure is governed by a preferential direction following a dominant orientation of one or two discontinuities, the use of the GSI to

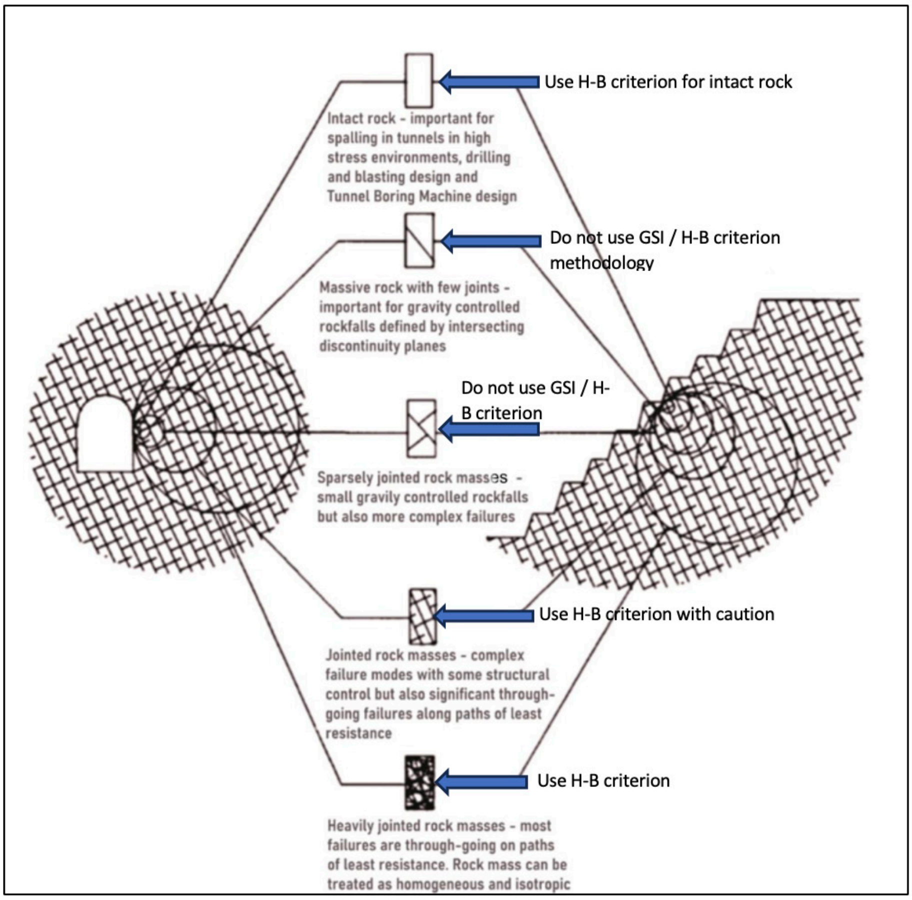


Figure 9: Idealised diagram showing the transition from intact to a heavily joined rock mass with increasing sample size (Hoek & Marinos, 2007).

characterise the entire rock mass is meaningless, as the failure is governed by the shear strength of the discontinuities. In a slope stability analysis involving a single well-defined discontinuity such as a shear zone, or fault, or schistosity, which is unfavourably oriented and governs the stability of the slope, it is appropriate to apply the Hoek-Brown criterion to the overall rock mass and to model the discontinuity as a significantly weaker element. In this case, the GSI value assigned to the rock mass should ignore the presence of the single major discontinuity. To define the properties of this discontinuity a different approach will be appropriate, such as laboratory shear testing of soft clay fillings or applying failure criteria for discontinuity surfaces (see Section 4.3).

For rock masses with a structure such as that shown in the bottom row of the GSI chart (see Figure 6), anisotropy is not a major issue, as the difference in the strength of the rock and that of the discontinuities within it is often small.

More information on the applicability and limitations of the H-B failure criterion and the GSI method can be found in Brown (2008), Marinos et al (2007) and ISRM (2014).

4.3 SHEAR STRENGTH OF DISCONTINUITIES

The shear strength along discontinuities such as bedding, joints, faults or shear zones in a rock mass is governed by the persistence of the discontinuity, roughness of discontinuity surfaces, infill material in the discontinuity, and the presence and pressure of water. Such discontinuities exhibit a wide range of shear strengths under the low effective stress levels in most slope stability problems, due to the strong influence of the natural characteristics of the discontinuity.

The shear strength of discontinuities can be measured by a combination of laboratory testing, the application of a failure criterion and simple field tests and measurements.

4.3.1 Shear strength of smooth and rough rock surfaces

For planar and smooth discontinuity surfaces, the **peak shear strength** and the normal stress can be related by the Mohr Coulomb equation:

$$\tau = c + \sigma_n \tan \varphi \qquad \text{Equation 10}$$

where τ is the shear stress required to cause displacement
σ_n is the stress normal to the discontinuity plane
c is the cohesive strength of the cemented surface
ϕ is the angle of friction

As the displacement continues, the shear stress will fall to a **residual** value and then remain constant, even for large magnitudes of shear displacement (Hoek, 2023). This residual friction angle is approximately equal to what is called in rock mechanics the basic friction angle ϕ_b, usually measured by direct shear strength tests on small specimens of saw-cut rock blocks or on rough discontinuity surfaces tested until they reach their residual strength. The residual shear stress is given by the following equation.

$$\tau_r = \sigma_n \tan \varphi_r \qquad \text{Equation 11}$$

where $\varphi_r \approx \varphi_b$

The plots of shear strength against displacement and normal stress in Figure 11 illustrate the differences between peak and residual strength.

The term cohesion adopted for rock discontinuities does not have the same physical meaning as the equivalent term in soils, where the cohesive strength is a result of the adhesion of the soil particles. The term cohesion in rock discontinuities refers to the mathematical quantity related to either cemented material within the discontinuity surface or a part of the roughness of the discontinuity surface. For a planar, clean (no infilling) discontinuity, the cohesion will be effectively zero and the shear strength will be defined solely by the friction angle.

The **peak friction angle** of a discontinuity surface consists of two components:

- The size and shape of the grains exposed on the fracture surface: a fine-grained rock, and rock with a high mica content aligned parallel to the surface, such as a phyllite, will tend to have a low friction angle, while a coarse-grained rock such as granite, will have a high friction angle (Wyllie & Mah, 2004). This component corresponds to the basic friction angle ϕ_b.
- The roughness[2] of the discontinuity surface: all natural discontinuity surfaces exhibit some degree of roughness. Surface roughness of the discontinuities consists of the irregularities and asperities (*i*) of the surface (see Figure 11). The surface roughness is a significant component of the friction angle of the discontinuity and is important for the stability of rock slopes governed by structurally controlled failures. Mathematically, the friction angle of discontinuities ϕ can be expressed as $\phi = (\phi_b + i)$.

Measurement of surface roughness of discontinuity surfaces should be an important part of surface mapping and core logging of rock formations. Guidance for the description of surface roughness is provided in NZGS (2005).

[2] Note that roughness can contribute to both cohesion and friction angle.

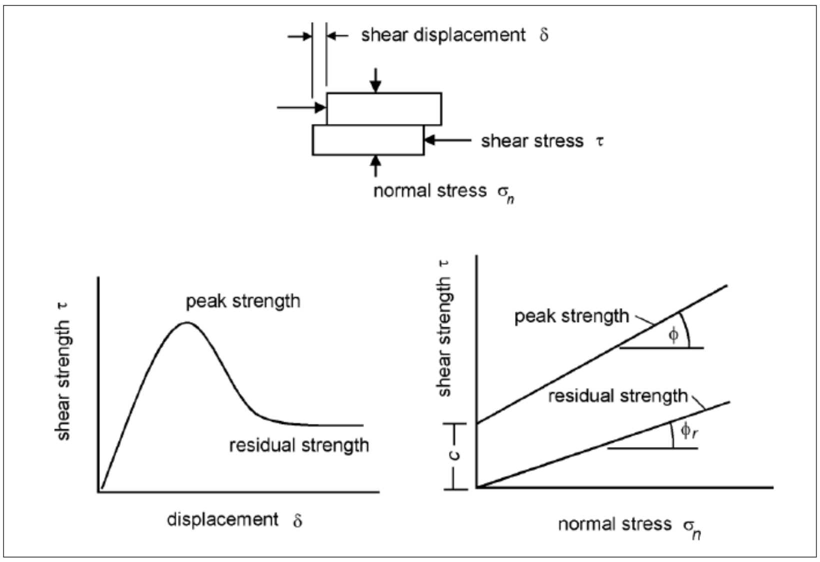


FIGURE 10: Shear strength plots from the direct shear test of discontinuities (Hoek, 2023).

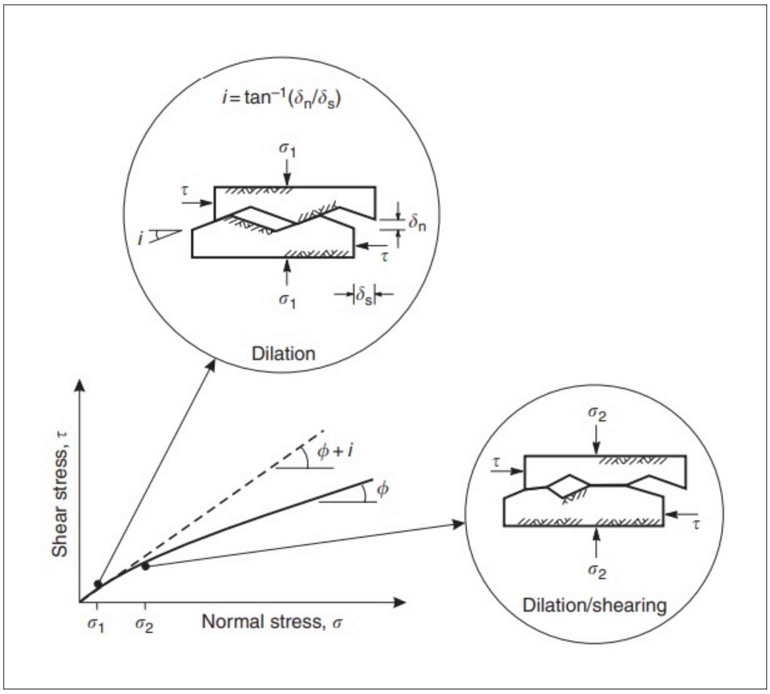


FIGURE 11: Effect of surface roughness and normal stress on the friction angle of the discontinuity surfaces (Wyllie & Mah, 2004).

4.3.2 The Barton – Bandis criterion

The shear performance of discontinuities has been quantified by the failure criterion developed by Barton & Bandis (1990), which takes into account the combined effects of the surface roughness, the rock strength at the surface, the applied normal stress and the amount of shear displacement. The criterion is expressed with the following equation:

$$\tau = \sigma_n \tan[\varphi_b + JRC \log_{10}(JCS/\sigma_n)] \qquad \text{Equation 12}$$

Where:

JRC is the joint roughness coefficient estimated in the field or borehole core, by comparing the appearance of a discontinuity surface with standard profiles published by Barton and Choubey (1977). JRC can also be estimated by a simple tilt test in which a pair of matching discontinuity surfaces are tilted until one slides on the other. The JRC value can be estimated from the tilt angle α.

JCS is the joint wall compressive strength. The compressive strength of the wall may be lower than the intact rock compressive strength, due to weathering and alternation of the walls. Methods for estimating the JCS are included in ISRM (1978). These methods are relatively simple and primarily based on field tests such as the Schmidt hammer test.

Both JRC and JCS values are influenced by scale effects, that is, as the discontinuity size increases, there is a corresponding decrease in JRC and JCS values. The reason for this is that small-scale roughness of a surface becomes less significant compared to the dimensions of the discontinuity, and eventually large-scale undulations have more significance than the roughness (Bandis, 1993; Bandis et al., 1983). The scale effect can be quantified by equations provided in Wyllie & Mah (2004) and Hoek (2023).

Hoek (2023) recommends that the most economical and practical way to define the shear strength of discontinuities is to carry out several small-scale laboratory tests to determine the basic friction angle and then apply the Barton & Bandis criterion, to allow for the roughness, conditions of the wall strength and normal stress on the discontinuity. If laboratory testing of rock discontinuities is not feasible, typical values of ϕ_b for various types of rock can be found in the literature.

Typical ranges of friction angles for a variety of rock types are given in Table 2. These values should be used as a guideline only because actual values will vary based on the site conditions. Laboratory testing should be carried out where possible, or the Barton-Bandis criterion should be used to determine the shear strength of discontinuities taking into account the local conditions.

Table 2: Typical ranges of friction angles for a variety of rock types (Wyllie & Mah, 2004)

Rock class	Friction angle range	Typical rock types
Low friction	20° – 27°	Schists (high mica content), shale, marl
Medium friction	27° – 34°	Sandstone, siltstone, chalk, gneiss, slate
High friction	34° – 40°	Basalt, granite, limestone, conglomerate

A simplified flow chart for the measurement of the shear strength of clean (no infilling) discontinuity surfaces is given in Figure 12.

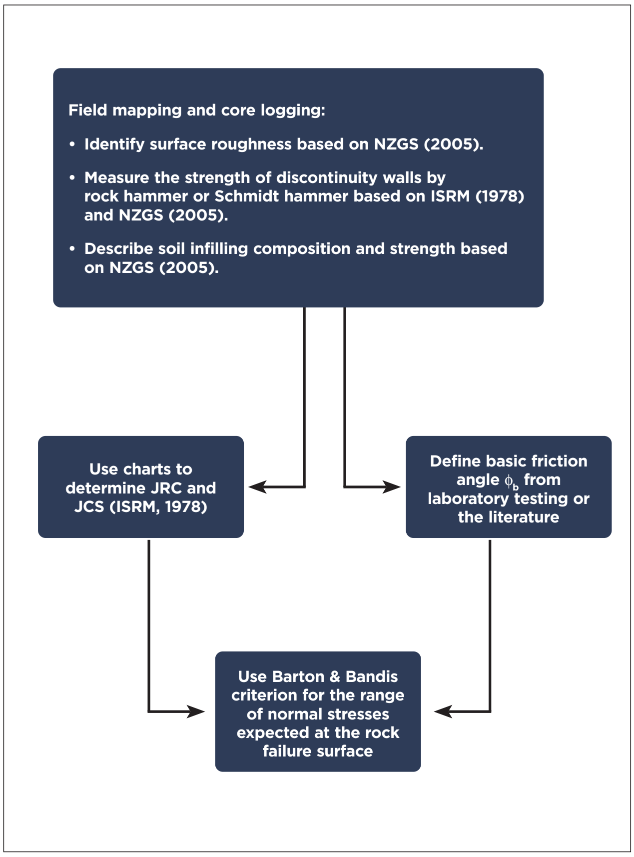


FIGURE 12: Simplified flow chart for the determination of the shear strength of discontinuity surface for clean (no infill) discontinuities.

4.3.3 Shear strength of infilled discontinuities

The presence of infillings along discontinuity surfaces can have a significant effect on stability. For example, one of the contributing factors to the massive landslide into the Vaiont Reservoir in Italy in 1963 was the presence of low shear strength clay along the bedding surfaces of the shale (Trollope, 1980).

When a discontinuity contains infilling, the shear strength properties are influenced by the thickness and properties of the infilling material. Infilling in discontinuities can range from soil materials of various thicknesses, such as clay in fault zones or gravelly silts or clays in crushed zones, or mineral coatings such as healed calcite fillings.

The thickness of the infilling plays an important role in the strength of the discontinuity. Hoek (2023) suggests that for a rough or undulating joint, the filling thickness must be greater than the amplitude of the undulations before the shear strength is reduced to that of the filling material. Goodman (1989) considered that if the thickness of the infilling is more than 25–50% of the amplitude of the asperities, there will be little or no rock-to-rock contact, and the shear strength properties of the fracture will essentially be equal to the properties of the infilling.

The strength properties of the infilling can be measured by direct shear tests in the laboratory when the infilling is thick enough to retrieve a suitable sample for testing. Hoek (2023) and Wyllie & Mah (2004) provide typical shear strength parameters of typical infilling materials from several shear strength tests that have been carried out internationally.

It is important that infillings be identified during geological mapping and logging of rock core, and that appropriate strength parameters be used in design. Methodologies for describing aperture and infilling of rock discontinuities in the field are provided in NZGS (2005).

4.3.4 Influence of displacement

Shear strength–displacement behaviour is an additional factor to consider. When analysing the stability of slopes, this will indicate whether there is likely to be a reduction in shear strength of the discontinuity surface with displacement. Where there is a significant decrease in shear strength with displacement, slope failure can occur suddenly following a small amount of movement.

For clean discontinuities, displacement of the discontinuity (failure) plane will cause shearing off of some the undulations and irregularities in the rock, thus reducing the value of surface roughness (and consequently the JRC). When analysing the stability of a structurally controlled failure mode where displacement has already occurred, this reduction in shear strength of the discontinuity surface must be taken into account.

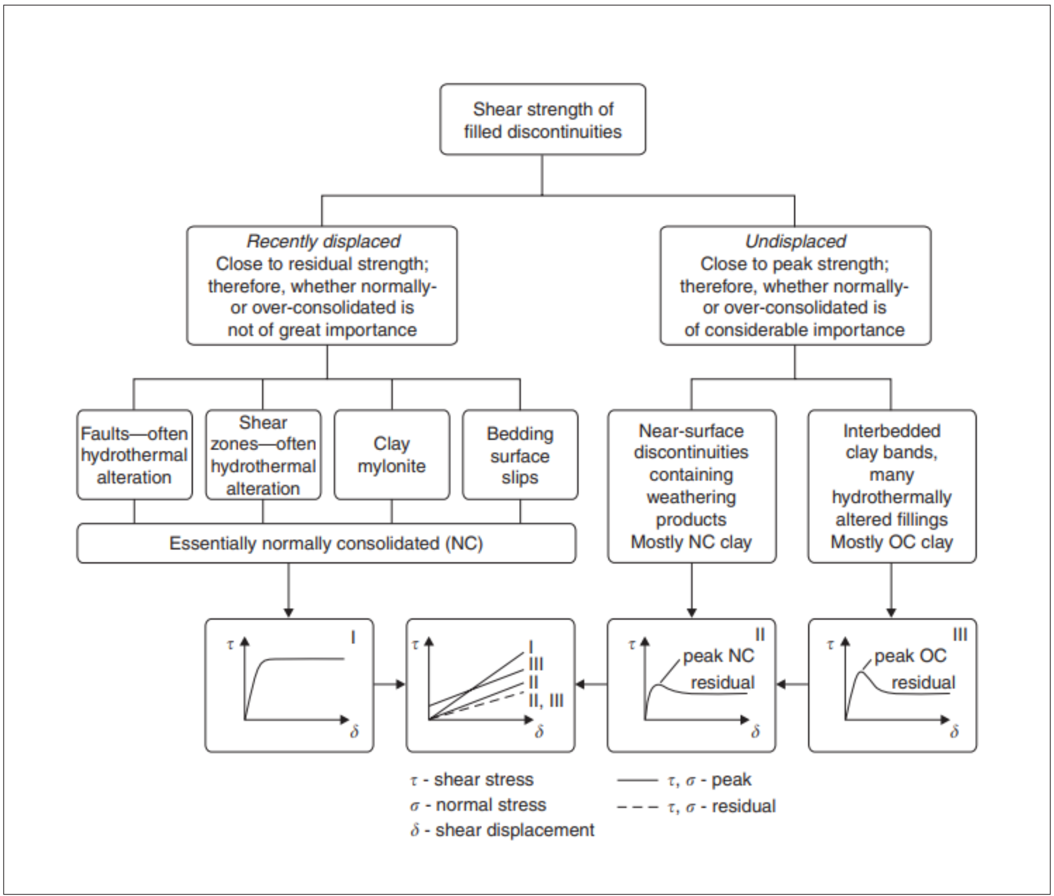


FIGURE 13: Simplified flow chart for the determination of the shear strength of discontinuity surface for filled discontinuities (Wyllie & Mah, 2004).

Filled discontinuities can be divided into two general categories, depending on whether there has been previous displacement of the discontinuity (Barton et al., 1974). These categories are further subdivided into either normally consolidated (NC) or over-consolidated (OC) materials. A simplified division of these materials is shown in Figure 13 and more discussion on this matter can be found in Wyllie & Mah (2004).

4.3.5 Influence of water

The most important influence of water in a discontinuity is the diminished shear strength resulting from the reduction of the effective normal shear stress acting on the surface. The effective normal stress is the difference between the weight of the overlying rock and the uplift pressure produced by the water pressure, and this reduction must be incorporated into the shear strength equations and the calculation of normal stress in the failure criterion.

In terms of the influence of water on the shear strength parameters of the discontinuity:

- In most hard rock and in many sandy soils and gravels, the strength properties are not significantly affected by water. The main shear strength reduction will result from the use of the effective normal stress to take into account the water pressures in the discontinuity as explained above. Therefore, the groundwater levels or seasonal flows through the discontinuities should be accurately recorded and monitored during the site investigations and appropriately considered in the slope stability analyses.
- In clays, shales, mudstones, and similar materials there will be significant reduction in shear strength with increases in moisture content. Therefore, it is important that any testing is carried out in well preserved specimens that maintain their initial moisture content. It is also important that the moisture content of the rock materials and soil infill materials is accurately described in the field description of the specimens.

5 ROCK SLOPE FAILURE MODES

As outlined in Section 3, rock slopes can be affected by a wide spectrum of failure mechanisms, due to the variability of slope geometry, lithology, internal structure, strength, loading conditions etc. If a rock slope is large and includes a range of rock types and structures, it can reasonably be expected that the slope could be affected by more than one failure mode.

Rock masses, even very weak ones, typically contain networks of discontinuities that have developed throughout their geological history. These commonly control rock mechanical behaviour at the large scale, including the strength, deformability and permeability of the rock mass. It is only under specific circumstances, e.g. at the scale smaller than the spacing of discontinuities or in weak rocks, that failure of intact material becomes significant. Consequently, the stability of rock slopes is principally controlled by discontinuities, with the effects on stability varying with the nature and extent of the discontinuities, the geometrical relationships between discontinuity planes, the properties of the intact rock between the discontinuities, the overall slope geometry, and the stresses involved.

Geotechnical investigation of the slope and development of a sound engineering geological model, particularly addressing the characteristics and geometry of discontinuities within the rock mass, are critical for identifying and analysing the feasible slope failure mechanism(s). The important factors for the development of the engineering geological model are discussed in Section 6. Analysis procedures often involve the simplification of complex rock slope behaviour into discrete mechanisms that can be analysed separately. These include:

- Toppling (overturning of rock columns);
- Planar sliding (translational block sliding on a single plane or bi-planar failure with a combination of two or more translational modes of sliding and internal shearing);
- Wedge sliding (translational block sliding on two faces simultaneously along their line of intersection);
- Complex non-planar or curvilinear failure (including rotational circular and non-circular failure surfaces), and
- Rock fall / ravelling (shallow failures from the surface of the rock slope).

There will often be uncertainty in determining the classification and mechanism of potential slope failures, in which case it may be necessary to consider multiple failure modes in the slope stability analysis. Planar sliding, wedge sliding and toppling are traditionally considered the fundamental instability mechanisms for rock slopes, and procedures for evaluating the kinematic feasibility and slope stability of these mechanisms are extensively covered in rock mechanics texts. Additional mechanisms identified above include more complex failure surface geometries (compound and non-planar sliding) and shallow ravelling or rock falls, which are also important to consider in assessing the stability of a rock slope. A brief discussion of these mechanisms is provided; for further detail the reader is referred to texts such as Hoek and Bray (1981) and Hudson & Harrison (2000).

5.1 TOPPLING

Toppling failures involve the overturning of blocks of rock and are associated with steep slopes and sub-vertical discontinuities dipping back into the slope. Two forms of toppling failure are typically observed: **Block toppling**, where the strata form rigid columns or blocks defined by discontinuity sets orthogonal to each other, and **flexural toppling**, where thin continuous steeply dipping strata, or columns, deform plastically and fail in flexure as they bend forward (Figure 14). The toppling process may start by sliding, excavation or erosion of the slope toe, with retrogression back into the rock mass, forming tension cracks.

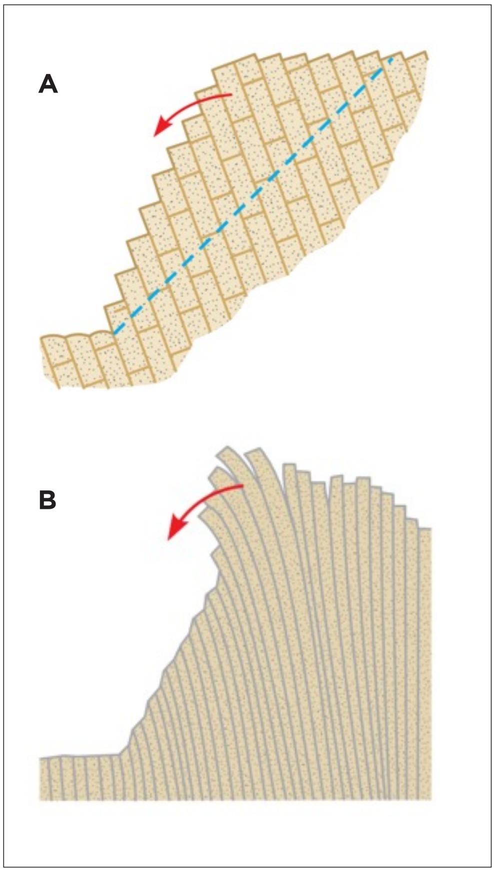


FIGURE 14. Rock topples: (A) Block toppling, (B) Flexural toppling (De Vallejo & Ferrer, 2011).

Additional modes of toppling failure have been described in response to undercutting of the toe of rock slopes, for example by erosion or anthropogenic cutting. In these cases, the primary failure involves sliding or physical breakdown of the rock, and toppling is induced in the upper part of the slope as a result. These are summarised by Goodman & Kieffer (2000) and Wyllie (2018).

5.2 PLANAR SLIDING FAILURE

Planar sliding failure takes place along a persistent discontinuity surface, such as a bedding plane, tectonic joint, fault or sheared zone. For sliding to occur, there must be discontinuities dipping approximately parallel with the slope face (i.e. the dip direction is within ±20° to 30° of the slope direction), and the failure plane must daylight on the slope face (i.e., the dip of the failure plane must be less than the angle of the slope, $\alpha < \psi$), with a dip angle greater than its friction angle ($\alpha > \varphi$, see Figure 15a). The lateral margins of the potential sliding block must also be delimited, for example by the presence of cross-cutting low strength discontinuities, or the daylighting of the sliding surface into adjacent topographic depressions such as gullies or excavations. Different types of planar failures depend on the distribution and characteristics of discontinuity sets in the slope, and the presence of tension cracks and groundwater pressure acting on the failure plane (Figure 15b).

In bedded, foliated or highly jointed rocks where the length of discontinuities is less than the height of the slope, translational sliding failure can occur along stepped or irregular surfaces (Figure 16a). These failures involve sliding on outward-dipping discontinuities such as bedding or schistosity with shear or tensile release along steeper secondary discontinuities such as joints. Secondary discontinuities of this type are common in highly fractured rock masses such as the greywacke bedrock formations that underly much of New Zealand.

More complex forms of failure include **bi-planar sliding**, where sliding occurs on two or more sets of persistent planar discontinuities and is accompanied by internal deformation of the sliding mass.
These failures typically occur in deformed rock formations (e.g. folded sedimentary rocks) or anisotropic rock masses intersected by major structures (Glastonbury & Fell, 2000). Kinematic release occurs on persistent discontinuities in the upper part of the failure, with shear failure through the rock mass or along a persistent discontinuity at the base (Figure 16b).

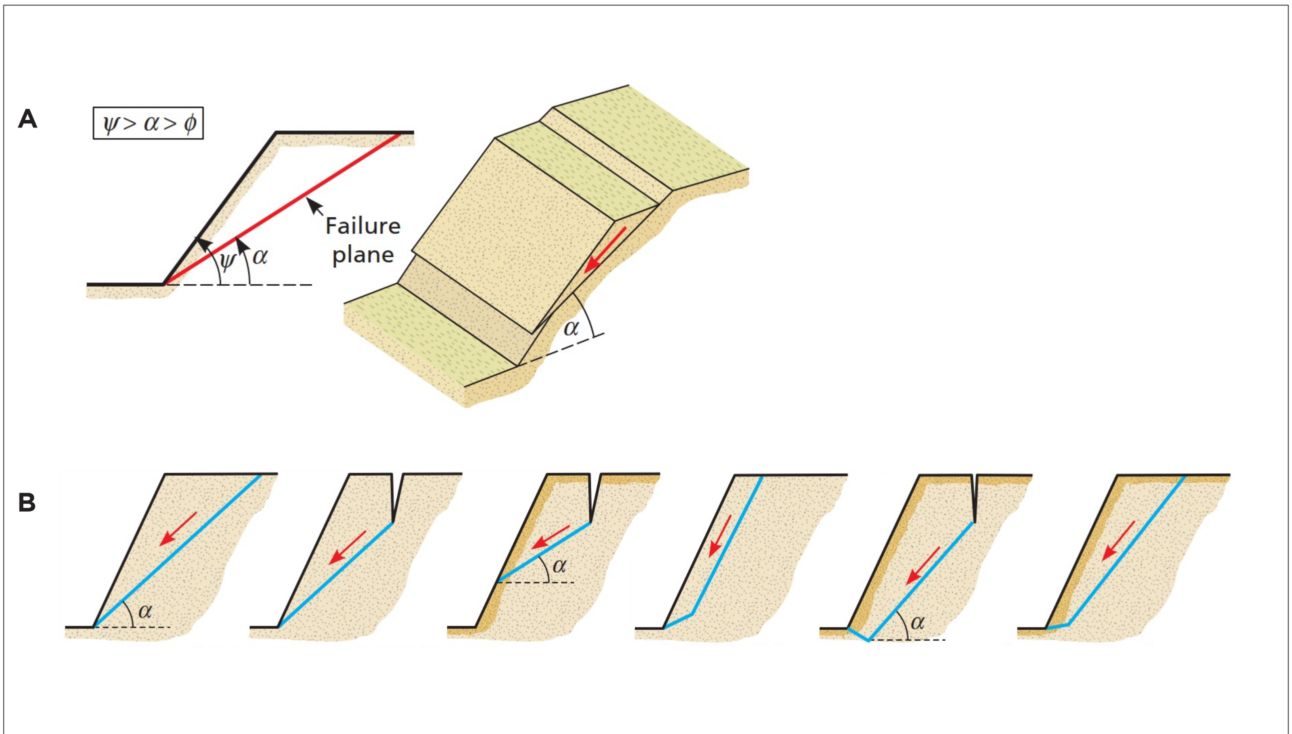


FIGURE 15. (A) Conditions for planar failure, (B) Types of planar failure (De Vallejo & Ferrer, 2011).

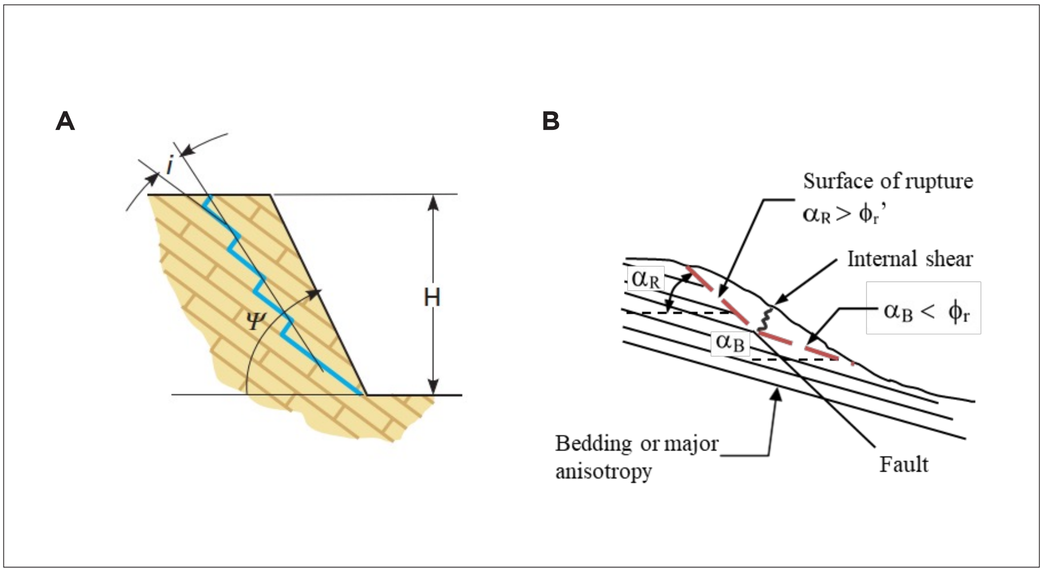


FIGURE 16. (A) Stepped failure surface in blocky rock mass (Gonzalez and Ferrer, 2011).
(B) Bi-planar composite slide (Fell et al., 2007)

5.3 WEDGE SLIDING

Wedge sliding failures consist of a wedge-shaped block formed by two planar discontinuities that slides outwards towards the slope face along the line of intersection between the two discontinuities. This type of failure usually occurs in rock masses with several sets of discontinuities, and their orientation, spacing and persistence will determine the shape and volume of the wedge. Comparing the angles of the slope, the line of intersection of the limbs of the wedge and the friction angles of the planes determines whether movement is kinematically feasible and if the wedge is stable or unstable. The condition for movement to occur is that the two planes and the line of intersection of the wedge daylight on the slope surface, i.e. $\psi > \alpha > \varphi$, where α is the plunge of the line of intersection (Figure 17).

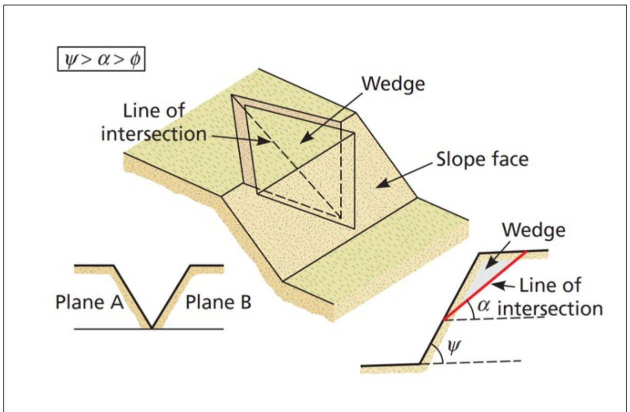


FIGURE 17. Conditions for wedge failure (Gonzalez & Ferrer, 2011)

5.4 COMPOSITE / NON-PLANAR FAILURE

Composite failures consist of a combination of sliding along discontinuities and shear or tensile failure through intact rock material. These are typically **non-planar** failures, with irregular, non-circular (curvilinear) and circular failure surfaces. These occur in structureless overburden material, highly weathered or very weak rock masses, or heavily jointed or broken rock masses in high slopes. The behaviour of these materials typically is not controlled by individual sets of persistent discontinuities; however, failure can occur by a combination of sliding along existing joints and failure through intact but weak material (Figure 18). In anisotropic rock masses such as laminated sedimentary rocks or schists the shape of the failure surface can become elongated parallel to the anisotropy. Circular failures may occur in rock masses which are intensely fractured in relation to the scale of the slope so they may be considered as randomly jointed and therefore isotropic.

FIGURE 18. Non-planar failure with irregular failure surface in heavily jointed rock mass (Gonzalez & Ferrer, 2011)

5.5 SHALLOW RAVELLING FAILURES AND ROCK FALLS

Ravelling is a process of mass wasting that consists of progressive loosening and erosion of cobbles or blocks from the surface of the rock slope under active weathering and gravity transport. This typically occurs in poorly cemented sedimentary rocks, very highly fractured hard rocks, and layered rock masses. Release of the rock materials from the slope may be controlled by the pattern of discontinuities and/or material deterioration, and is often triggered by rainfall, earthquakes, or vegetation jacking. Slopes steepened by erosion, undercutting or anthropogenic modification (i.e., cutting) are prone to ravelling of the surficial materials in response to stress release and physical weathering of the newly exposed rock.

Rock falls involve detachment and rapid movement of rock fragments from steep rock slopes. The initial detachment can occur by a range of mechanisms (e.g., tensile failure, sliding or toppling), while the movement occurs by free fall, bouncing, and rolling. The falls may be triggered by earthquake, rainfall, root-wedging or freeze-thaw, or they may occur without any obvious trigger. For discussion of the analysis and mitigation of rock fall hazards, see Unit 1 and Unit 5.

Figure 19 below illustrates the sequence of events that may occur as rockfall debris moves down a slope.

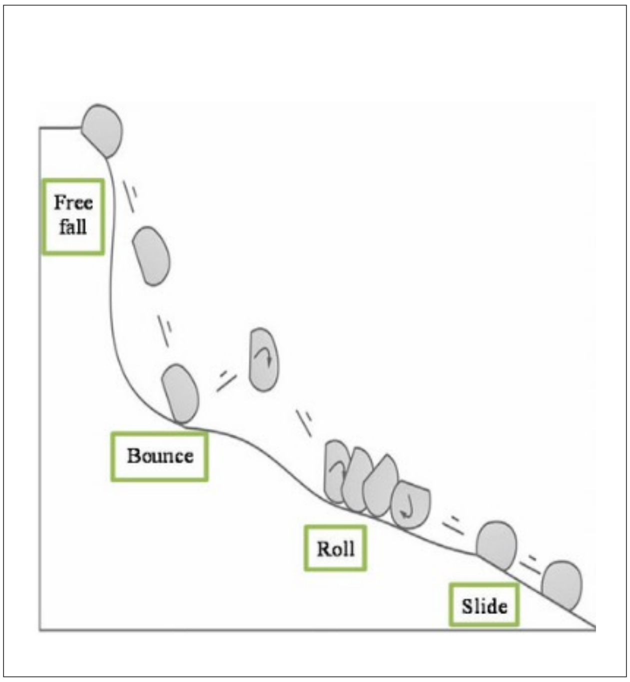


FIGURE 19: Motion mechanisms of a typical rockfall from (Yan, Zhang, & Fanq, 2020)

6 ENGINEERING GEOLOGICAL MODELS FOR ROCK SLOPES

Part 5 of Unit 1 notes that for any landslide assessment, the Engineering Geological Model (EGM) represents the best interpretation of the surface and subsurface conditions and forms the basis for hazard assessment and slope stability modelling, and provides guidance on model development.

Sound engineering geological models are important for rock slope stability assessments. A comprehensive engineering geological model for rock slopes should include the following main inputs:

- Lithology (rock type or types),
- Rock mass characteristics,
- Structure (discontinuities),
- State of weathering and deterioration of rock, rock characteristics in their unweathered state (if known).
- Hydrogeological conditions.

The geomorphology of the slope surroundings also contributes to the development of a sound engineering geological model. For example, potential cracking on the ground above the slope, steep scarps on slope faces and other geomorphological features may indicate initiated slope instability. Guidance on geomorphological mapping can be found in Unit 2.

In New Zealand, understanding the seismicity at the area of the rock slope is necessary to define the relevant seismic actions affecting rock slope stability.

Field descriptions of rock masses and discontinuities enable the design of rock slopes with a minimum of expensive in situ testing (ISRM, 1978). Refer to Unit 2 for the appropriate methods and techniques of geological mapping and investigation.

6.1 LITHOLOGY

Lithology refers to the type (or types) of rock found on the slope. Due to the different nature and origin of the different rock types, their inherent geological features are also different. Furthermore, the properties of the same type rocks may differ between areas depending on the geological history of the rock, its geographical location and the geomorphological characteristics of the area.

For example, in softer or weaker rocks such as the Tertiary rocks in New Zealand, the intact rock can be the predominant controlling factor of slope instability. In hard, indurated rocks such as Torlesse Greywacke and Otago Schist, the major discontinuities are expected to control the stability. Limestones may have karst features along discontinuities, which may also trigger failures, particularly in steep cliff slopes.

Geological science terms that are used to describe rocks often have no direct significance to engineering characteristics. The name of the rock can broadly indicate the range of engineering properties to be expected. For example, engineers readily appreciate that there are clear differences between the likely engineering properties of greywacke and limestone. However, the engineering geologist must ensure that the engineer is provided with a full understanding of the rock and the rock mass before undertaking stability analysis or designing slopes or remedial works because not all limestones or greywackes (for example) have the same properties.

6.2 DISCONTINUITIES

The most important factor controlling the stability of slopes in jointed rock masses is the presence of discontinuities such as bedding, faults, shear zones, joints, schistosity etc, particularly when they are adversely oriented with respect to the orientation of natural or cut slopes. Depending on the slope height, rock slope failures may involve several discontinuity-controlled and composite mechanisms as discussed in Section 5.

Understanding the characteristics of discontinuities is important for the assessment of the stability of an existing rock slope, and for the design of a new slope or mitigation measures on an unstable rock slope. Knowing the discontinuity characteristics such as orientation, spacing, persistence, roughness, infilling etc is critical for the appropriate estimation of rock mass and rock discontinuity parameters. Limited or unrealistic assessment of discontinuity characteristics, particularly of orientation, spacing and persistence may result in inappropriate modes of failure and block sizes being used in analyses, and consequently cause unrealistic engineering geological models to be established (Ulusay, 2019).

It is best to measure most of the characteristics of critical geological structures from surface exposures during geological mapping. Today geological and geotechnical data collection techniques are well developed, and different techniques can be used for the mapping of rock discontinuities, such as scan-line survey, window mapping, photogrammetric method or laser scanning techniques. Guidance on how to collect discontinuity characteristics during field mapping and rock core logging can be found in NZGS (2005) and Unit 2. Discontinuity orientations in boreholes can also be collected by down-hole geophysical surveys using the Optical and Acoustic Televiewer methods (OTV and ATV).

The collected raw discontinuity data from the site investigations must be statistically analysed with the use of stereonets to identify mean sets, if applicable, and consequently for carrying out kinematic analysis to identify the discontinuities and sets that are critical for the stability of the slope.

6.3 WEATHERING

Rocks are subject to weathering and alteration when exposed to atmospheric conditions and/or hydrothermal fluids circulating through the rock mass. Due to physical and chemical weathering processes, the strength of the rock may deteriorate significantly. Weathering and alteration influence the rock discontinuities in terms of their wall strength, spacing, aperture, and the presence and type of infill material. Understanding and recording the degree of weathering and alteration of the rock is essential for assessing the parameters of intact rock, rock masses, and discontinuity surfaces.

Superimposed on the lithology and structures, physical and chemical weathering effects can be dominant in controlling the modes of rock slope failure (Ulusay, 2019). Some examples where the weathering of the rock mass can generate slope instability in New Zealand are:

- Surface physical weathering of soft Tertiary rocks exposed on steep and near vertical slopes and cliffs. Weathering weakens the rock material and may also exacerbate the initiation and development of stress relief joints, which tend to be sub-parallel to the slope face and result in structurally controlled instabilities, such as toppling or planar failure (Figure 20A), colloquially called “slabbing”.
- Differential weathering of interlayered strong and weaker rock types, such as the alternating sandstone, siltstone and mudstone layers of Tertiary rocks. Higher degrees of weathering of the weaker materials can cause undermining and loss of support of the stronger materials causing falls or structurally controlled failures (see Figure 20B).
- Intense weathering of rock masses to highly - completely weathered grades, transforming rocks into a soil or soil-like material. The slope may fail in the form of shallow circular sliding as commonly observed in soil slopes. This mode of failure is common in highly to completely weathered Wellington Greywacke rock (see Figure 20C).

The spatial distribution of the different weathering grades of both the rock material and rock mass in a slope must be recorded both during the field mapping and rock core logging. The weathering grade of the rock material must be considered in the scheduling of in situ and laboratory testing. For example, UCS testing of intact rock samples of different weathering grades may have a significant scatter in the results. If the weathering grade of the different samples is not identified, this may lead to under- or over-estimation of rock mass parameters. The same applies for the field testing of rock strength or discontinuity wall strength using the geological hammer or the Schmidt hammer. The spatial distribution of the different weathering grades must be identified in plan and section, along or in depth in a slope, and the rock mass parameters must be differentiated taking the different rock material weathering grades into account.

A

B

C

FIGURE 20. (A) Slabbing of late Tertiary age siltstone in a subvertical bluff in the lower Whangaehu River valley. (B) Erosion of weak Tertiary mudstone undermining more competent sandstone beds, Buller River. (C) Circular failure of a road cut slope in completely weathered greywacke, Wellington.

There are several methods and scales for describing rock material and rock mass weathering internationally, examples include ISRM (1978), BS 5930:2015 and AS 1726:2017. In New Zealand the rock mass weathering is described based on NZGS (2005). If different methods of describing weathering are used, these must be clearly referenced in the borehole log or reporting of geological mapping results.

6.4 GROUNDWATER

As in every slope, the presence of groundwater plays an important part in the stability of rock slopes. Groundwater flow and circulation in fractured rock masses primarily occur along the discontinuities, because of the generally low conductivity of most intact rocks, other than porous rocks. The conductivity of rock masses will be influenced by the characteristics of the discontinuities.

In New Zealand where the rainfall levels are high, it is important that the groundwater conditions are clearly identified in the engineering geological model of the slope, including the type and depth of the groundwater table and its seasonal fluctuations. It is water pressure, not rate of flow, that is responsible for instability in slopes and it is essential that measurement or calculation of this water pressure forms part of site investigations for rock slope stability studies (Wyllie & Mah, 2004).

It may be a mistake to assume that ground water is not present within the slope just because no seepage appears on the slope face. The seepage rate may be lower than the evaporation rate, and hence the slope surface may appear dry and yet there may be water at significant pressure within the rock mass.

7 METHODS OF ROCK STABILITY ANALYSIS

This section presents the widely used methods of analysis for rock slopes. The modes of failures predominantly covered are the first four described in Section 5, i.e. toppling, planar, wedge, and composite / non-planar. Methods of modelling, analysis and design of protection structures for rockfall failures are covered in the MBIE Passive Protection Structures Guidelines (MBIE, 2016) and the NZ Transport Agency Rockfall Protection Structures Design Guidance (NZTA, 2023).

For all but very weak rock materials, the analysis of rock slope stability is fundamentally a two-part process. The first step is to analyse the structural data from the site to determine whether the orientation of the discontinuities could result in instability of the slope under consideration. This determination is usually accomplished by means of stereographic analysis of the structural data and is referred to as kinematic analysis (as further discussed in Section 7.1.2).

If a kinematically possible failure mode is present, the second step requires a limit-equilibrium stability analysis to compare the forces resisting failure with the forces causing failure. The ratio between these two sets of forces is the Factor of Safety (FoS).

For very weak rock where the intact material strength is of the same magnitude as the induced stresses, the discontinuities may not control stability, and classical soil mechanics principles for slope stability analysis will apply. These procedures are discussed in Section 7.2.3.

7.1 IDENTIFICATION OF MODES OF FAILURE

7.1.1 Study of precedent behaviour of slopes

Study of existing slopes near the slope of interest, as part of the geological mapping and site investigations, will provide insights into the general performance of slopes and stable and unstable slope angles. It will provide information on the frequent, systematic and predominant modes of failure, the critical discontinuities that may contribute to instability and the characteristics of the failures.

Even small failures on the slope, that may appear insignificant at the time of mapping, should be recorded and evaluated, as they can be indicative of bigger failures that are kinematically possible and could occur given certain adverse conditions. In combination with the study of historical imagery if available and a geomorphological assessment, the study of existing slopes may also provide insights into the conditions of failure.

This study is especially important if the designer does not have experience in the design of slopes in a particular geological formation and geographical area. The observations of the study should be used to calibrate the various considerations of the slope stability analysis and provide guidance on a possible slope angle appropriate for the specific area and geological formation.

7.1.2 Kinematic analysis

Kinematic analysis of discontinuity data using a stereonet can identify potential failure types within slopes, but more rigorous methods are needed for a robust stability analysis. This is because kinematic analysis is based on geometric relationships, assumes continuous/persistent controlling discontinuities, only considers the component of friction in the shear strength of the discontinuities and ignores cohesion, and does not allow for groundwater conditions or external loads such as earthquakes.

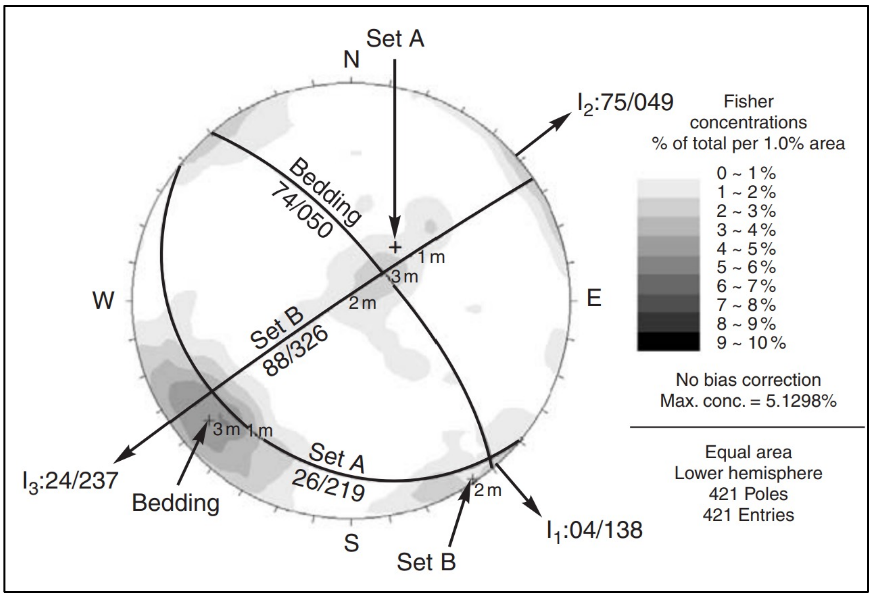


FIGURE 21: Contoured stereonet diagram and great circles of mean discontinuity sets (Wyllie & Mah, 2004)

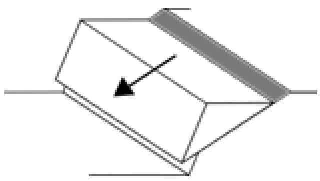
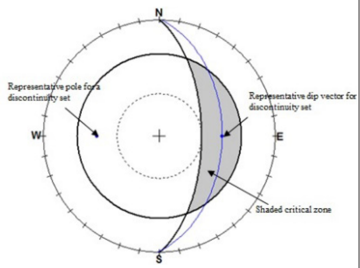
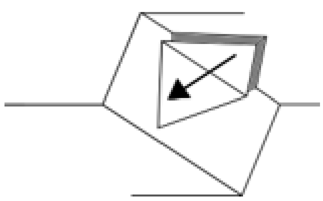
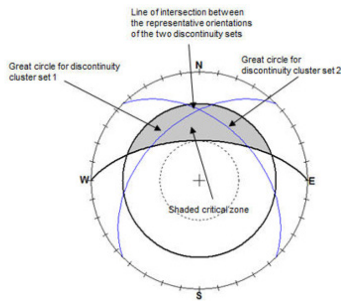
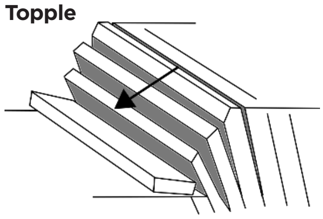
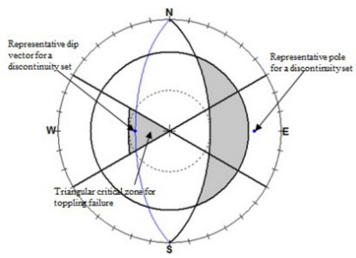
Furthermore, kinematic analysis is based on mean orientations of discontinuity sets. Local variability may allow small scale failures to occur when considering all individual discontinuities and variations in the slope geometry rather than just the mean orientation (planes/poles) of the major discontinuity sets.

An initial assessment of the kinematically possible mechanisms of failure can be carried out using the stereographic projection of the structural geology data collected from site investigations on a stereonet diagram. A typical stereonet diagram with pole concentration contours used for statistical analysis of the mean sets, and with the great circles of the mean sets, is shown in Figure 21. Simplified geometrical conditions denoting the three structural modes of failure are shown in Table 3. In rock slopes with many discontinuity sets, multiple modes of failure created by different combinations of discontinuity sets may be present.

The kinematic analysis is an initial, quick and easy method that provides insights into the possible mechanisms of failure and direction for the next steps of analysis. It enables an initial assessment of the slope angle that will avoid the predominant and systematic structurally controlled modes of failure. This analysis, however, only considers the kinematic potential for a specific mechanism of failure and has limitations as explained above. Further analytical assessment with the detailed methods presented in the following sections is required to refine the initial slope angle defined from the kinematic analysis and assess the stability of the remaining kinematically possible failure modes considering all the factors affecting stability.

Instruction on how to plot and statistically analyse structural data on a stereonet diagram and how to use kinematic analysis to carry out the initial slope stability assessment is presented by Wyllie & Mah (2004). Detailed description of investigation methods for collecting representative structural geological data of the rock mass on a slope is provided in Unit 2.

Table 3: Criteria for discontinuity-controlled failures and kinematic analysis using a stereonet.

Failure Mode	Criteria	Stereonet	Comments
Planar 	Plane failure may occur when a discontinuity dips in the same direction (within 20° - 30°) as the slope face, at an angle gentler than the slope angle but greater than the friction angle along the failure plane. Lateral release of the sliding block through discontinuities or topography must be present. NOTE: Plane failure may involve stepped surfaces.		If the dip vector (middle point of the great circle) of the great circle representing a discontinuity set falls within the shaded area (area where the friction angle is higher than slope angle), the potential for a plane failure exists
Wedge 	Wedge failure may occur when the line of intersection of two discontinuities, forming the block, plunges in the same direction as the slope face and the plunge angle is less than the slope angle but greater than the friction angle along the planes of failure		If the intersection of two great circles representing discontinuities falls within the shaded area (area where the friction angle is higher than slope angle), the potential for a wedge failure exists.
Topple 	Toppling failure may result when a steeply dipping discontinuity is parallel to the slope face (within 30°) and dips into it. NOTE: There are several possible mechanisms for toppling		The potential for a toppling failure exists if dip vector (middle point of the great circle) falls in the triangular shaded zone.

7.2 ROCK SLOPE DESIGN CHARTS

Simplified slope stability charts are available in the literature and can be used for preliminary assessment of the factor of safety of wedge, toppling and circular modes of failure identified from the kinematic analysis in the initial stages of slope design. These charts may be handy at the initial stages of a project, especially when it includes the design of multiple slopes.

7.2.1 Wedge failure

A rapid check of stability of the slope against identified kinematically possible wedge failures can be carried out using a series of charts presented by Wyllie & Mah (2004). These charts examine wedge failures against static conditions only and assume that the slope is drained, while the sliding planes have friction only and zero cohesion. Wyllie & Mah indicate that a wedge having a factor of safety in excess of 2.0 (obtained from the wedge stability charts) is unlikely to fail under even the most adverse conditions.

In the preliminary analysis of a project involving multiple slopes, the friction-only wedge stability charts can be used for identifying those slopes that are stable or prone to wedge failures, based on the slope angle, orientation of slopes and discontinuities and the friction angle of the discontinuities. No further analysis will be required in subsequent stages of design for those slopes that present a factor of safety higher than 2.0 against wedge failure. Slopes with a factor of safety of less than 2.0, using the friction-only charts, must be regarded as potentially unstable against wedge failure and require further detailed examination.

The wedge stability charts can be also used when the observational method is implemented during construction. The charts enable a rapid check of stability conditions as the slope faces are being mapped while the excavation is proceeding, and decisions are required on the adequacy of the selected slope angle in the design and need for support. If the factor of safety is evaluated as less than 2.0 using the charts, more detailed analysis is required considering all loading conditions.

7.2.2 Toppling failure

Slope design charts for stability analysis of flexural toppling failure have been developed from Adhikary et al (1997) that relate stability to the slope angle, the dip of the blocks into the face and the ratio of the slope height to the width of the slabs. Another input parameter is the tensile strength of the rock, because bending of the slabs induces tensile cracking in their upper face. These design charts can be used for the preliminary design of the slope, as they provide the allowable face angle for specific geological conditions and slope height.

7.2.3 Circular failure

In the cases of weak and of highly fractured rock masses with randomly orientated discontinuities the anticipated mode of failure can be approximated with a circular failure surface.

A series of slope stability charts for rock slopes and the detailed methodology for their use are presented in Wyllie & Mah (2004). These charts enable a rapid check of the factor of safety of a slope, or of the sensitivity of the factor of safety to changes in ground water conditions, slope angle and material strength properties. These charts should only be used for the analysis of circular failure in rock materials that are homogenous and where the conditions apply that were assumed in deriving the charts. The applicability of these charts is for materials that can be appropriately analysed using the H-B criterion as discussed in Section 4.2.

The charts for rock materials presented by Wyllie & Mah (2004) correspond to the lower bound solution for the factor of safety, obtained by assuming that the normal load is concentrated on a single point on the slide surface. These charts differ from those published for soil (e.g. by Taylor, 1937 – see Section 14) in that they include the influence of a critical tension crack and of ground water.

These charts are useful for initially identifying the depth and shape of potential slides and for estimating the friction angle when back-analysing existing circular slides.

7.3 LIMIT EQUILIBRIUM METHODS (LEM)

7.3.1 Structurally controlled failures

The LEM is the most commonly used method for analysing the stability of a rock slope against structurally controlled failure modes, such as planar, wedge and toppling in competent rock masses, where failure of massive blocks of rock along one or a combination of discontinuity surfaces is possible, or for analysis of rock slopes at a smaller scale as explained in Section 3 (e.g. between slope benches).

For each of the structurally controlled modes of failure, LE equations and analytical methodologies have been developed over the years and are available in various references including Hoek & Bray (1981) and Wyllie & Mah (2004). More recent publications have attempted to also incorporate dynamic loading (Ghosh & Haupt, 1989 and Kumsar, et al., 2000). Practical and commercial software (such as Slide2 and SLOPE/W) has been developed for the LEM analysis of structurally controlled instabilities.

The inputs required for the analysis of structurally controlled failures using LEM are outlined below. These should be defined during the site investigations of a rock slope (refer to Unit 2, and Section 4 and Section 6 of this Unit).

- Identification of the geometry and depth of the critical failure plane(s), parameters which define the size of failure.
- Presence and location of back or side release surface and tension cracks.
- Inclination of the slope immediately above the structurally controlled failures, especially for planar and wedge failures.
- Infill materials and water pressures in the discontinuities and tension crack.
- Shear strength of discontinuities.

7.3.2 Failure through rock mass

In the case of closely fractured rock with randomly oriented discontinuities, the sliding surface is free to find the path of least resistance through the slope, which may consist of both pre-existing or incipient discontinuities and failure through the rock mass. In weak and highly weathered rocks failure through the rock mass, without the contribution of discontinuities, is possible. In both cases, observations of slope slides have shown that the mode of failure can be generally approximated with a circular surface, which can be modelled and examined in two-dimensional slope stability analysis (Wyllie & Mah, 2004). The circular mode of failure needs to be examined in addition to the structurally controlled patterns of failure when kinematic potential exists.

The characteristics of a circular failure in rock are similar to those for a classical rotational failure in soil. A circular sliding surface in a homogenous weak or highly weathered or highly fractured rock mass is likely to be a shallow, large radius surface extending from a tension crack close behind the crest (see Figure 22a and b). In New Zealand, circular modes of failure are likely to be encountered in weak rock masses such as Tertiary rocks, highly to completely weathered or highly fractured Wellington Greywacke and highly to completely weathered East Coast Bays Formation.

The shape of the sliding surface may be influenced in some cases by the geometry of the rock discontinuities, contributing to the formation of the failure plane. Figure 22c, d and Figure 23 show examples where the shape of the sliding surface is modified by the structural geology. Stability analyses in such cases can be carried out using noncircular sliding surfaces that are incorporated in

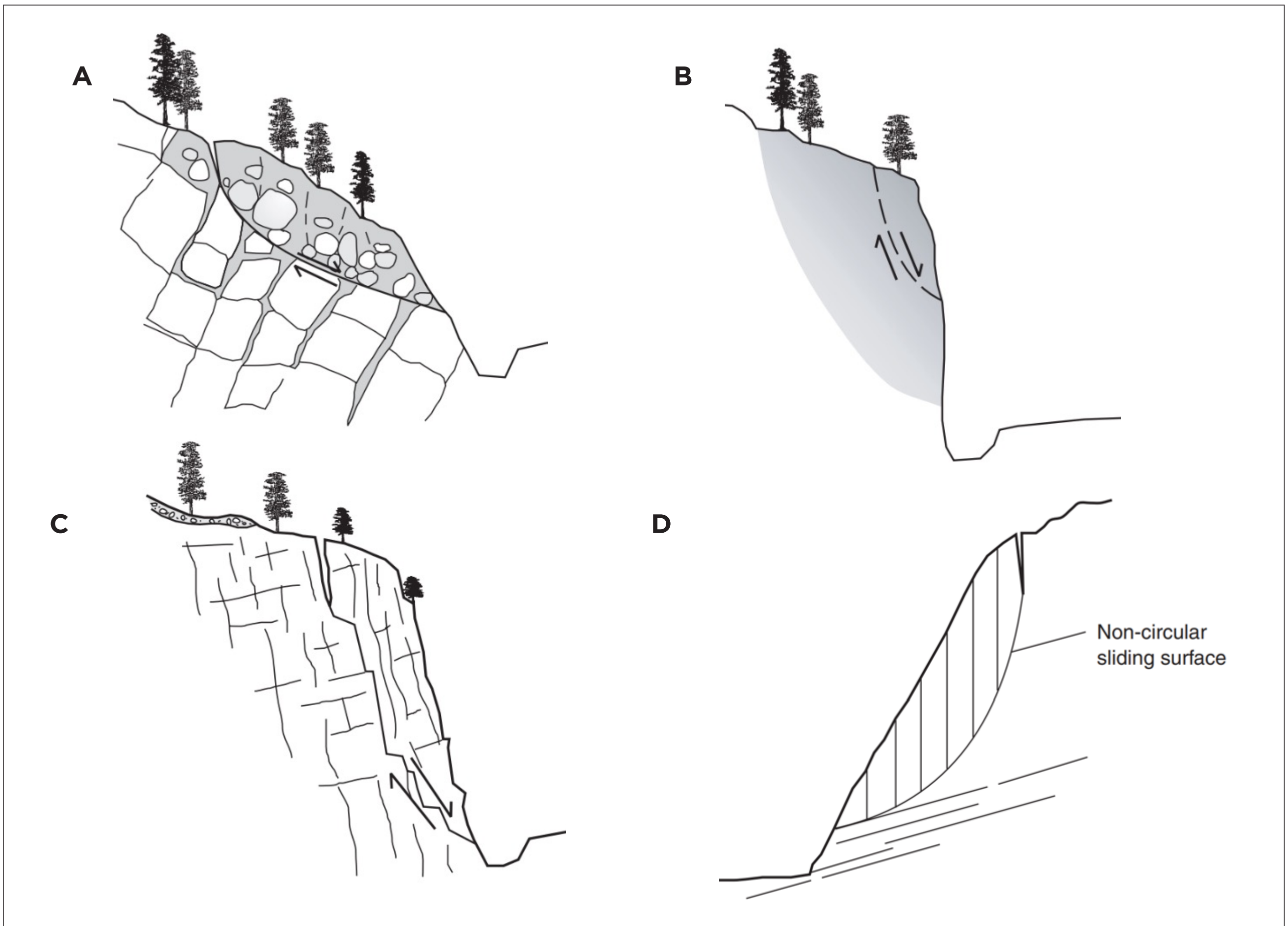


FIGURE 22: (a) Large radius circular failure in residual soil and weathered rock, (b) shallow circular failure in residual soil or weak rock not controlled by discontinuities (c) & (d) composite noncircular failure influenced by structural geology (Wyllie & Mah, 2004).

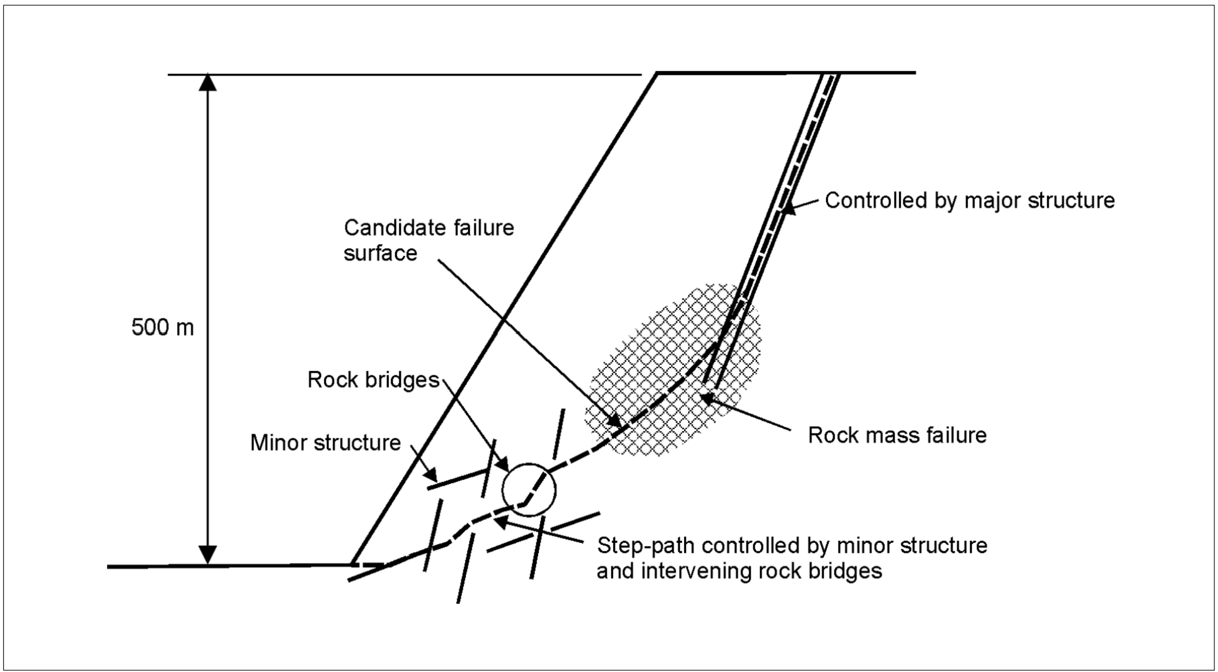


FIGURE 23: Composite noncircular failure surface partially influenced by an existing discontinuity. It also includes failure through the rock mass and step-path failures controlled by minor or incipient structures and rock bridges (Hoek, 2023).

modern 2D LEM slope stability analysis software using a user-defined noncircular sliding surface (and a block, path or polyline search) and a method of analysis that satisfies moment and force equilibrium, such as the Spencer or GLE methods. The Janbu methods provide a good lower bound for noncircular surfaces, while Bishop is not generally recommended for analysis of noncircular surfaces (Rocscience Inc., n.d.).

7.4 NUMERICAL METHODS

Numerical modelling techniques have been developed to provide approximate solutions to more complex rock slope stability problems. Numerical methods for rock slopes consist of the following approaches:

- Continuum modelling
- Discontinuum modelling
- Hybrid modelling

Continuum modelling is best suited for the analysis of slopes that comprise massive weak rocks and highly fractured rock masses with randomly oriented discontinuities. The techniques that can be used are Finite Element (FEM) and Finite Difference (FDM) methods - see Figure 24. Some modern continuum numerical analyses software includes tools to incorporate discrete rock features, such as bedding and shear planes or faults. The salient advantages and disadvantages of continuum methods are discussed by Hoek et al (1993), with more recent advances described in Hoek (2023).

Two-dimensional continuum codes assume plain strain conditions, which are frequently not valid in inhomogeneous rock slopes with varying structure, lithology and topography. Recent 3-D continuum codes, such as FLAC3D, enable 3-D analyses of rock slopes and model complexities such as changes in geology, topography, and pore water pressures, and consider in situ stresses and dynamic loading.

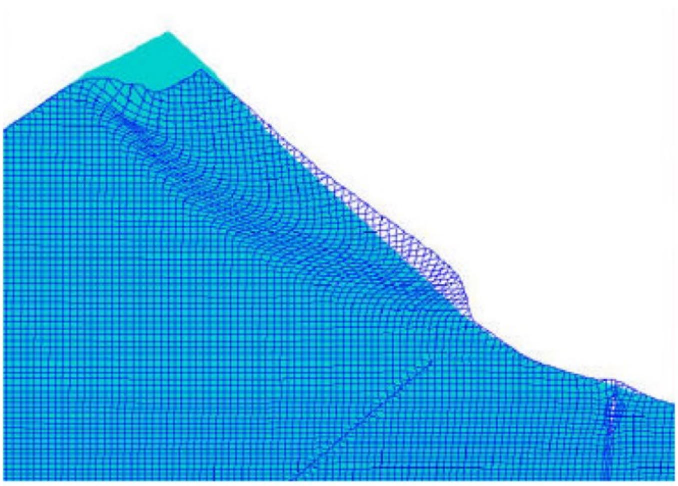


FIGURE 24: Finite difference model showing large-strain failure of a rock slope modelled with an elastoplastic constitutive model based on a Mohr Coulomb yield criterion (Eberhardt, 2003).

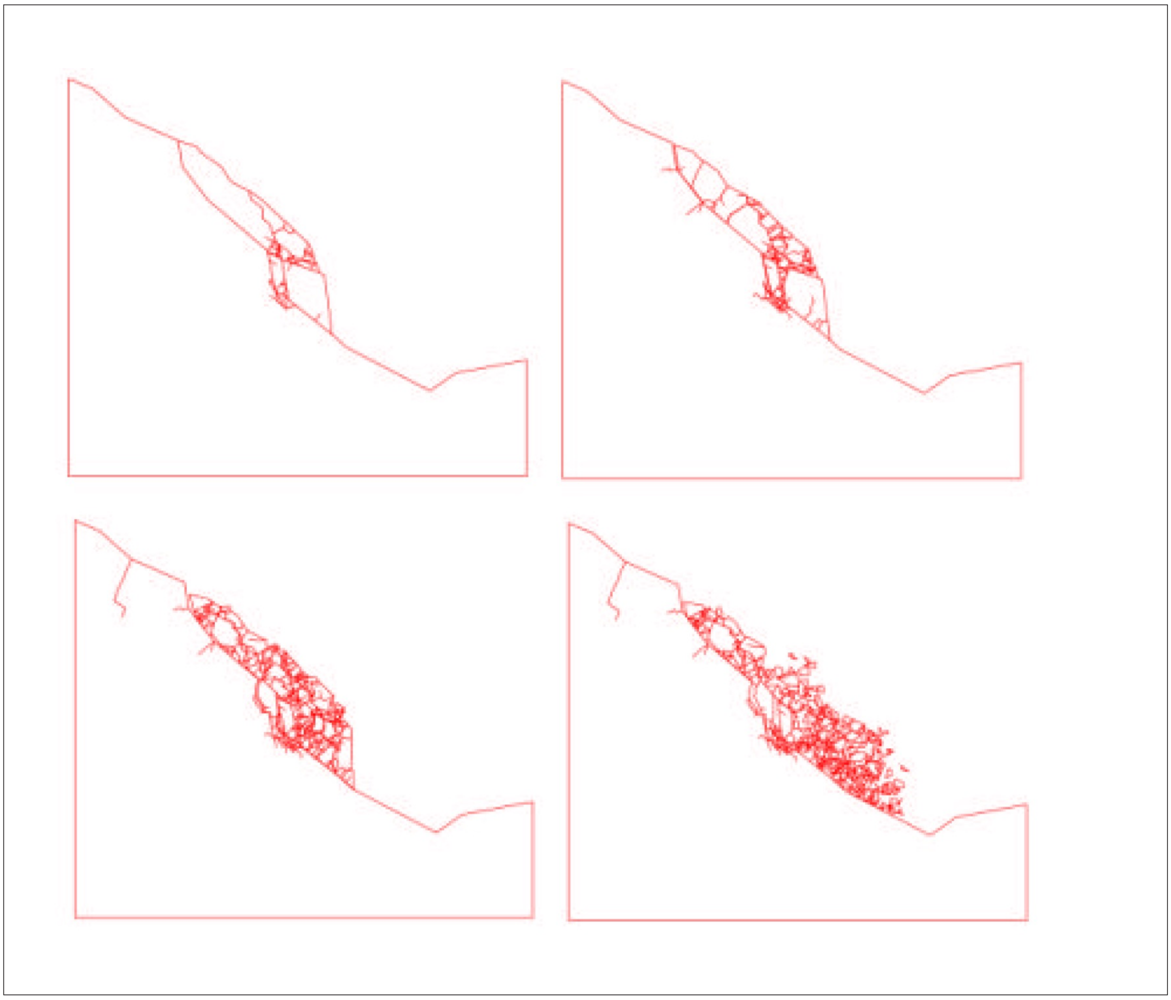


FIGURE 25: Hybrid finite-/distinct element analysis of a rockslide showing several progressive stages of brittle failure (Eberhardt, et al., 2002).

Discontinuum models are useful in analysing modes of failures that are influenced or controlled by rock discontinuities. Discontinuum methods consider an assemblage of distinct interacting bodies or blocks that are subjected to external loads and expected to undergo significant motion with time. These methodologies are collectively referred to as the Discrete Element Method (DEM). DEM allows sliding along the blocks and for complex non-linear interaction between the blocks. Variations to the DEM are discussed by Eberhardt (2003).

Hybrid approaches can also be used for rock slope stability analysis. These may include combined analyses using LEM for stability and FEM for groundwater flow and stress analysis, as is adopted by some commercial software. Hybrid analysis may consist of coupled finite-/distinct element analysis; commercial codes are available that incorporate adaptive remeshing and can analyse complex failure mechanisms that involve both pre-existing discontinuities and brittle fracturing of intact rock or failure through the rock mass. An example of a two-dimensional finite-/distinct element hybrid analysis is shown in Figure 25. This analysis enables modelling of the complete failure process from initiation through transport to deposition (Eberhardt, et al., 2002).

8 PRINCIPLES OF ROCK SLOPE DESIGN

8.1 STATIC DESIGN

The stability analysis methods applicable in most practical rock slope problems can be separated into the categories listed below. Categories 1 and 2 will be applicable for most common slope stability cases. Category 3 is expected to be required for high importance and complex slopes that require more rigorous analysis.

1. The first category includes the analysis of the kinematically possible structurally controlled failures. The initial selection of the slope angle from the kinematic analysis should reduce the number, size and probability of failure of structurally controlled failures, or eliminate them if possible. If elimination is not possible, the factor of safety of the remaining structurally controlled failures should be analysed using the LEMs discussed in Section 7.3.1. This type of analysis should be used for intermediate slopes (e.g. between benches) or slopes up to 10 or 20 m high in strong, jointed rock masses. The design of such slopes can sometimes be based upon analysis of structurally controlled failures only.

2. The second category includes non-structurally controlled failures, in which some or all the failure surfaces pass through a weak rock mass without discontinuities or one that has been weakened by the presence of randomly or chaotically oriented structural features. In this case the rock mass strength can be defined using the H-B criterion and the failure plane can be approximated by a circular failure surface. Noncircular surfaces can be used when predominant and persistent structural features influence the shape of the failure surface. These approaches should be used for the analysis of the overall stability of slopes higher than 20 m in heavily jointed moderately strong and weak rock masses. LEM using circular or noncircular surfaces can be used for the design of these slopes, as discussed in Section 7.3.2, ensuring that the key geological features are incorporated and modelled in the analysis and appropriate parameters are used for the rock mass and the structural features (see Section 4).

3. For slopes higher than 100 m, deformations and displacements of the rock mass and progressive failure phenomena may exist and these are better analysed by numerical methods of analysis. To analyse these high slopes and more complex phenomena the methods described in Section 7.4, sometimes in combination with LEM analyses, are more appropriate.

8.2 SEISMIC DESIGN

The pseudo-static method is used for incorporating seismic ground motions in LEM analyses. Seismic actions are modelled as a static force acting horizontally and vertically, in the case of circular or noncircular failures, or at a direction selected by the user (horizontal, vertical or parallel to the sliding plane) for structurally controlled failures. The seismic motions should be defined with the use of the current standard for seismic design incorporated in the New Zealand Building Code and other relevant current MBIE guidance or standard for earthquake geotechnical design. For more information on selection of seismic actions refer to Section 17.2.

The Newmark sliding block method can be used for rock slopes to calculate displacements during earthquake loading. While the Newmark method of analysis is idealised and the calculated displacements should be considered order-of-magnitude estimates of actual field behaviour, it is useful for design if there are guidelines on the relationship between slope stability and the calculated displacement. The process of estimating seismic displacements using the Newmark sliding block theory are presented in Section 17.6. Guidelines on interpreting calculated slope displacements for brittle materials are presented in Section 17.7 and Table 18. These guidelines should be used in the cases where project-specific guidelines have not been developed by the project owners.

For the co- and post-seismic behaviour of the slopes, consideration should be given to the amount of displacement that is acceptable before the residual strength of the sliding surface has been reached and brittle failure occurs. When displacement occurs, either along a single discontinuity failure plane or along a circular or complex noncircular surface, the asperities of the discontinuities are sheared off and the interlocking of rock mass blocks between the randomly oriented discontinuities is loosened. For example, for smooth or clay infilled discontinuity surfaces and poorly interlocked rock masses (GSI<30), a few mm or cm of displacement may be enough for the residual strength to be reached. The post-seismic behaviour of the slope should be also checked, using residual strength parameters for the sliding surface.

For very high or seismically critical slopes (Category 3 in Section 8.1), dynamic analyses using numerical methods are more appropriate to model seismic behaviour. This may provide insight into the behaviour of the rock (ductile or brittle) and what deformations need be allowed for in design.

8.3 TOPOGRAPHICAL AMPLIFICATION

Ground shaking can be significantly amplified by topographic features such as long ridges and cliff tops. For guidance on taking this topographical amplification into account, refer to Section 17.2.3 and Table 15.

8.4 APPLICABILITY AND LIMITATIONS OF ANALYSIS METHODS

Conventional LEM analyses are simplified and typically quick to use. In most cases they are appropriate for the level of geotechnical knowledge and investigation available and sufficient for the rock slope stability problems encountered in most projects.

Conventional LEM may in some cases oversimplify problems as they do not consider geometric complexities, non-linear behaviour of the rock mass, in situ stresses and the presence of several coupled processes (e.g. pore water pressures, progressive failure under seismic loading). For very high slopes and for slopes affecting seismically critical infrastructure, it is recommended that numerical modelling techniques are used, which can address these limitations.

Hoek (2023) provides guidance on recommended analysis methods and acceptability criteria for different problems in rock engineering, including slope stability, for the different modes of failure considered herein.

Recommended design approach methods for slopes along transportation corridors in New Zealand, depending on the importance level of the route and the scale and complexity of the slopes, are provided in Brabhaharan et al (2018).

9 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 & Sobhan (2016).

9.1 DEFINITIONS

Undrained conditions occur when load changes happen faster than water can flow into or out of the soil, meaning that the excess 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 water pressure.

$$\sigma' = \sigma - u \qquad \text{Equation 13}$$

Where:
σ' = effective stress
σ = total stress
u = pore water pressure

Where soils are fully saturated, it is the pore water pressure that is of interest. However, some soils like loess can be partially saturated with air pressure also acting on the soil particles. These soils have a modified version of the effective stress equation to account for the combination of both water and air pressures – this is discussed in Section 11.

Drained shear strength is the strength of a soil loaded to failure under drained conditions, in which any induced pore pressures drain away as rapidly as the load is applied. Load applied normal to the failure surface results in a change in effective stress, with no change in pore pressures.

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. Applied loading normal to the failure surface results in a change in pore pressure, but no change in effective stress.

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.

9.2 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 & Schuster, 1996).

It may be that some soil layers in a slope are drained, and others undrained for the same load condition, because of the difference in permeability of the layers. 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 & Schuster, 1996).

The analysis type (total versus effective stress) selection process is illustrated in Figure 26. A discussion of common loading conditions for analysis is included in Section 13.4.

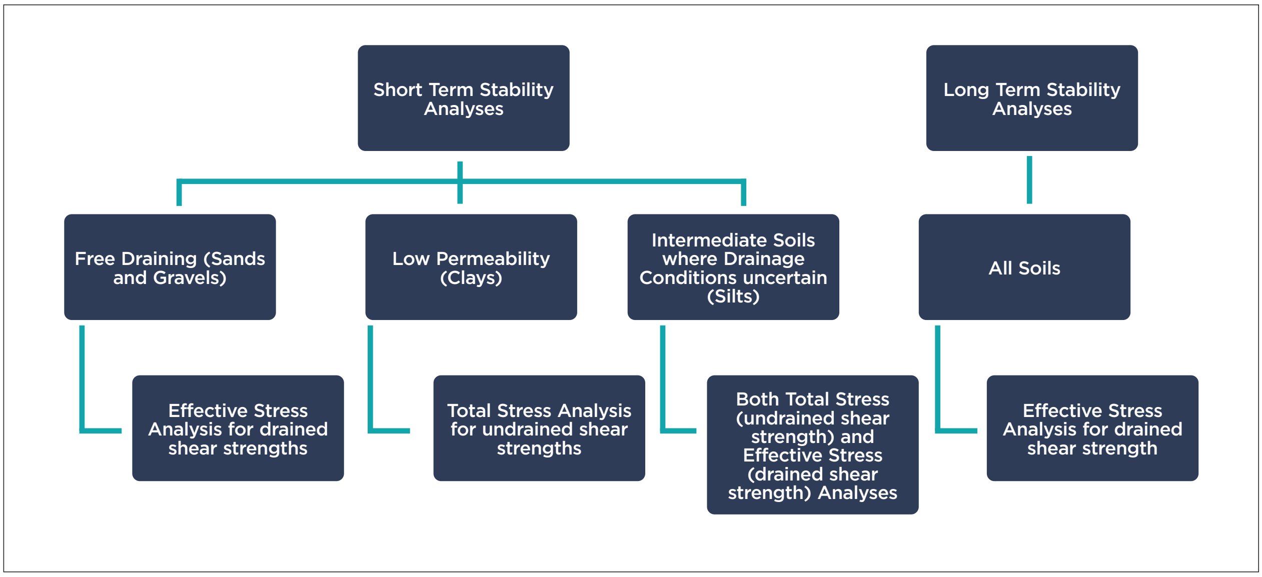


FIGURE 26: Analysis type is dependent on load duration and soil type

9.3 DETERMINING DRAINAGE CONDITIONS

Determining the drainage conditions of soils in response to rapid loading or unloading (such as an earthquake or a new cut) 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^{-9}m/s will typically be undrained. 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 & Schuster, 1996).

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.

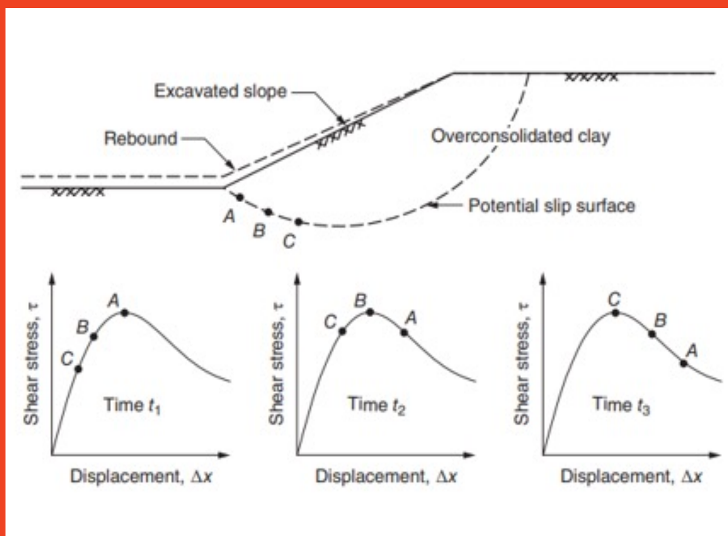


FIGURE 27: Mechanisms of progressive failure on an excavated slope in overconsolidated clay. From Duncan et al (2014), Figure 3.9.

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.

10 SHEAR STRENGTH OF SOILS

10.1 GENERAL PRINCIPLES

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 effective stress failure envelope for a saturated soil (shown in Figure 28) is the relationship between the soil's shear strength and effective normal stress and can be expressed by the following equation:

$$\tau = c' + (\sigma - u)\tan\varphi'$$
$$= c' + (\sigma')\tan\varphi' \qquad \text{Equation 14}$$

Where:

τ = soil shear strength – the effective shear stress on the shearing surface at failure
c' = cohesion intercept in terms of effective stress
φ' = internal friction angle in terms of effective stress
σ' = effective normal stress on the failure plane at failure
σ = total normal stress on the failure plane at failure
u = pore water pressure on the failure plane at failure

If the shear stress on any plane within the soil exceeds the value given by the above equation, failure will occur on that plane.

The effective cohesion (c') results from bonding between soil particles and is independent of the effective normal stress. The effective friction angle (φ') is primarily due to friction between the soil particles and frictional shear strength is dependent on the effective normal stress acting on the failure plane. If the soil is not saturated, then there may be an apparent cohesion due to suction. This is discussed in detail in Section 11.

Undrained loading of saturated soils, where no change in water content occurs, results in a horizontal, total stress failure 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. The total stress (undrained strength) failure envelope (shown in Figure 28) for a saturated soil is defined by:

$$\varphi = 0°, \text{ therefore } \tau = c = S_u \qquad \text{Equation 15}$$

Where:

c = cohesion intercept in terms of total stress
S_u = undrained shear strength

10.2 USE OF MOHR'S CIRCLES

To establish the Mohr-Coulomb failure envelopes shown in Figure 28, and hence the shear strength parameters, a graphical construction known as a Mohr's circle is normally used to illustrate the stress states at failure from laboratory shear strength tests, such as triaxial compression tests.

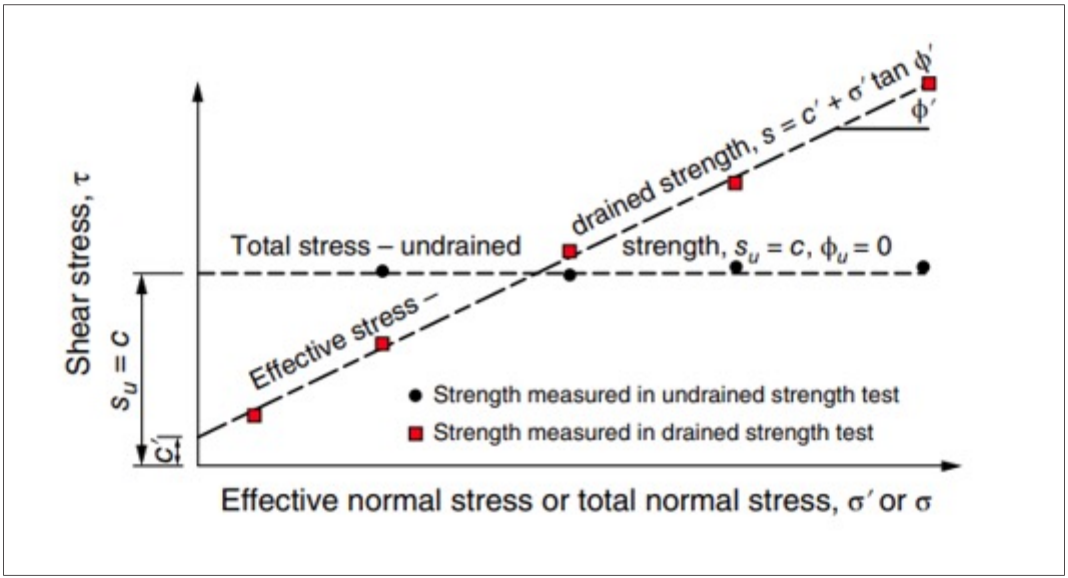


FIGURE 28: Drained and Undrained Strength Envelopes for Saturated Clay (Duncan et al., 2014).

In triaxial compression tests, the major principal stress (σ_1) and the minor principal stress (σ_3) on the failure plane act in the vertical and horizontal directions, respectively. The cell pressure applied to the cylindrical soil sample in the first stage of the triaxial test results in an equal all-round stress, which corresponds to σ_3. In the second stage of the triaxial test, the vertical stress applied to the sample is increased from σ_3 to σ_1 by a ram to the top of the sample. The difference ($\sigma_1 - \sigma_3$) is known as the deviator stress.

The values of σ_3 and the deviator stress in the triaxial test, at failure, are used to construct a Mohr's circle (actually a semi-circle) on a graph, with the x-axis representing normal stress (σ' or σ) and the y-axis the shear stress (τ), as shown on Figure 29, i.e. the same stress axes as Figure 28. As shown on Figure 29, the diameter of the Mohr's circle is equal to ($\sigma_1 - \sigma_3$) and the left intersection of the Mohr's circle with the x-axis corresponds to σ_3.

A series of Mohr's circles are then constructed for several triaxial tests undertaken at different cell pressures and the tangent line to those circles then defines the Mohr-Coulomb failure envelope, as shown on Figure 29.

The Mohr's stress circles for a CU triaxial test and a UU triaxial test are shown in Figure 36 and Figure 37 respectively.

10.3 SHEAR STRENGTH PROPERTIES

This Unit provides a brief overview of soil shear strength and its estimation, but the topic is covered extensively in other texts. A selection of those texts is listed below, and we encourage the reader to review these texts for more detailed discussion.

- Blake et al (2002). Recommended Procedures for implementation of DMG SP 117.
- Duncan et al (2014). Soil Strength and Slope Stability.
- Turner and Schuster (1996). Landslides: Investigation and Mitigation.
- Bowles (1996). Foundation Analysis and Design.
- Holtz et al (2011). An Introduction to Geotechnical Engineering.
- Look (2017). Handbook of Geotechnical Investigation and Design Tables 2nd edition.

Of relevance to determining the shear strength of residual soils are the following two texts, which include examples of soils in New Zealand:

- Wesley (2010a). Fundamentals of Soil Mechanics for Sedimentary and Residual Soils.
- Wesley (2010b). Geotechnical Engineering in Residual Soils.

A general overview of shear strength properties by soil type is presented below. This section predominantly relates to the static strength of soils; dynamic soil strengths are discussed in Section 17.3.

10.3.1 Clay

The presence of clay and the complex interaction of clay and water contribute to many slope instabilities. Consequently, understanding the shear strength of clay is often critical to the slope stability assessment. Because of the low permeability of clays both undrained and drained conditions can occur.

Sedimentary Clay: The strength characteristics of sedimentary clay depend on stress history, i.e. if the clay has not been subject to higher pressures in the past compared to its present in situ state (normally consolidated, NC), or if it has been subject to higher pressures (overconsolidated, OC). Fully saturated strength characteristics of clays can be broadly grouped into (1) normally and lightly consolidated clays, (2) heavily overconsolidated clays, and (3) clays at their residual strength.

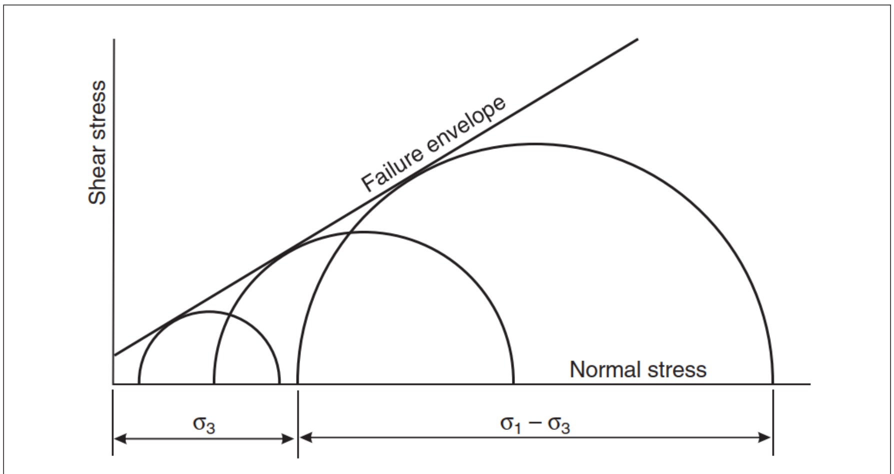


FIGURE 29: Mohr's circles (Wesley, 2010a)

Normally Consolidated and Lightly Overconsolidated Sedimentary Clays
- Normally consolidated saturated clays have an effective cohesion of zero (c'=0).
- Stability under loading will generally be most critical under short-term loading conditions (S_u) due to excess pore pressures. The stability of loaded slopes can increase over time as the excess pore pressure dissipates and strength increases. This strength gain from consolidation allows for staged construction of embankments over soft soils.
- Stability under unloading (i.e. cuts and excavations) may be critical under short term undrained or long term drained conditions (Blake et al., 2002). Both should be checked.
- Peak strengths can typically be adopted for normally consolidated low plasticity cohesive soils provided they have not been subject to significant previous shear deformations (Blake et al., 2002).

- Heavily Overconsolidated Sedimentary Clays – also termed stiff-fissured clays (also applies to clayey bedrock)
  - When saturated and loaded in drained conditions, these clays absorb water, leading to softening and a reduction in strength to a fully softened state. This fully softened strength is similar to the strength of normally consolidated clays (Figure 30). Consequently, stability of these soils in the long-term drained condition is usually the most critical but both short-term undrained and long-term drained conditions should be checked (Blake et al., 2002; Turner & Schuster, 1996).
  - These clays exhibit lower strengths in the field compared to laboratory measurements due to this softening which doesn't occur during short-term lab tests. Additionally, fissures, common in heavily overconsolidated clays, significantly affect field strength but are not well-represented in lab samples unless the specimens are large enough (Blake et al., 2002; Turner & Schuster, 1996).
  - To account for this tendency of laboratory tests to overestimate field effective strength of OC clays, fully softened strengths should be measured and used in analysis, where laboratory testing is used to derive effective stress shear strength parameters. Deriving the fully softened shear strength from laboratory testing involves remoulding the sample, consolidating to the desired overburden pressure and testing the sample. Direct shear tests and ring shear tests can be used to measure fully softened strengths.
  - Stability analysis of slopes in these clays should consider progressive failure, with the design shear strength representing the average shear strength along the rupture surface.
  - While most of the research on the shear strength of these materials focusses on effective stress shear strength as long-term loading tends to be the most critical case, the undrained shear strength of fissured clays is also impacted by fissures. Wright & Duncan (1972) showed that measured undrained shear strength decreases with increasing tested specimen size. Small specimens are likely to be intact with few fissures and therefore stronger than a larger representative mass of the material.
- Residual Shear Strength of clays
  - When large shear displacements occur within a narrow zone in clay, the clay particles become aligned along the direction of shear and a polished surface or slickenside forms. In natural slopes slickensides form along the failure surface of old landslides, bedding planes or zones of deformation (Turner & Schuster, 1996).
  - Shear strength along these surfaces is not dependent on stress history (i.e. NC or OC) and is described by the effective residual friction angle, φ_r'.
  - φ_r' depends on soil minerology which makes it possible to correlate φ_r' with index properties such as PI, LL, and clay content (Duncan et al., 2014; Turner & Schuster, 1996; Stark & Hussain, 2013).
  - In stability analyses, residual strength should be applied to slopes or zones within slopes that have failed or undergone large displacements.

Residual Soils (Clays and Silts): Residual soil slopes cover large portions of developed areas of New Zealand, such as around Auckland. The weathering of the interbedded sandstone and mudstone parent rock of the East Coast Bays Formation in the Auckland area results in the formation of residual soil comprising interbedded layers of sandy silt and clay. Residual soils are generally produced by the physical and chemical weathering of the underlying parent rock, but in some cases, may be derived by weathering of fresh volcanic ash, resulting in clay minerals such as allophane, imogolite and halloysite, which have unusual properties and are only found in residual soils (Wesley, 2010b).

- As residual soils have not been deposited by a sedimentation process, stress history is not relevant to them, i.e. the NC and OC classifications, that are applicable to sedimentary clays and silts, are not applicable to residual soils (Wesley, 2010b). However, the principle of effective stress and the Mohr-Coulomb failure criteria are applicable to both residual and sedimentary soils.
- Properties of residual soils vary widely, depending on the parent material and weathering degree.
- Residual soils are often partially saturated, and moisture content changes significantly with seasons, affecting shear strength. Residual soil slopes often fail during periods of heavy rainfall because of increased moisture contents which reduce soil suction and shear strength.
- If the soil is likely to become saturated, effective stress analysis for the saturated condition provides a conservative estimate of stability. Alternatively, total stress analysis can be carried out using undrained shear strengths, but strengths used must correlate to the in situ moisture content for the scenario or condition that the geoprofessional is modelling (Turner & Schuster, 1996). The shear strength of partially saturated soils is discussed in detail in Section 12.

10.3.2 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 low liquid limit (non-plastic) silts, which exhibit behaviour similar to fine sands, and high liquid limit (plastic) silts, which behave more like clays. 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.

10.3.3 Granular Soils – Sand and Gravel

Because of their high permeability, granular soils are usually fully drained and cohesionless (except during liquefaction). Consequently, the shear strength of granular soils is defined in terms of the effective friction angle.

- 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.
- Granular soil shear 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 stresses can be unconservative (Figure 31).
- The undrained behaviour of loose granular soils is important when considering slope stability in earthquakes. Large excess pore pressures can develop reducing shear strength to well below its peak, i.e. liquefaction. Liquefaction is covered extensively in Module 3. Consideration of liquefaction in slope stability is discussed in Section 17.3.1 of this Unit.

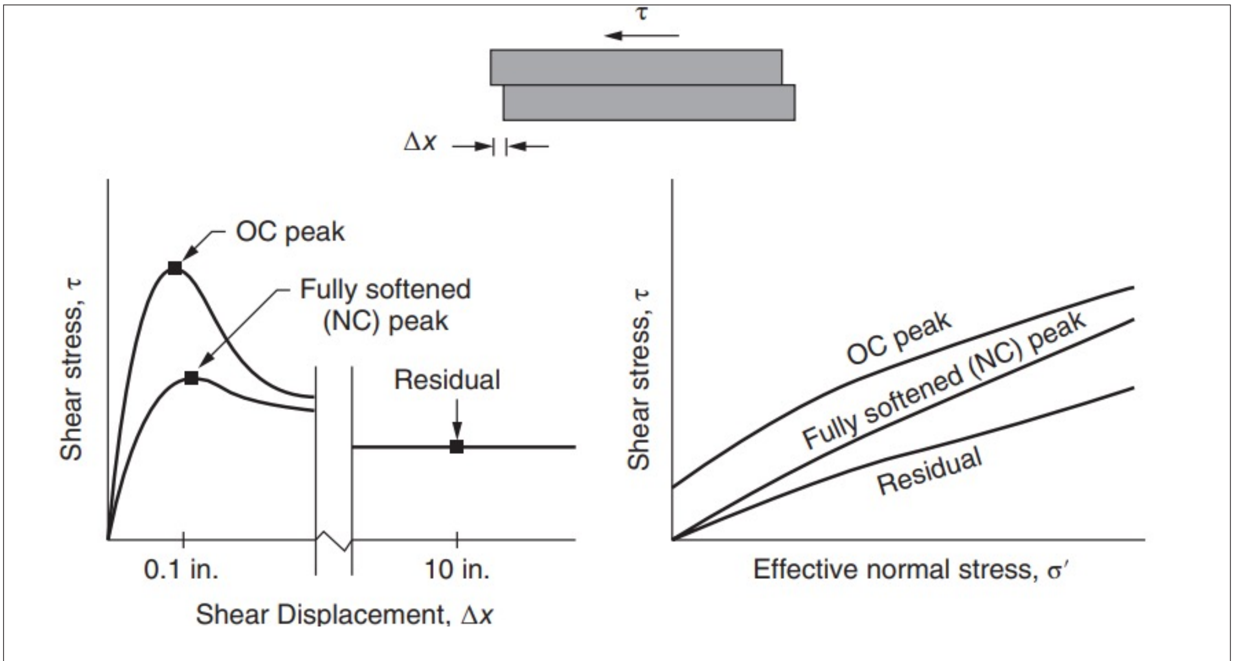


FIGURE 30: Drained shear strength of heavily overconsolidated clays. (Duncan et al, 2014).

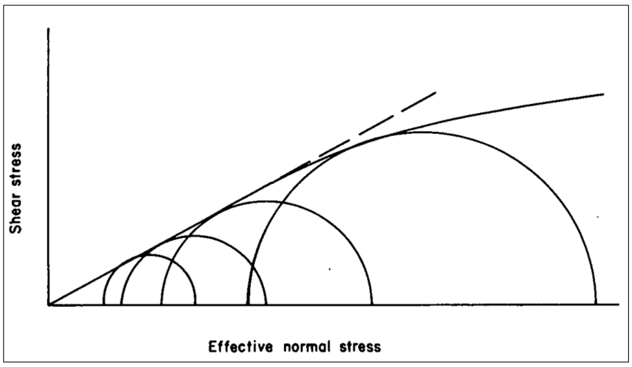


FIGURE 31: Curved effective stress shear strength envelope (Turner and Schuster, 1996)

10.4 METHODS FOR EVALUATING SOIL SHEAR STRENGTH

In broad terms, the methods for evaluating the soil shear strength include the following:

- Back-analysis of failed and/or intact slopes in the vicinity
- Reference to shear strength parameters for the same soil type from other reliable geotechnical investigation reports (GIR) or the literature (e.g. Pender, 1980 for weathered Wellington Greywacke)
- Laboratory shear strength tests, such as triaxial (CU, CD and UU), direct shear (DS) and unconfined compression (UCS) tests. Laboratory shear strength tests are discussed in more detail in Section 10.5.
- In situ tests, such as CPTs, DMTs, SPTs and shear vane tests.
- Tactile assessment of in situ soil
- Correlations with classification test values, such as Atterberg limits and particle size gradings.

The selection of the evaluation method for soil shear strength on any project will depend on several factors such as:

- The nature of the soils being assessed (e.g. granular vs cohesive)
- The consequences of slope failure
- The availability of existing failed (i.e. landslides) or intact slopes in the vicinity for back-analysis
- The availability of reliable shear strength parameters for the specific soil under consideration from other GIRs or the literature
- Accessibility for in situ test equipment
- The geotechnical investigation budget.

Judgement will need to be exercised by the practitioner, taking account of these factors, when selecting the optimum evaluation methods for any particular soil. Where possible, multiple methods of measuring and estimating shear strength parameters and multiple correlations should be used to capture the variability in the test and correlation methods.

Wesley (2010b) states:
"*In evaluating the properties of residual soils it is very important to first observe carefully their behaviour in the field, before looking at the results of laboratory tests*" and this applies not just to residual soils but to all soils and rocks as well.

Notwithstanding the above factors, a broad hierarchy of evaluation methods for determining the shear strength of the three main soil types, mostly in descending order of preference, is provided in Table 5. Comments and considerations relating to the various evaluation methods are also shown in the commentary column of the table.

"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 in situ testing. This approach is generally not adequate for slope stability analysis, except for low-consequence-of-failure 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?**
- **What are the uncertainties associated with these parameters? Are these considered 'best estimate' values or conservative estimates?**
- **How sensitive is stability to reasonable variance in the selected parameters?**

10.5 LABORATORY SHEAR STRENGTH TESTING

In New Zealand, it's relatively common for soil strength assessments used in analysing slope stability to be conducted without laboratory strength testing. Despite the additional cost and limited availability of soil laboratories, especially outside major cities, there is substantial benefit to laboratory shear strength tests, particularly for clayey soils. Laboratory tests can capture the range of conditions that the soil may be subject to over the design life. These conditions aren't always captured during in situ testing as in situ testing only reflects the specific conditions (saturation levels, stress conditions, levels of deformation etc.) at the time and location of testing. Including laboratory shear strength testing in the geotechnical evaluation can reduce uncertainty in the ground model, lessen the chance of unexpected slope issues, and possibly lead to less conservative analyses, resulting in more cost-effective designs and savings on the overall project. Laboratory shear strength testing, particularly in clayey soils, is encouraged as good practice for slope stability studies in New Zealand.

Some considerations in selecting and interpreting test results from laboratory strength tests are outlined below:

- **Sample Disturbance** - Sample disturbance affects the laboratory measured undrained shear strength of clays and silts, reducing the measured shear strength. The goal should be to collect samples as undisturbed as possible for laboratory testing by (1) Using thin-walled tube piston samplers, (2) Sealing tubes upon retrieval to prevent water loss, and (3) storing and transporting carefully. The SHANSEP procedure (Ladd & Foott, 1974) can be used to address sample disturbance – this procedure involves consolidating samples beyond in situ stresses to reduce disturbance effects (Duncan et al., 2014). UU test and UCT tests are significantly influenced by sample disturbance.
- **Drainage Condition** – The selection of appropriate laboratory testing depends on the likely drainage conditions the soil will be subject to. Commentary on drainage conditions for typical soils is provided in Section 11.
- **Rate of Loading** – The undrained shear strength of soils generally increases with the rate of loading. Most laboratory tests bring soil to failure within hours or days, while field loading can take weeks or months (Turner & Schuster, 1996). Higher loading rates in laboratory tests can lead to higher measured shear strengths. Laboratory tests are not typically corrected for rate effects. Where UU tests have been used, the increase in measured shear strength due to high load rate is partially offset by the decrease in measured shear strength due to sample disturbance (Duncan et al., 2014). Blake et al (2002) recommends a 30% decrease in the measured undrained shear strength for static slope stability of fine-grained alluvium under new loading to account for rate effects. We recommend that a 30% decrease be applied to the undrained shear strength measured in consolidated undrained (CU) triaxial testing (where disturbance effects are less than rate effects) to account for rate effects.
- **Soil Anisotropy** – The undrained shear strength of clays is anisotropic; that is, it varies with the orientation of the failure plane (Figure 32). This is due to both the fabric of the soil and the anisotropy of stresses the soil is subject to. Different laboratory tests measure shear strength at different orientations of stresses. For example, if the soil deposit has horizontal layering, then lower shear strength may be measured using direct shear tests (where the failure plane is horizontal) than using triaxial tests (where the shear plane is inclined). Where the soil is horizontally bedded, contains shear zones, or is otherwise highly anisotropic, the geoprofessional should consider that anisotropy and orientation of stresses in the slope to select the laboratory tests that best reflect the stress orientation the soil is likely to be subject to. Further discussion and recommendations for addressing anisotropy is provided in Duncan et al (2014) and Blake et al (2002).
- **Effect of Confining Stress on Soil Failure Envelope** - For most soils, the Mohr failure envelope is curved (Figure 31). It is therefore important to specify the correct confining stress range for the triaxial tests to match those encountered in the field (Fell & Jeffery, 1987). Modern slope stability software packages allow representation of curved failure envelopes.

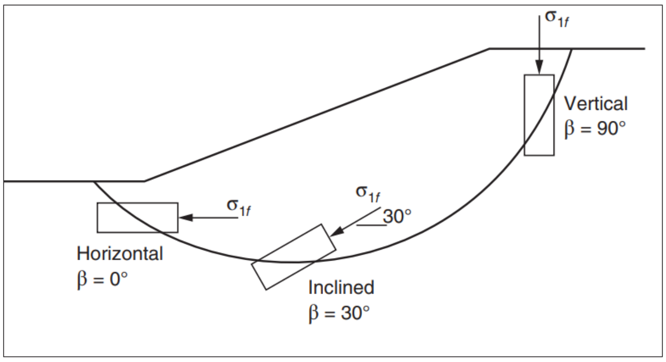


FIGURE 32: Stress orientations at failure (Duncan et al., 2014).

A brief description of common laboratory soil shear strength tests is provided in Table 4. Considerations for their use to estimate shear strength for particular soils is included in Table 5, and some additional comments on laboratory testing and use are presented below:

- Laboratory derived shear strengths should be checked against published correlations and if significant deviation is found, some justification should be provided or the laboratory shear strength value revised (Blake et al., 2002).
- In situations where the size of the project and investigation budget does not allow for laboratory strength testing, there is value in index testing for use in correlations. These index tests typically comprise Atterberg Limits, clay content, and moisture content tests for fine grained soils, and particle size distribution tests for coarse grained soils.
- Table 4 describes laboratory shear strength tests common in New Zealand. Other shear strength tests such as laboratory vane shear testing, direct simple shear tests, cyclic direct simple shear, and cyclic triaxial tests are available but are not as commonly employed.

Table 4: Common Soil Laboratory Shear Strength Tests

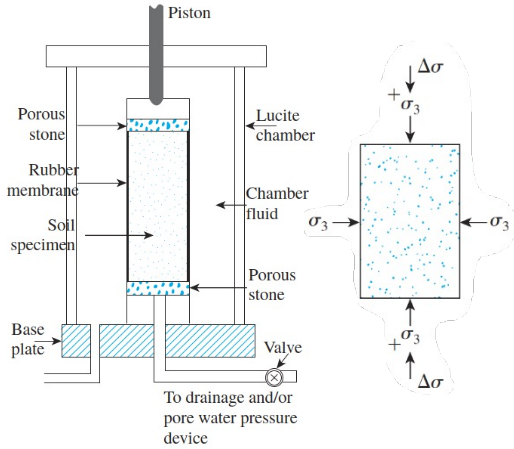
<table>
<tr><th colspan="2">Test and Parameters Measured</th><th colspan="2">Test Description</th></tr>
<tr><td rowspan="3">Triaxial Compression Tests</td><td>Consolidated Undrained (CU) Test
ϕ', c', S_u</td><td rowspan="3">A cylindrical soil specimen is enclosed in a thin rubber membrane and placed on a porous disc mounted on the base pedestal inside a triaxial cell, which is filled with fluid. Pressure is applied to the fluid, subjecting the specimen to a cell (confining) pressure (σ_3). A back-pressure (p) can be applied via the drainage line and porous disc, to dissolve any air and ensure the specimen is saturated and to prevent cavitation. The consolidation pressure is the difference between the cell and back-pressures (σ_3 -p). An axial stress ($\Delta\sigma$) is applied to the specimen at a constant strain rate via a loading ram, and drainage can be controlled and pore pressures measured.
• CU test[1] – The sample is allowed to consolidate (i.e. allowed to drain) under the selected confining pressure. Then the axial load is applied with the drainage lines closed leading to the development of excess pore pressure during shearing.
• UU (or Q) test[2] - A confining pressure and axial load are applied without allowing drainage or consolidation to simulate rapid loading conditions. Testing should be carried out on three samples at the desired depth.
• CD test – The sample is allowed to consolidate under the selected consolidation pressure then sheared slowly enough to allow drainage and prevent buildup of excess pore pressures.</td><td rowspan="3">

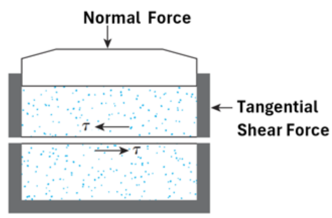
FIGURE 33: Triaxial test equipment and stress application (Das, 2009).</td></tr>
<tr><td>Unconsolidated Undrained (UU) (or Q) Test
S_u</td></tr>
<tr><td>Consolidated Drained (CD) Test
ϕ', c'</td></tr>
<tr><td colspan="2">Consolidated Drained Direct Shear Test (DDS)
ϕ', c'
ϕ_r', c_r'</td><td>The soil specimen is enclosed in a split box. A normal force is applied vertically, and the soil is allowed to consolidate, then sheared at a constant strain rate. The shear force and displacement are recorded. Drainage is not controlled, and pore pressures are not measured so the test should be carried out sufficiently slowly to ensure no significant pore pressures develop. The test can be used to subject the sample to multiple cycles of shearing to allow estimation of residual shear strength, but results may be unconservative (Watry & Lade, 2000) and therefore ring shear tests are preferred for measuring residual shear strength.</td><td>

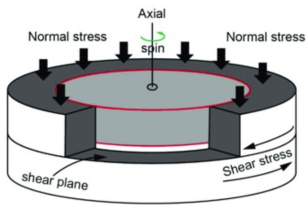
FIGURE 34: Direct shear test diagram (adapted from Das, 2009)</td></tr>
<tr><td colspan="2">Ring Shear Test
ϕ_r', c_r'</td><td>Used to measure the residual shear strength of soils, the test involves placing a remoulded, annular-shaped soil sample in a ring-shaped apparatus and applying continuous rotational shear to the sample, allowing for unlimited shear displacement. This method is preferred for measuring residual shear strengths because it can simulate the conditions of large-scale soil movements.</td><td>

FIGURE 35: Ring shear diagram (Wang et al., 2022).</td></tr>
<tr><td colspan="2">Unconfined Compression Test (UCT), S_u</td><td colspan="2">A cylindrical soil specimen is loaded axially without any lateral confinement until failure occurs. The undrained shear strength (S_u) is taken as one-half of the compressive strength.</td></tr>
</table>

[1] Results of a CU triaxial test, including the Mohr's stress circles, are shown in Figure 36.

[2] Results of a UU triaxial test, including the Mohr's stress circles, are shown in Figure 37.

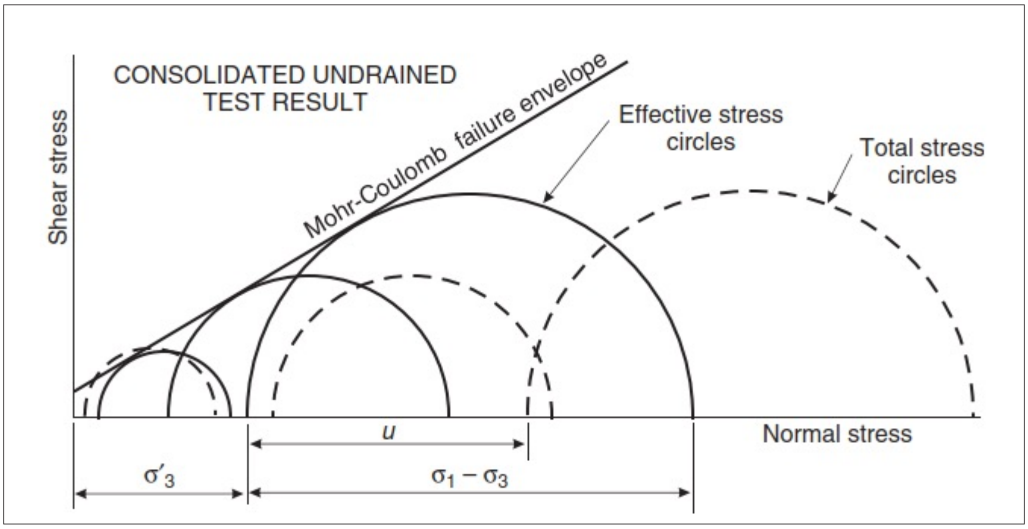


FIGURE 36: Results of a CU triaxial test on saturated cohesive soil (Wesley, 2010a, Figure 9.9)

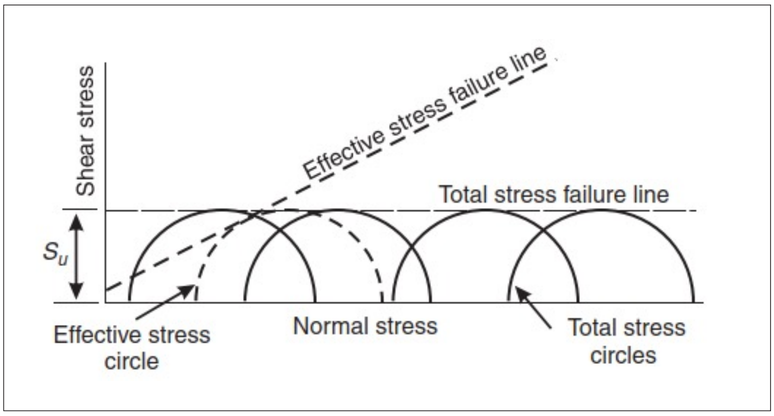


FIGURE 37: Results of a UU triaxial test on saturated cohesive soil (Wesley, 2010a, Figure 9.8)

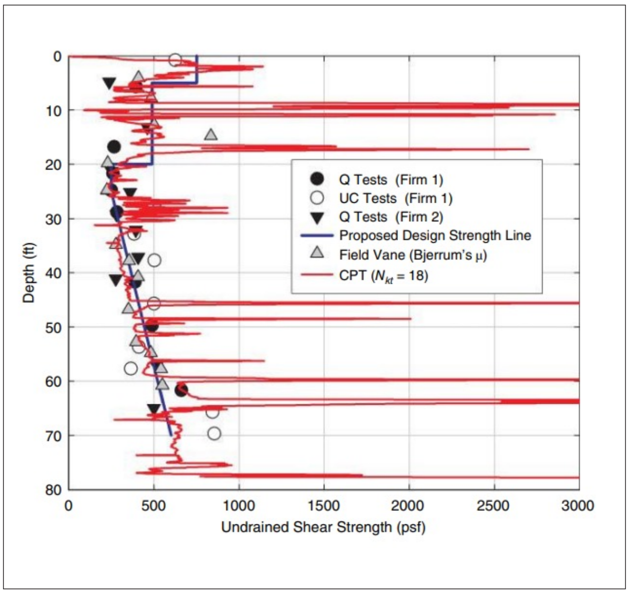


FIGURE 38: Undrained shear strength determined by multiple tests (Duncan et al., 2014)

10.6 SELECTING DESIGN SHEAR STRENGTH PARAMETERS

The appropriate number of investigations and tests to develop design strength parameters depends on several factors including local experience with the ground conditions, the complexity of the ground conditions and the consequence of failure. Levels of investigation are discussed in Part 5, Section 4 of Unit 1, and Section 13.6 of this document discusses levels of investigation in relation to target FoS.

The number of tests should be adequate to represent the variations expected from natural processes or the construction methods that created the deposit. If several tests are carried out, it is good practice to check the calculated FoS for sensitivity against a range of shear strengths e.g. average, lower quartile and lower bound (see Section 19 for discussion of sensitivity analyses).

A summary of test shear strength data for each unit assessed in the stability analysis should be provided in the analysis documentation, along with justification for the selected design shear strength value for the unit. An example of a comparison of estimated strength for a soil unit using multiple types of tests is shown in Figure 38.

Table 5: HIERARCHY OF METHODS FOR EVALUATION OF SOIL SHEAR STRENGTH

Explanation of the hierarchy: Back-analysis, where possible, is commonly the most preferred method for all soils, hence it is listed first. A typical values approach is usually the least preferred, hence is listed last.
Within in each soil-type, the methods are listed broadly hierarchically, with the most preferred method first.

	Soil Parameter	Method		Comments and Considerations
ALL SOILS	ϕ', c' ϕ_r', c_r', S_u	Back-analysis of failed and intact slopes		• The back-analysis method for determining shear strength is often a better method (when applicable) than laboratory testing because it eliminates problems associated with sample size and quantity and inherent problems with different shear test apparatus (Blake et al., 2002). • Only applicable if suitable slopes are available in the vicinity. • Shear strengths obtained from back-analysis should be compared against and used in combination with other available strength data (in situ testing, correlations, laboratory tests). • Where there is an existing failure plane, the field residual shear strength should be used for the failure plane, regardless of how long ago the failure occurred. • For stabilisation of landslides, the shear strength can be determined by back-analysis and stabilisation works designed on the basis of increasing the FoS. Considerations for target FoS for landslide remediation is briefly discussed in Section 13. • In most cases, one shear strength parameter (say ϕ') should be determined using other information and the other parameter (c') can then be obtained by back-analysis. • Back-analysis is discussed in more detail in Section 18.
CLAYS	**Laboratory Testing**			
CLAYS	ϕ', c', S_u	Triaxial Tests	Consolidated -Undrained Triaxial test with pore pressure measurement (CU test)	• To avoid problems with partial saturation, the test samples should be saturated by percolation followed by back pressure saturation. The strain rate must be sufficiently low so that pore pressure is equalised throughout the sample. If the strain rate is too fast, the pore pressure change measured at the ends of the sample is less than the actual change at the centre where shearing is occurring. • The triaxial test cannot be used to obtain the softened or particularly the residual shear strength as the shear displacement is insufficient to reach these values. • Conventional CU tests usually involve testing three separate samples of the selected soil. Staged tests, where a single sample is saturated and then consolidated and sheared at the lowest confining (cell) pressures and again at the second and third confining pressures, allow the same fabric features to be tested at a lower cost (due to less sample preparation time than for 3 non-staged samples). The staged procedure generally gives acceptable results but tends to give lower shear strengths for the second and particularly the third stage due to sample deformation and displacement on the shear plane. Staged testing should not be used for sensitive or cemented soils (Fell & Jeffery, 1987). • The test measures undrained shear strength, but where pore pressures are measured, the peak effective stress shear strength parameters (ϕ', c') can be obtained.
CLAYS	S_u	Triaxial Tests	Unconsolidated -Undrained Triaxial Test (UU or Q test)	• Quicker and more reliable than an unconfined compression test (UCT), a 3-point test should be carried out with three separate samples from the same depth. • Historically, it has been the most popular triaxial test and has been the main laboratory method of determining undrained shear strength in geotechnical engineering practice. • Some researchers have cautioned against the use of UU tests as accurate results rely on the incidental cancellation 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 and consolidation tests.
CLAYS	ϕ', c'	Triaxial Tests	Consolidated Drained Triaxial test (CD test).	• CD tests are uncommon because they take a long time and are not recommended for clays or silts – CU tests are preferred.

CLAYS	ϕ'_r, c'_r ϕ'_{fs}	Ring shear test	• Ring shear tests are the preferred test for residual shear strengths due to the large deformations achievable. • Correlate well with direct shear tests on the slide plane (Fell & Jeffery, 1987). • Ring shear tests can also be used to derive fully softened shear strength (ϕ'_{fs}). • Ring shear tests either under-estimate by 1° to 2° or approximate the field residual shear strength obtained by back-analysis of landslides (Fell & Jeffery, 1987). • Only remoulded samples can be tested.
	ϕ', c' ϕ'_{fs} ϕ'_r, c'_r	Consolidated-Drained Direct Shear test (CDD test)	• If the soil deposit has horizontal layering, then lower friction angles may be measured using direct shear tests where the failure plane is horizontal, than in triaxial tests, where the shear plane is inclined (Duncan et al., 2014). • Drainage occurs more rapidly in a CDD test than a CD test so is often more practical than a CD test for effective shear strength parameters (Duncan et al., 2014). • Direct shear tests are used to measure fully softened shear strength (ϕ'_{fs}). • CDD tests can be used to estimate residual shear strength (ϕ'_r). However, multiple reversals are required to accumulate sufficient displacement and CDD tests will probably over-estimate the field residual strength by 1° or 2° (Fell & Jeffery, 1987) and are not recommended. Ring shear tests are preferred for residual shear strengths.
	S_u	Laboratory Miniature Vane Shear Test	• Good for soft saturated clays, simple to perform.
	S_u	Unconfined Compression Test (UCT)	• Often the cheapest, least reliable, and gives the lowest undrained shear strengths as most sensitive to sample disturbance. Other tests/methods are preferred.
	In situ Testing		
	S_u	Shear vane tests	• NZGS (2005) states that: 'Undrained shear strength can be determined using either field or laboratory tests. The most common field test in NZ is the hand held shear vane'. • The shear vane directly measures shear strength of the soil. The other in situ measures outlined here are correlations with in situ testing. • The handheld Field Shear Vane is a good and cheap way of estimating S_u in saturated soils. The S_u measured with a handheld shear vane should be calibrated against the value from a UU test. NZGS (2001) recommends that the vane readings are adjusted according to BS 1377:1990. • Geonor Field Vane is a push-in type vane operated from a drill rig for measuring soil shear strengths at depth, typically in very soft to firm soils. • Bjerrum (1972) provides vane shear strength correction factors for soft clays, depending on the PI value, to determine the field undrained shear strength. It is, however, recommended that the vane shear strength be calibrated against UU triaxial test values if reliable S_u values are required, rather than relying on the Bjerrum correction factors.
	S_u ϕ', c'	Cone Penetration Test (CPT)	• S_u is related to the CPT cone resistance and overburden using a N_{kt} factor. N_{kt} typically varies between 10 to 18 (Robertson & Cabal, 2015). Holtrigter et al (2017) provides typical values of N_{kt} in Auckland clays. Wesley (2010b) Figures 6.8 and 6.9 (after Pender) provides an N_{kt} value of 12 for volcanic 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 (S_{ur}) can be estimated to be equal to the CPT sleeve friction, f_s, but due to inherent difficulties in accuracy the estimate should be viewed as a guide only (Robertson & 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) • While results from CPT testing have traditionally been used to assess undrained shear strength using total stress parameters (S_u), correlations have been developed to estimate effective stress shear strength parameters of clays using the Norwegian Institute of Technology (NTH) solution. These relationships are detailed by Mayne (2016).
	S_u ϕ', c'	Dilatometer (DMT)	• S_u can be estimated based on relationships with K_d. The relationship can be improved with calibration based on field shear vane tests. • The NTH solution developed for the CPT has been extended to the DMT to provide correlation between DMT results and the effective stress shear strength parameters. See Mayne (2016) for details.

CLAYS	S_u ϕ', c'	Standard Penetration Test (SPT) "N" values	• Relationships of undrained shear strength (S_u) or effective stress shear strength (ϕ', c') with SPT blow count provides only a crude estimation of shear strength and are not recommended, especially for SPT 'N' values below 10. Other in situ methods are preferred.
	S_u	Tactile Assessment	• Table 2.9 of NZGS (2005) provides a basis for tactile assessment of in situ (undisturbed) fine (cohesive) soil. The table sets out six descriptive consistency (or stiffness) terms, ranging from very soft to hard, and corresponding ranges of undrained shear strength and diagnostic tactile features. The diagnostic feature for each consistency band relates to a wide range of shear strength; for example, the diagnostic feature for Firm consistency – 'Indented by strong finger pressure and can be indented by thumb pressure' relates to a shear strength range from 25 – 50 kPa and that for Stiff consistency – 'Cannot be indented by thumb pressure' to a shear strength range from 50 – 100 kPa. • The tactile assessment involves uncalibrated finger/thumb penetration tests (size and shape of finger or thumb and pressure applied by the person doing the test not calibrated/defined). Also, the term "strong finger pressure" and "thumb pressure" depend on the subjective judgement of the person conducting the 'tactile' test. The tactile assessment method is consequently imprecise and uncalibrated and cannot therefore be relied upon to provide an accurate measure of undrained shear strength. • While a tactile assessment is a handy, quick, field test, it should not be definitive and other methods should also be used.
	Correlations		
	Many useful correlations between clay shear strength parameters and other index properties have been developed. These correlations are useful to check the general validity of laboratory or insitu test results, or to develop preliminary shear strength estimates. However, we emphasise that reliance on these correlations should be avoided for all but preliminary estimates and that any assumptions regarding shear strength parameters should always be verified through testing.		
	ϕ', ϕ'_{fs}, ϕ'_r	• Wesley (2010a), Figures 9.36 and 9.37 provides correlations between ΔPI see footnote 2 and ϕ' and ϕ'_r respectively – applicable to both sedimentary and residual soils. • Duncan et al (2014) presents correlations developed by Stark & Hussain (2013) of the fully softened (ϕ'_{fs}) and residual (ϕ'_r) friction angles based on Liquid Limit, clay fraction, and effective normal stress.	
	S_u	• A number of empirical correlations exist relating the undrained shear strength to plasticity index (PI) and overconsolidation ratio (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, 1977). • Atterberg limit and particle size tests relate to the disturbed/remoulded soil and do not provide data on the peak shear strength of the soil in its undisturbed state. • A few commonly referenced correlations are shown below but reliance on these correlations in isolation should be avoided for all but preliminary estimates of shear strength. These correlations are only relevant to sedimentary soils and have no relevance to residual soils in their undisturbed state (refer Wesley (2010b)). - Skempton (1957) - S_u = (0.11+0.0037 PI) σ'_v. This correlation relates to normally consolidated soils. - Mesri (1989) - S_u / σ'_v = 0.22(OCR). - A relationship between (S_u / σ'_v) and PI for marine clays is given in Simons & Menzies (1977), after Bjerrum & Simons (1960), which shows (S_u / σ'_v) ranges from 0.10 to 0.35 for PI values ranging from 5 to 70. - Jamiolkowski et al (1985) – known as the SHANSHEP equation: $S_u / \sigma'_v = 0.23\ (OCR)^{0.8}$. The general form of this equation was introduced by Ladd and Foott (1974) and site-specific correlations for a particular soil unit can be developed. This equation is not suitable for highly sensitive or structured clays.	

SILTS	S_u ϕ', c'		• Due to the wide range of behaviour silts exhibit (i.e. sand-like to clay-like), methods for estimation of shear strength also vary. In general, the shear strength for high plasticity silts can be estimated as for clays and the recommendations for clays in this table apply. The shear strength of non-plastic to low plasticity silt may be better estimated using the recommendations for sands, however the fine-grained nature makes the drainage conditions during in situ strength testing uncertain, and there are some special considerations for these types of silts as outlined below: • It is difficult to obtain "undisturbed"[3] samples in non- to low plasticity silts. • Non- to 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. Application of a back pressure in CU triaxial tests in excess of that required to obtain saturation avoids cavitation effects as a negative pore pressure will not develop. • 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 handheld shear vane to measure the shear 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, at the tested level of saturation 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 and saturation when measured is similar to those in the model.
GRANULAR SOILS – SANDS AND GRAVELS	ϕ'	In situ Shear Strength Testing and Correlations	• The shear strength of granular soils is often best estimated from: – correlations with in situ tests (SPT, CPT, dilatometer) and/or – correlations with material properties (gradation, relative density, and confining pressure). • In many cases reliable values of effective friction angle can be derived by using in situ tests to estimate relative density, and then using correlations such as those outlined in Duncan et al (2014) between friction angle, relative density and PSD. • Correlations between cone resistance and relative density given in the literature are primarily based on studies of clean quartz sand of fairly uniform grading and may not be valid for sands of different particle hardness and grading, such as sands derived from volcanic material which are seldom clean, hard-grained material. For pumice sand which comprises vesicular particles that are easily crushed, the cone resistance is not significantly affected by the relative density (refer Wesley (2010)b, Section 9.5).
	ϕ'	Laboratory Tests, CDD and CD triaxial tests.	• Laboratory testing of granular soils can be problematic due to the difficulty in obtaining "undisturbed"[3] samples for laboratory testing, and limitations on grain size that can be accommodated by the laboratory equipment. Specialist sampling techniques to collect undisturbed samples of granular soils have been developed but are not typically employed in routine practice. • Direct shear testing (DDS) on reconstituted samples, compacted to field density and consolidated to field overburden stresses can be useful alongside in situ testing to define the soil shear strength.
	S_r	Correlations and Laboratory Testing	• The undrained shear strength of liquefied sands (S_r) in earthquakes is routinely estimated using correlations of the residual shear strength ratio (S_r/ overburden, σ'_{vo}) with normalised CPT tip resistance or SPT blowcount. Commonly used older correlations include Olson & Stark (2002), Seed & Harder (1990), and Idriss and Boulanger (2008). More recent correlations include Weber (2015) and Robertson (2021). Module 3 provides further discussion. • While not routinely carried out, the shear strength properties and behaviour of granular soil in earthquakes can be assessed using cyclic laboratory tests. While these tests remain mostly in the research realm, there is potential for significant value to larger projects where liquefaction is driving design.
ALL SOILS	S_u ϕ', c'	Typical values	• Typical values of soil shear strength parameters are commonly provided in geotechnical texts such as those outlined in Section 10.2. For example, a rough indication of the range of typical values of c' for clays is given in Wesley (2010a) Table 9.3. These values are often general and therefore crude and often of little practical use in slope stability studies. In some cases, typical values may be developed for specific, well-studied soils deposits. These values are likely more reliable as they relate to specific deposits and conditions. The suitability of reliance on these types of deposit-specific shear strengths depends on how well-studied the soil deposit is and the scatter in the data. In many cases, reliance on these values should be avoided for all but preliminary estimates. As a minimum, sensitivity analyses should be performed to study the changes in the margin against instability due to uncertainties in the shear strength values.

[1] Refer to Robertson & Cabal (2015)

[2] ΔPI = distance above or below A-line on plasticity chart (i.e. $\Delta PI = PI - 0.73(LL-20)$).

[3] No sample is ever truly undisturbed.

11 GROUNDWATER MODELLING TECHNIQUES

11.1 INTRODUCTION

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 the Engineering Geological Model (EGM).

As will be noted in Section 13, slope stability modelling should consider the pore-water pressures in both the Long-term Static condition (which includes the typical wet-season ground water conditions) and the High Ground Water condition.

In this guidance the "High Ground Water" condition is defined as:

The ground water pressure distribution in the slope that is likely to cause the factor of safety to reduce to a value that is only reached once every five to ten years.

11.2 CALCULATING PORE WATER PRESSURES

There are multiple ways of calculating pore water pressures as listed below.

- Approximation based on the piezometric surface or water table. When approximating pore pressures based on the piezometric surface, and that surface has little or no slope, pore pressures may be estimated as the pressure head h_p times the specific weight of water (i.e. $u_w = \gamma_w * h_p$ – Duncan et al, 2014). The pressure head can be taken as the depth below the water table in unconfined aquifers, or the depth below the piezometric line representing the hydraulic head in confined aquifers. However, this method will significantly overestimate the pore pressures if the water table is on anything more than a gentle slope because, if used in a slope stability model, it implies vertical equipotential lines, whereas the equipotential lines slope if the water table slopes as illustrated in Figure 39.
- Where the phreatic surface is straight but not horizontal, h_p can be estimated as $z \cos^2\beta$ where β = slope of the water table and z =vertical depth below the water table (Duncan et al, 2014). This concept is sometimes called the "phreatic correction" and is illustrated in Figure 39. It may be useful for hand calculations and is handled directly by stability analysis software – although the user should check that it is turned on in the software -see the box below for details.
- 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. Pore pressures are calculated at each finite element nodal point and interpolation schemes are used to calculate pore pressure at the base of failure surface slices.
- For slopes with complex pore water pressure distributions, when the future conditions are not known, slope stability software packages (Slide2, SLOPE/W used in partnership with SEEP/W) allow specification of pore water pressures in a variety of ways beyond a simple piezometric line.

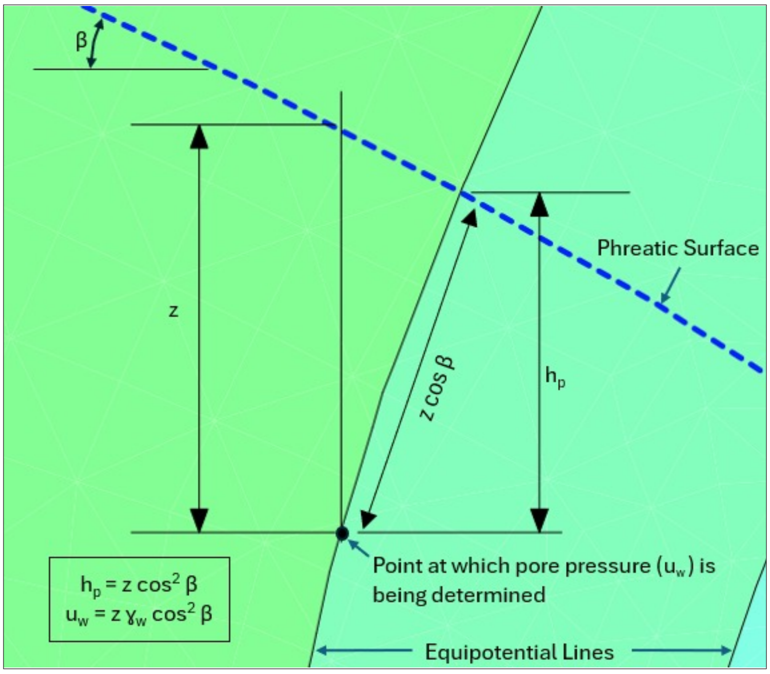


FIGURE 39: Pore pressure estimations for inclined phreatic surface (adapted from Duncan et al, 2014)

- Performing simple hand drawn flow nets is useful for validating complex computer models. They can be drawn quite quickly especially if it is assumed that the horizontal and vertical permeabilities are the same. They can quickly identify issues such as what changes in groundwater flow might arise from changes in water levels.

Estimating pore water pressures based on the piezometric or phreatic surface provides a good approximation in many cases. However, where the phreatic surface is steeply inclined (i.e. groundwater flow becomes strongly non-horizontal) such as through a low permeability dam core[3], these approximations can be unconservative. In these cases, it is better to use finite element seepage analysis (Duncan et al, 2014).

When the pore pressure distribution is complex, there are other modelling techniques including:

- 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 can be specified in slope stability software using a matric suction friction angle (ϕ^b) which defines the additional component of shear strength due to matric suction as discussed in Section 12.1.
- Excess pore pressures – excess pore pressures can be generated from applied loads in some software packages.

It is possible to assess the pore pressure distribution of slopes by modelling the rainfall infiltration, and this is most successful in slopes with relatively homogeneous permeability – see Fell et al (2000) for details and Wesley (2010b) for a worked example.

Seasonal fluctuations and extreme groundwater levels should be checked in the analysis to understand the sensitivity of stability to plausible changes in groundwater levels. High sensitivity may identify the need to consider drainage measures to prevent future uncontrolled increases in water pressures with consequential stability reductions.

[3] This is the example provided in Duncan et al. We emphasise that Unit 3 does not purport to be a dam guidance document.

PHREATIC SURFACE VS PIEZOMETRIC SURFACE VS WATER TABLE VS ZONE OF SATURATION – WHAT'S THE DIFFERENCE?

The phreatic surface is the line of zero pressure at the upper boundary of the seepage region (Duncan et al., 2014). In soil mechanics, when we write "zero pressure" we actually mean "atmospheric pressure".

The water table is just another name for the phreatic surface.

The piezometric surface is the surface defined by the piezometric levels observed in a series of piezometers. Below a piezometric line, the pore water pressure is equal to the depth below the piezometric line multiplied by the unit weight of water (Duncan et al., 2014). If there is no confining stratigraphy above the flow, the soils are in an unconfined aquifer and the piezometric line and the phreatic surface are the same. If the aquifer is confined there isn't a phreatic surface and the piezometric line represents a hypothetical 'confined groundwater table' referred to as the potentiometric surface. The potentiometric surface is elevated above the saturated zone of the confined aquifer.

Slope stability software Slide2 distinguishes between the water table and the piezometric surface, in that if a water table lies above the ground surface, a ponded water region is created, whereas this does not occur if the piezometric surface lies above the ground surface.

The water table (or phreatic surface) is not necessarily a boundary below which the soil is fully saturated and above which it is unsaturated or partially saturated. In theory, all soils below the water table will be saturated (although some P-wave testing in Christchurch has suggested otherwise). But it is likely, in fine-grained soils, that some soils above the water table will also be saturated, and the finer the grain size, the further above the water table the saturated soils will exist (Wesley, 2010a). Indeed, according to Wesley, clays above the water table only become unsaturated near the ground surface, due to evaporation.

11.3 BACK-ANALYSES USING Ru GROUNDWATER PARAMETER

An approximate method of back-analysis of failed shallow soil slopes using the pore pressure parameter Ru is often used in slope stability software. This Ru parameter assumes that the groundwater pressure is a proportion of the soil depth (or slice in stability software) at the time of failure. This mimics the situation of high intensity rainfall where rainwater infiltrating the sloping ground is retarded by a permeability contrast with underlying soils and/or a perched groundwater condition results from high water infiltration.

If possible, strength parameters obtained from back analyses should be used in assessing existing slopes when using Ru, either from an existing failure (FoS=1.0) or from a non-failed slope with a FoS ≥ 1.1, along with good judgement and experience (refer to Section 18 for more information about back-analysis).

The Ru parameter is defined as the ratio:

$$Ru = \Upsilon_w h_w / \Upsilon_s h_s \qquad \text{Equation 16}$$

where Υ_w = density of water
h_w = height of water (in the slice, in the software)
Υ_s = density of soil
h_s = height of soil (in the slice, in the software)

An Ru value (typically 0.3 to 0.5) is assumed and the failure then back-analysed to determine critical strength parameters. These parameters are then carried forward into analysis of long term, short term and seismic conditions for slope arrangements and remedial works.

Note, a 'groundwater at surface' condition ($h_w = h_s$) for a soil density of 1.8 t/m^3 and a water density of 1.0 t/m^3 correlates to an Ru of 1/1.8 = 0.55. A lower Ru can be chosen to reflect groundwater flow down the slope (rather than a standing 'water reservoir' condition) and the uncertainty of the model. A lower value of Ru is more conservative for a back-analysis as it results in lower back-analysed strength parameters. Good judgement and experience are required for this type of analysis.

11.4 PORE PRESSURE ESTIMATES AND SLOPE STABILITY ANALYSIS REQUIRED FOR DIFFERENT SLOPE TYPES

Despite the variety of tools available, as described above, estimating the pore water pressures within a slope and the potential for consequent slope instability is not straightforward. Wesley (2010b) notes, with respect to residual soils, that "*Neither the slope characteristics nor the rainfall pattern will ever be known with the degree of reliability needed to make realistic prediction*" while consoling us that "*in rare situations, where past records enable soil parameters to be determined from back-analysis, and where rainfall records are comprehensive, it might be possible to make predictions that are not entirely unrealistic.*" Duncan et al (2014) are slightly less pessimistic, merely warning that "*groundwater and seepage conditions are often not well known.*"

HOW DO SLOPE/W AND SLIDE2 CALCULATE PORE WATER PRESSURE

SLOPE/W and Slide2 are amongst the most used slope stability programs in New Zealand. In both these programs, the user can choose the method by which the pore water pressure distribution is calculated. Options available are listed below.

Method	SLOPE/W	Slide2
By calculating the vertical distance below the piezometric line.	Available	Available
For sloping profiles, multiplying the result from the vertical distance approach by $\cos^2\delta$	There is a "phreatic correction" checkbox to turn on or off.	The default is that the phreatic correction is off. To enable it, select material properties, water parameters tab, and change Hu type to "Automatically Calculated". This must be done for each material type below the water table.
Ru coefficients	Available	Available
B-bar coefficients	Available	Available
Spatial function	This is where the user defines the pore water pressure at discrete known points, and SLOPE/W calculates the pressure distribution everywhere.	Not available
Using a seepage analysis	Available using SEEP/W for steady-state or transient analysis.	Available through FEA tool within Slide2.

Refer to the user manual (GEO-SLOPE International, 2021) for further information on SLOPE/W and SEEP/W. For Slide2, consult the Rocscience website.

The following list of appropriate analyses by slope type closely follows that presented in Fell et al (2000) and considers the likely lack of available knowledge. Fell et al (2000) and the text below refer to limit equilibrium and numerical methods, types of slope stability analyses which are described in Section 14.

11.4.1 Shallow landslides in natural slopes (anticipated or existing)

For landslides less than about 5 m deep, the factor of safety can be particularly sensitive to input assumptions regarding shear strength and pore pressures. If using LEM for shallow slides, geoprofessionals should carry out sensitivity analyses and, where possible, consider historic performance of the slopes to assess the likelihood of future landsliding.

11.4.2 Medium landslides in natural slopes (anticipated or existing)

Medium landslides are greater than 5 to 10 metres in depth and up to several hundred thousand cubic metres in volume. For slides of this size, it is reasonable to carry out limit equilibrium analysis of stability. These landslides are usually just as complex hydrogeologically as shallow slides. Instrumentation may be helpful to establish the ground water conditions – considering the challenges in instrumentation noted in Section 11.5.2.

11.4.3 Large landslides in natural slopes (anticipated or existing)

For slides of this size (more than 1 million m^3) limit equilibrium analyses should be carried out, often backed up by numerical analyses to model the internal deformations.

Instrumentation should be carried to establish the ground water conditions – considering the challenges in instrumentation noted in Section 11.5.2.

Cornforth (2005) states that the best way to obtain peak groundwater levels for existing landslide analysis is to measure them directly over one or more winter seasons using vibrating wire piezometers and an automatic data acquisition system. In practice, this opportunity is not often available because (i) landslides often must be remediated before the next wet season, and (ii) of the high cost. On large landslides however, sufficient time and funds may be available. Once the piezometers have been installed, continuous monitoring with remote real-time observation is fast becoming normal practice at an acceptable cost - savings are made in not having to return to site on a regular basis to download measurements - and trigger warnings of excessive groundwater response can be set.

11.4.4 Cut slopes

The analysis of stability of cut slopes should be done by limit equilibrium methods, sometimes supported by numerical analyses to determine stresses and deformations, and to assess the likelihood of progressive failure due to shear strain induced weakening along the potential failure surface. Pore pressures in cut slopes are complex, with the problems associated with any natural slope, compounded by the effects of the negative pore water pressures due to unloading (excavation).

For existing cuts, the installation and monitoring of piezometers, and relating these to the hydrogeological conditions and the rainfall and evaporation which control the pore pressures, is the only way to get a reasonable estimate of conditions. However, it must be recognised that cracks and open joints, etc., may allow high transient pore pressures to occur in the slope and these are likely to be missed by the instruments.

For new cut slopes, it is usually difficult to accurately predict the magnitude and lateral distribution of pore water pressures. It is often necessary to rely on an understanding of the hydrogeology and observations of precedent in similar slopes on or near the site. The ideal situation is to monitor the pore water pressures as the slope is constructed, and install borehole drains or other measures to reduce these pressures if they remain too high after excavation.

11.4.5 Fills

The analysis should be done by limit equilibrium methods. In some cases, e.g. in the investigation of unusual deformations which have been experienced by the slope, it will be necessary to use numerical methods. Where strain weakening and progressive failure is likely, numerical methods should be used.

For existing fills which do not have drainage layers, the only reliable way to estimate pore pressures is to install piezometers and monitor them as detailed above for cuts and natural slopes. For new fills, where seepage may flow into the fill from the slope on which it is constructed, the slope should be engineered with a drainage layer to control the pore pressures. If there is significant uncertainty in the design groundwater conditions, a more conservative approach to analysis and design should be applied.

11.5 CHALLENGES IN ASSESSING PORE WATER PRESSURE DISTRIBUTIONS

11.5.1 Challenges in modelling

For natural slopes, and slopes built on or into natural slopes, it can be difficult to model the piezometric conditions with accuracy, because the slopes include preferential flow paths for water (such as in open jointed rock or sandy beds), perched water tables, and the pore water distribution is affected by construction and rainfall.

For fills the situation is simpler, but it is still not easy to predict the pore pressures with confidence because of the effects of compaction, giving potentially high (but difficult to predict) ratios of horizontal to vertical permeability (Fell et al., 2000), the effects of partial saturation, and dependence of permeability on the confining stress.

It is possible to assess the pore water pressure distribution of slopes by modelling the rainfall infiltration, but often the real situation is too complex to model successfully – the real slope has roots, root holes, fissures, cracks, soil pipes, and varying soil permeability. Alternating layers, differing significantly in permeability, create perched and pressurized bodies of groundwater which are frequent causes of shallow slope failures. Figure 40 (a) and (b) present schematics of idealised and real slope conditions. Especially for shallow landslides, geoprofessionals using LEM need to be aware of the potential difference between the reality and the model and carry out sensitivity analyses and/or assess historical slope performance, where applicable.

For medium and large landslides, Fell et al (2000) consider that LEM analysis becomes more worthwhile, although care is still required.

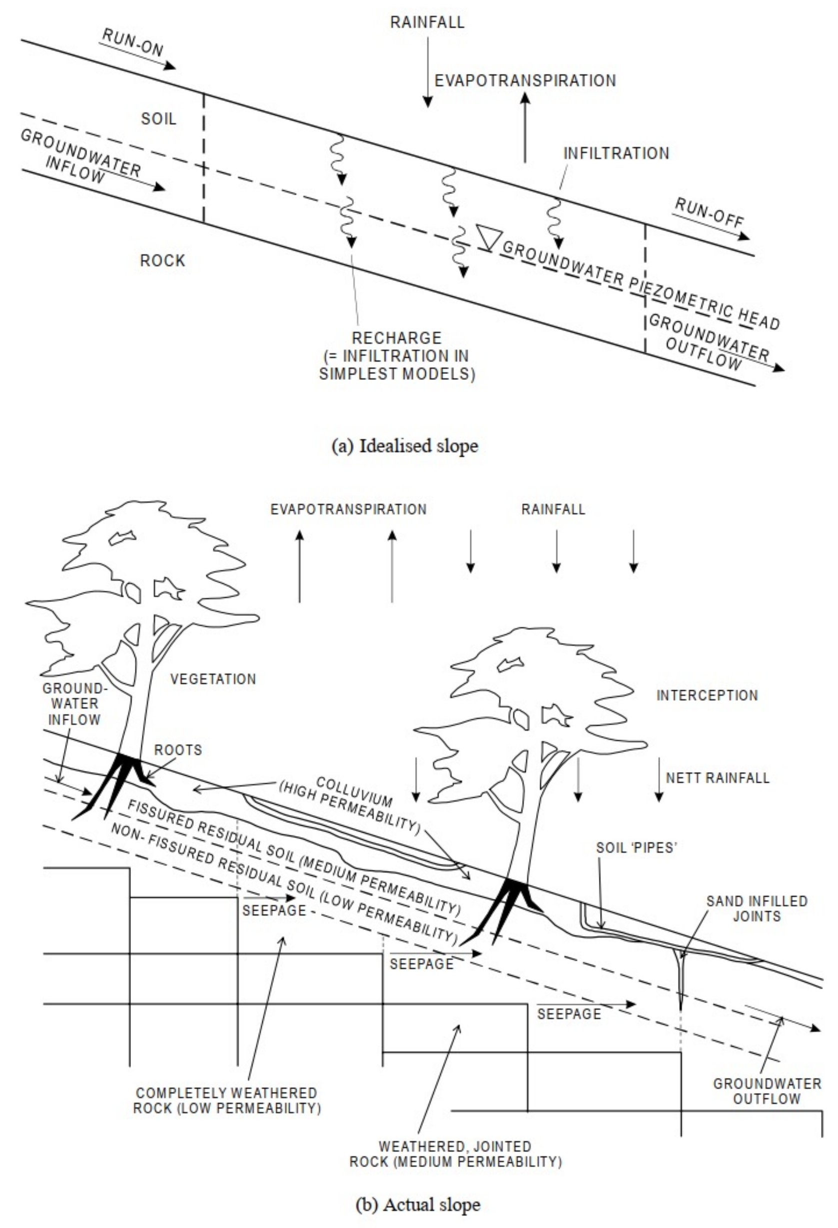


FIGURE 40: Hydrogeological conditions in shallow landslides on natural slopes (Fell et al, 2000).

Wesley (2010b) gives an example of how to calculate pore water pressures in a homogeneous clay using a transient seepage analysis. It requires an assumption that the permeability is similar throughout the slope, which would need to be assessed by the geoprofessional.

It may be tempting, in the absence of much information on ground water levels, to assume for the High Ground Water conditions that the water table sits at the surface, but this may be over conservative. A transient seepage analysis, such as that described in Wesley (2010b), despite its many simplifying assumptions, is sensible to help evaluate plausible upper limits for the water table. If, however, there is insufficient information to carry out a transient analysis and/or there remains significant uncertainty, then the assumption that the ground water table is at the surface is appropriate.

In cold climates, the effects of snowmelt may also need to be considered as a source of water infiltration.

For medium and large landslides, the input of a hydrogeologist and/or hydrometeorologist may be needed. They will consider the rainfall, infiltration rates, catchment size, aquifer storage and permeability to assess potential pore water pressure distributions. One of the challenges in this type of analysis is the variability of permeability with confining stress and degree of saturation. Another is the influence of the underlying rock - it could be either a source of water pressure acting on the base of the lower permeability soil slope or a drain resulting in decreasing water pressure with depth, and all possibilities in between.

11.5.2 Challenges in instrumentation

To develop a hydrogeological model, it is often necessary to instrument the slopes with piezometers and monitor them over a sufficiently long period to establish the relationships between pore pressures and rainfall, while also reading them often enough to detect the changes in pore pressures which occur in the slope in response to rain.

That can be challenging because (as per Fell et al, 2000):

- Slopes can be heterogeneous, and pore pressure response may vary significantly over the slope, even in piezometers quite close to each other.
- It is useful to estimate the critical rainfall duration (although it probably won't have a unique value) along with the effect of antecedent rainfall. However, sometimes only 24-hour rainfall data is available, when shorter duration rainfalls are more critical. This is variable across New Zealand, but with some Councils' databases, rainfall is captured as time per millimetre of rain, hence it is possible to generate records of rainfall per hour (or shorter duration) for storms.
- A long period of record is needed - possibly several years of data - with rainfall recorded at short (for example 15-minute) intervals.
- To establish a sound understanding of how the slope reacts to pore pressure and rainfall, data on slope movement is required to compare to rainfall and piezometric pressure responses, all across the same time period.

11.6 ASSESSING LIKELIHOOD OF SLOPE INSTABILITY BASED ON RAINFALL

Because of the challenges in modelling pore water pressure distributions, as discussed above, it may be useful, particularly for shallow landslides, to directly correlate the likelihood of slope instability to rainfall, using past performance of the subject slope, or similar nearby slopes, if satisfactory rainfall records are available.

Because of the variability in soil permeability parameters, the relationship between rainfall, pore water pressure increases, and landslide triggering cannot be easily established and can vary considerably from one slope to another, and one region to another. Various researchers have derived a variety of conclusions on the relative importance of rainfall intensity, rainfall duration and antecedent rainfall, with the meta-conclusion being that landslide activity results from a combined effect of antecedent rainfall and rainfall intensity (Fell et al., 2000). Rahardjo et al (2007) carried out numerical modelling that indicated that slopes with high permeability ($k \geq 10^{-5}$ m/s) are most likely to be affected by short-duration rainfall, with the failure of low permeability ($k \leq 10^{-6}$ m/s) slopes closely related to long-duration or antecedent rainfall.

Glade et al (2000) carried out a study based on historical records of landslides and daily rainfall records across Wellington, Hawke's Bay and Wairarapa. They produced graphs for those regions, showing the relationship between daily rainfall, Antecedent Daily Rainfall Index and the probability of landslides occurring within that region (Figure 41 a, b and c). Antecedent Daily Rainfall Index is a measure of the rainfall that has occurred on previous days, which places less weight on rainfall the longer ago it occurred. Glade et al reference a previous similar study for Otago Peninsula (Crozier & Eyles, 1980), which is produced in Figure 42.

It can be seen from Glade's work that both daily rainfall and antecedent rainfall are important. Glade et al (2000) inferred that, in Wellington and Hawke's Bay, daily rainfall is slightly more important than antecedent rainfall, whereas in Wairarapa both are of approximately equal importance. Glade et al (2000) also observed that of the three regions in their study, Wellington appears to have the slopes most suspectable to landsliding – i.e., Wellington needs the lower amount of rainfall for landslides to occur.

Justice et al (2018) studied the effects of daily rainfall and antecedent rainfall on the steep greywacke and colluvium slopes near Kaikoura in 2017 and 2018, following the M7.8 November 2016 Kaikoura earthquake. They concluded that:

- Active landslides are prone to further debris movement in small rain events, proportional to the antecedent rainfall condition and the amount of rainfall on the day of slope failure.
- Antecedent rainfall has a strong influence on the amount of further rain required to trigger slope movement.
- Few failures are initiated under heavy rainfall with low antecedent rainfall.
- Relatively large landslides are commonly initiated following the cessation of rain under high antecedent rainfall conditions.

Justice et al (2018) produced a similar graph to those of Glade et al (2000) and Crozier & Eyles (1980) – see Figure 43 - albeit with project action trigger levels rather than lines of probability.

It appears that the different researchers have used slightly different formulations of Antecedent Daily Rainfall Index – indeed, Glade et al (2000) considered that a different formulation is required for different regions.

As Glade et al (2000) state, "*insurance companies or regional government may be able to use these probability figures to define the appropriate level of either preparedness or … estimate … costs resulting from landslide damage*". For individual projects, these charts or newly created site-specific charts could be used in the following cases:

- For an existing landslide with a well recorded history of previous sliding, charts could be used to derive trigger levels and action plans for future sliding, if rainfall above certain thresholds is forecast.
- Projects with large areas, particularly long infrastructure projects on steep ground (such as the NCTIR project, for which the Justice et al, 2018 chart was created) could use charts to develop triggers levels and action plans.

It is unclear, however, that such charts would be of any practical benefit when designing new slopes or physical mitigations for existing slopes, except to remind geoprofessionals of what they should already know – that new rainfall, causing a weakening of the near surface soils, and prior rainfall, causing an increase in the ground water table, are both important and should be addressed in slope stability modelling and design (see also Figure 45).

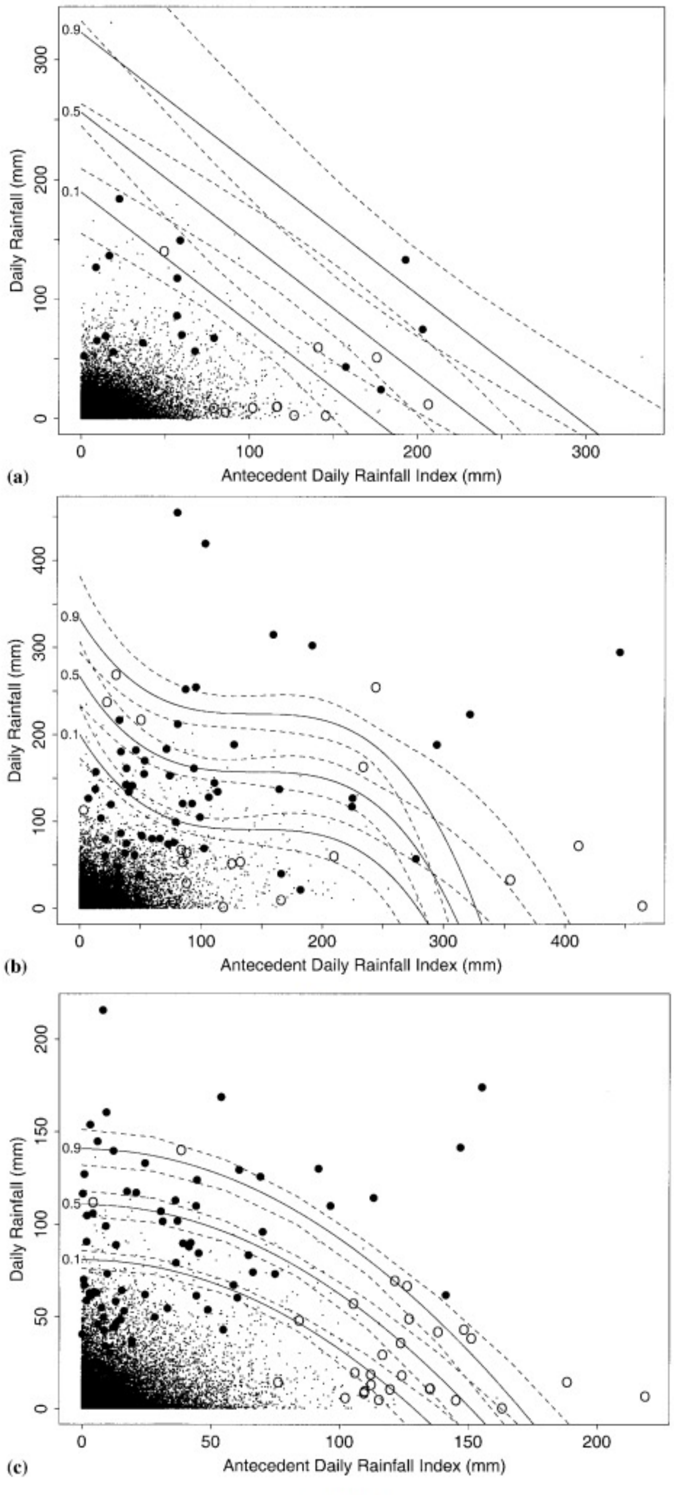


FIGURE 41: Probability of landsliding based on daily and antecedent rainfall for (a) Wairarapa (b) Hawke's Bay and (c) Wellington[4] (Glade et al, 2000).

4 Calculation is based on rain days (> 0.1 mm) only. Large dots relate to rainfall which triggered landslides, open circles relate to rainfall with probable landslide occurrence, and small dots relate to rainfalls which did *not* trigger landslides. The graphs have different scales. Confidence intervals are indicated for each probability curve by dashed lines.

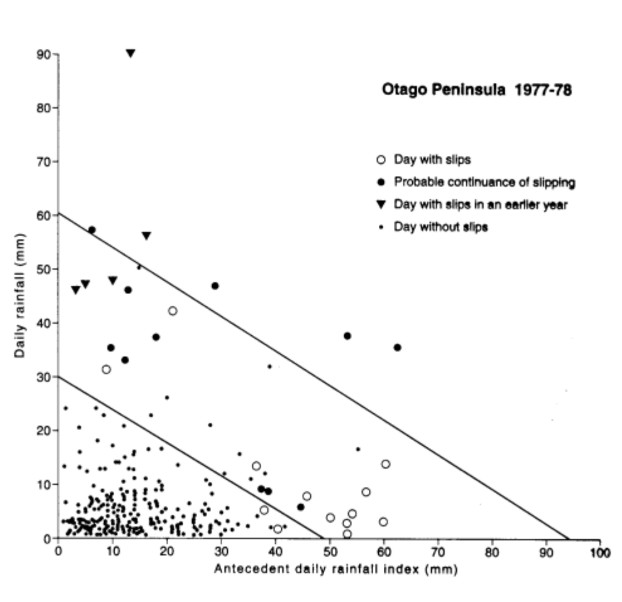


FIGURE 42: The Antecedent Daily Rainfall Model applied to landsliding episodes in Otago Peninsula, New Zealand (Crozier and Eyles, 1980).

11.7 CONCLUSIONS

There are numerous ways in which LEM software can model pore pressures. The difficulty is in understanding the actual pore pressure conditions likely to occur within the slope. Owing to the complexity of natural slopes, it can be difficult to model the piezometric conditions with accuracy.

When assessing the potential for small landslides on existing slopes, geoprofessionals should carry out sensitivity analyses and, where possible, consider past behaviour of the subject slope or similar nearby slopes to make forecasts of future behaviour, allowing for the possibility that future storms may be more intense than previous storms.

When assessing the potential for medium and large landslides on existing slopes, or existing landslides, the use of hydro-geological modelling to estimate plausible pore water pressure distributions is recommended but those distributions must be confirmed on site, and engineering measures installed if needed to lower the water table.

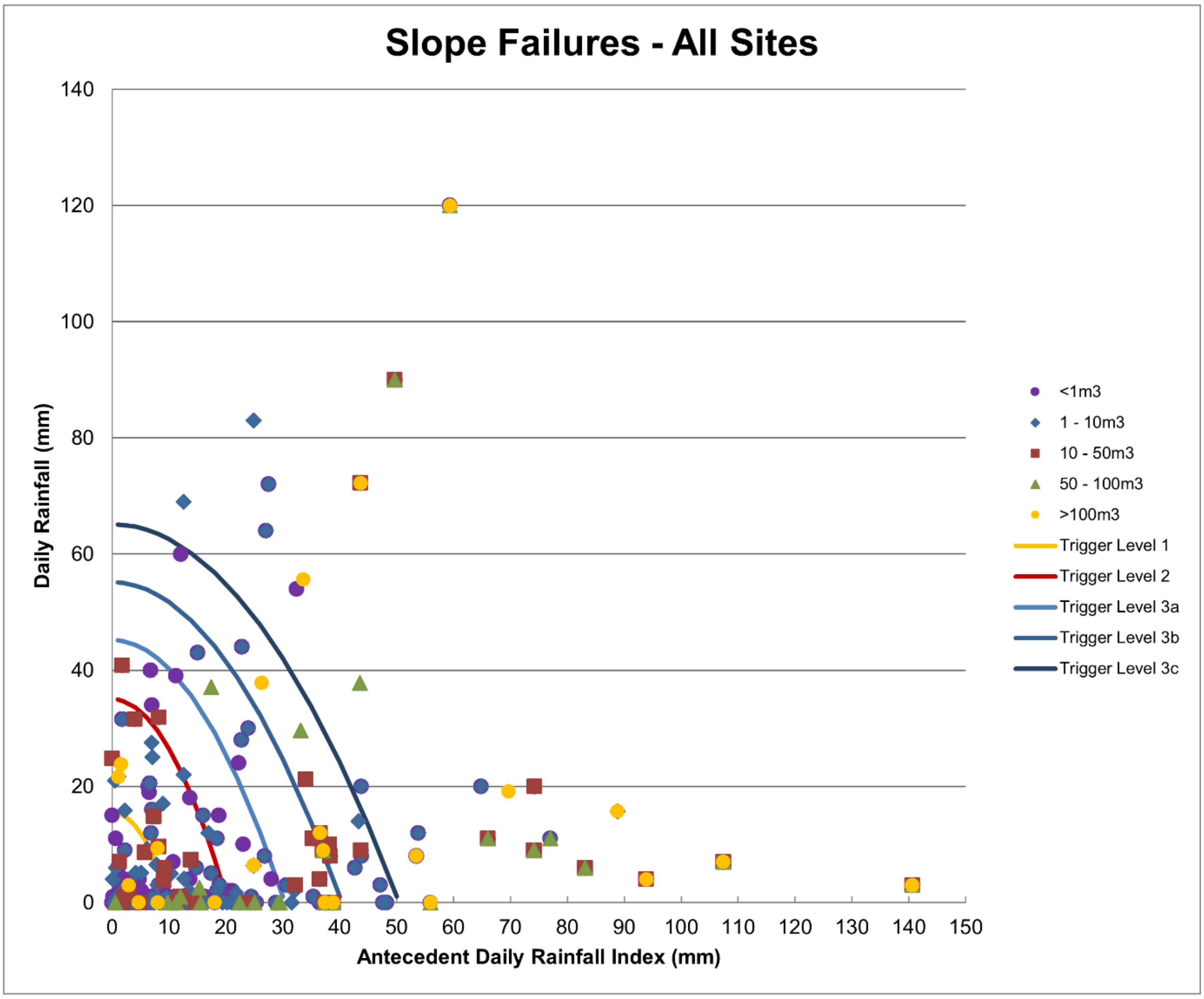


FIGURE 43: Observed rainfall induced failures and trigger levels, Kaikoura, 2017-2018 (Justice et al., 2018).

Stability of new cut and fill slopes can generally be assessed using LEM methods. For cut slopes, if the consequence of shallow slope failure is unacceptably high, mitigations should be designed. In all cases where stability is assessed using LEM, the uncertainty in any pore water pressure distribution must be acknowledged by using sensitivity analyses.

In this guidance we have used the term “High Groundwater Condition” to describe adverse pore pressure conditions that the slope is reasonably likely to be subject to over the design life. This includes changes to both the groundwater table and soil strength due to loss of suction from water infiltration. These adverse conditions typically result from rainfall because it is the most common cause of landslides in New Zealand. Where LEM are used to assess the High Groundwater Condition, particularly in fine-grained soils the geoprofessional must consider:

- **The effect of rainfall infiltration on matric suction and the loss of soil strength of the upper soils.** Where the depth of saturation from rainfall infiltration can be estimated, this soil depth can be modelled as fully saturated, with c’ equal to or near zero, or with a saturated undrained shear strength. Where the depth of saturation cannot be estimated, saturated soil strengths should be assumed in all low permeability soils. The geoprofessional should consider how the available strength tests capture the strength of the saturated (or higher moisture content) state of these soils. It is unlikely that in situ testing has captured the adverse strength conditions the geoprofessional should model in a High Groundwater scenario. For details, see Section 12.
- **The likely increase in the water table elevation.** The water table does not always need to be modelled at the ground surface.

12 PARTIALLY SATURATED SOIL

12.1 SHEAR STRENGTH OF PARTIALLY SATURATED SOILS

Many soil shear strength analyses consider soils to be either saturated or completely dry. However, soils above the water table can be partially saturated[5], with substantial moisture held within the voids by capillary effects or suction (Sivakugan, 2021). In general, unsaturated soils have a higher shear strength than the same soil saturated.

When a soil matrix isn't fully saturated and contains both air and water, capillary rise occurs in the narrower soil pores, creating a curve or meniscus in the water at the air-water boundary. This curved surface results in a pressure imbalance: the capillary-held water pressure is less than atmospheric pressure, which manifests as soil suction or negative pore water pressure. This soil suction results in additional soil shear strength.

The relationship between the moisture content and the suction is referred to as the Soil-Water Characteristic Curve (SWCC). Typical examples are provided in Figure 44, which shows that the largest suction values are developed in silts and clays.

5 In this guidance, "partially saturated" and "unsaturated" mean the same thing.

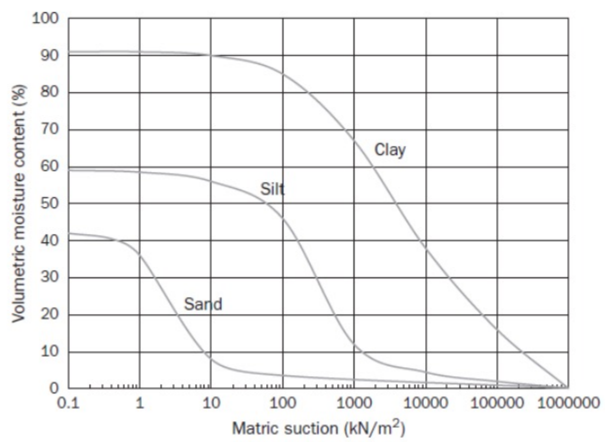


FIGURE 44: **Typical SWCC for clayey, silty, and sandy soils (Sivakugan, 2021). Alternatively, degree of saturation can be plotted on the vertical axis.**

Various equations have been proposed to account for suction in partially saturated soils.

Fell et al (2000) proposed that unsaturated shear strength could be expressed as follows (based on the work of Bishop and many others, and called the Bishop equation herein):

$$\tau = c' + \sigma' \tan \phi' \qquad \textbf{Equation 17}$$

$$\sigma' = (\sigma - u_a) + \chi\,(u_a - u_w) \qquad \textbf{Equation 18}$$

$$\chi = [\,(u_a - u_w) / (u_a - u_w)_e\,]^{-0.55} \qquad \textbf{Equation 19}$$

Where:

τ = soil shear strength

c' = effective stress cohesion (the same as for soil when saturated)

σ' = effective normal stress

ϕ' = effective stress internal friction angle (the same as for soil when saturated)

σ = total normal stress

u_a = pore air pressure

u_w = pore water pressure, which is negative in the unsaturated zone, and hence $-u_w$ in the equation is positive

χ is a measure of saturation and takes the value of 1 for saturated soils and 0 for dry soils

$(u_a - u_w)$ is the suction (commonly called the matric suction), sometimes denoted as "s".

$(u_a - u_w)_e$ is the air entry suction – the suction needed for air to enter the soil (and thus for water to leave) during de-saturation. It is inversely proportional to the pore diameter. Therefore, clays have larger air-entry values than sands (Sivakugan, 2021).

Wesley (2010a) notes that, in soil slopes, u_a usually approximates atmospheric pressure, while the pore pressure will be negative. As, in soil mechanics, atmospheric pressure is set to zero, u_a is usually approximately zero. Equation 18 therefore reduces to:

$$\sigma' = \sigma - \chi u_w \qquad \textbf{Equation 20}$$

Since χ is less than 1 for partially saturated soil, the effect of the air in the partially saturated soil is to increase the effective stress relative to saturated soil.

Fredlund et al. (1978) proposed that unsaturated shear strength could be expressed as follows:

$$\tau = c' + (u_a - u_w) \tan \varphi^b + (\sigma_n - u_a) \tan \varphi'$$

Equation 21

Where:
ϕ^b = matric suction friction angle. Although the "b" is usually presented as a superscript it does not appear to be intended to represent an exponent. Yates & Russell (2023) dispense with the "b" and call this ϕ'. Rahardjo et al (2007) select the same value for ϕ^b and ϕ' for their computer modelling, suggesting that they believe the parameters typically have similar or identical values. GEO-SLOPE (2021) state that, for practical purposes, ϕ^b can be taken to about 0.5 ϕ'. Wesley (2010a) quotes Fredlund & Rahardjo as saying that the value of ϕ^b is usually found to be between 15° and 20° but theoretically could equal 45°.
σ_n = total normal stress
$(\sigma_n - u_a)$ is referred to as net normal stress

Equation 21 is comparable to Equation 14, but with the addition of the term $(u_a - u_w) \tan \phi^b$, which is the shear strength derived from matric suction.

According to Fell et al (2000), there are several difficulties in the practical application of the Fredlund equation, including that ϕ^b varies with suction (and hence with degree of saturation), and that extensive and time-consuming specialist laboratory testing in the unsaturated state is required to provide useful values of the parameters.

Fell et al (2000) consider the Bishop equation to be more practical than the Fredlund equation, because it reduces the number of parameters, eliminates the need for laboratory testing in an unsaturated state, and enables saturated and unsaturated states to be considered simultaneously. Nonetheless, knowledge of the pore air pressure and air entry suction are required, parameters seldom handled by geotechnical engineers. Most importantly, an understanding is required of the suction present in the field.

Wesley (2010a) considers the Fredlund equation to be less than satisfactory from a theoretical viewpoint because it implies that the increase in shear strength from the negative pore pressure is a cohesive contribution rather than a frictional component.

Yates & Russell (2023) carried out a suite of sophisticated laboratory tests and a 1.5-year programme of field instrumentation on a 95m^2 area of loess slope near Akaroa Harbour. The field monitoring showed substantial variability in suction in the top 2 m, with the suction near the surface varying from 10 kPa during wet conditions to up to 5030 kPa during dry conditions. The shear strength derived from suction, χ s tan ϕ' (effectively the $\chi(u_a-u_w)$ tan ϕ' term in the Bishop equation), peaked at several hundred kPa but appeared to never drop below 3 kPa. Yates & Russell (2023) noted variability in χ s tan ϕ' across the monitoring site and concluded that "*even for seemingly homogeneous sites, local variability in hydraulic conditions may remain*". The work demonstrated that:

(a) Substantial effort, money and time are required to calculate dependable values of suction in the field and

(b) It is not clear that their results are directly transferable to other sites (not even necessarily other nearby loess sites) and hence, unless and until a substantive body of other similar work is carried out and published, site-specific measurement for other projects will be required.

Yates & Russell (2023) note that the term c'+ χ s tan ϕ' can be treated as an equivalent cohesion, which combines the true cohesion (c') with a suction-dependent component (χ s tan ϕ'). Therefore, back-analysis of an intact slope can provide a minimum equivalent cohesion value (c'+χ s tan ϕ') for that slope and, if c' is known from laboratory testing, a minimum χ s tan ϕ' value.

Commonly, the beneficial effects of shear strength due to suction are ignored in practice, which is conservative and appropriate. It is recommended that the beneficial effects are only allowed for if:

- There is a high level of confidence that unsaturated conditions exist in the field and will continue to exist throughout the design life of the structure in the loading condition being considered, and
- There are enough field data or laboratory data or comprehensive regional studies in the relevant soil type available to provide reasonable estimates of the parameters involved.

12.2 CHALLENGES MODELLING UNSATURATED GROUND CONDITIONS

12.2.1 Missing the effects of suction in back-analysis

If a back-analysis is carried out on a slope, based on observed stability of the slope under Long-term Static conditions[6], then high values of soil shear strength, particularly of cohesion, can be derived for soils above the water table. This is termed equivalent cohesion ($c' + \chi$ s tan ϕ') by Yates & Russell (2023), combining the true cohesion (c') with a suction-dependent component (χ s tan ϕ'). Much of that equivalent cohesion, possibly unbeknownst to the geoprofessional, is therefore due to suction. During and after rainfall, the geoprofessional may anticipate that the water table will rise, reducing the effective stress in the soils below the new water table location, and hence reducing the shear strength there. What may be less apparent is that an increase in the water table reduces the suction component (χ s tan ϕ') of strength in soils above the water table, reducing the shear strength of the soils in that area. Hence slope failure of the soil above the water table may be more likely than the geoprofessional believes.

For more information about back-analysis, refer to Section 18.

12.2.2 A wetting front from above as well as from beneath

It is common to model the effects of rain infiltration by an increase in the level of the water table. However, rainfall infiltrating the slope may also increase the moisture content of the near surface soils such that they become saturated, or nearly saturated, despite being above the water table. The apparent cohesion, due to suction, of the near-surface soils then reduces partly or entirely (Lumb, 1975; Yates & Russell, 2023), meaning that shallow instability may occur, and that the instability may not be predicted by the stability model. This phenomenon commonly occurs in slopes where the near surface soils are partially saturated, particularly loess and volcanic soils where partial saturation of the near surface soils is a typical condition. Numerous shallow landslides during storms are visible evidence of this process. Fell et al (2000) consider that prediction of such landslides in a slope stability programme is difficult, and that it may be better to rely on observations of past performance of the slope. This may be a viable approach in existing slopes, but for proposed slopes some computer modelling is likely to be required and, for soils with a high cohesion under Static conditions, it may be appropriately conservative to model the near-surface cohesion as zero (or true cohesion, c', if it has been established) for the High Ground Water case, acknowledging that preventing full slope saturation is impractical.

A schematic of the combined effects of the principles discussed in Sections 12.2.1 and 12.2.2 is presented in Figure 45.

Research demonstrates that the effect of wetting fronts (from above) caused by heavy rainfall, penetrating partially saturated slopes, can reduce soil suction to critically low levels up to 2 m depth, resulting in shallow slope failures (Yates & Russell, 2023; Brand, 1985; Lumb, 1975 and Rahardjo et al, 2007).

Under high groundwater conditions it is recommended that effective cohesion of low permeability soils should be set to zero[7] for drained analysis, and in undrained analysis an S_u representative of the saturated state of that soil be used, unless:

- The cohesion has been demonstrated to reliably exist by either laboratory or field testing under saturated conditions, or
- Prior comprehensive regional studies in the relevant soil type or from detailed field studies in the slopes near the subject site shows that the cohesion for a particular soil type, when saturated, is greater than zero, or
- The cohesion, when saturated, has been demonstrated by back-analysis to exist, and the slope has remained stable for at least the last 10 - 20 years[8], and hence has been subject to sufficient variability in the water table to mimic likely future design conditions. Even then, there may be a possibility that, due to climate change, future storms are more intense than past storms, so some risk of the cohesion reducing would remain. If back-analysis provides the only proof of reliable cohesion, it is recommended that the geoprofessional carry out a sensitivity analysis on near-surface cohesion.

6 Long-term Static conditions are defined in Table 6.

7 Setting the cohesion to zero can result in many slip surfaces of negligible depth having an FoS of less than 1, which can be distracting, as these failures are usually unimportant. In this case, the user should set the minimum slide mass thickness in the slope stability software to a meaningful value, such as 0.5 m.

8 10 - 20 years is an indicative range for guidance. It is based on AGS (2007)'s notion that a slope has become an "existing development" if it has shown non-failure performance "over at least several seasons or events of extended adverse weather, usually being a period of at least 10 to 20 years".

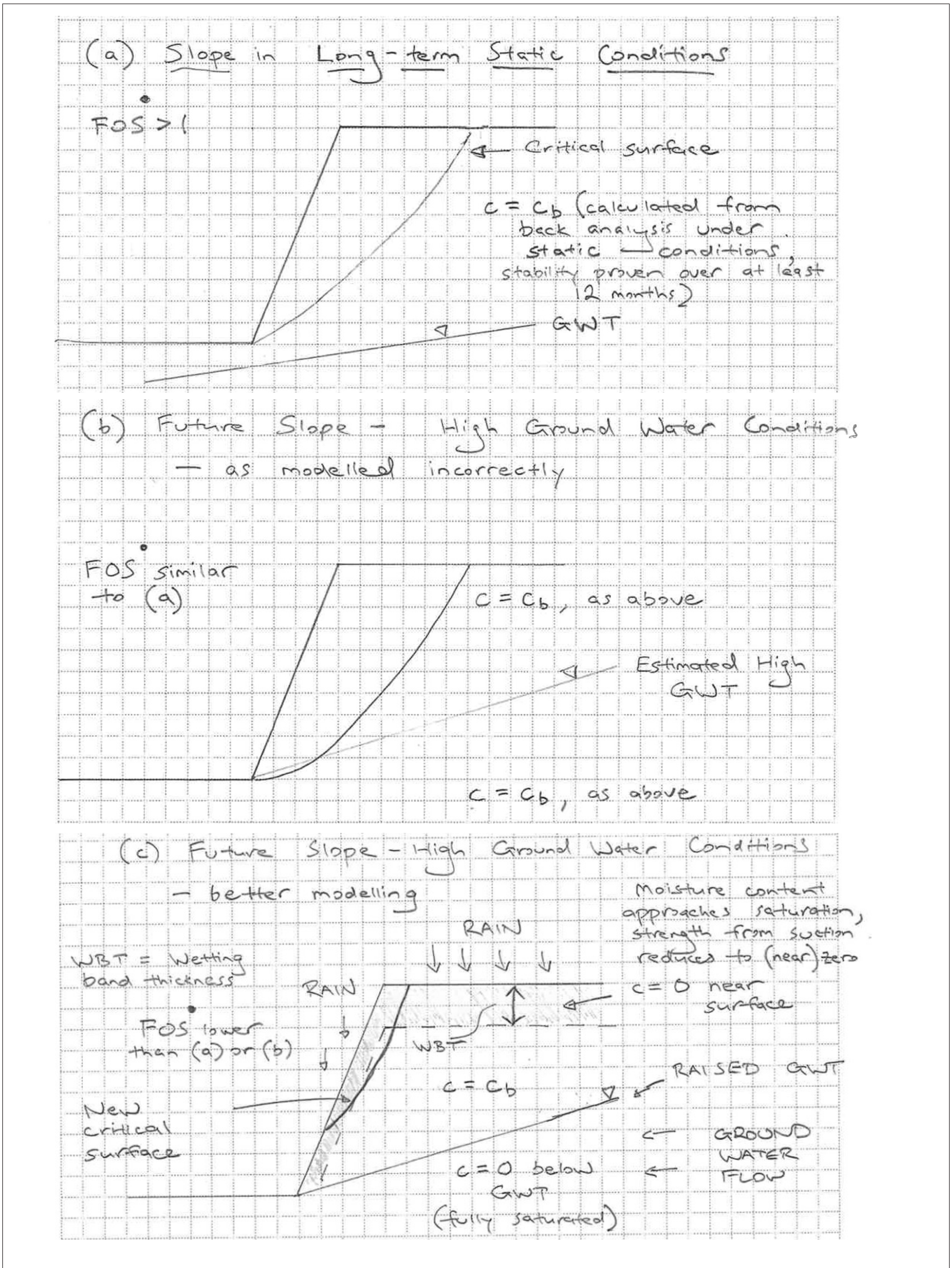


FIGURE 45: Slope stability modelling in High Ground Water Conditions. Models shown are intended to be effective stress models with the friction angle ϕ not shown because it does not change. For case (c), it would also be acceptable to model the wetting band thickness soils with s_u instead of c / ϕ, so long as the s_u was obtained from saturated samples. In case (c), cohesion can be greater than zero in the wetting band thickness if the one of the conditions listed in the following section is satisfied.

It is reasonable to restrict the thickness of the zero-cohesion band under High Ground Water conditions, but only when that thickness is well known from prior regional studies in the relevant soil type or from detailed field studies in the slopes near the subject site.

12.2.3 Testing the shear strength of unsaturated soils

Various methods are available to test the shear strength of soils, and these are discussed in Section 10. This section provides additional considerations for unsaturated conditions.

Laboratory testing to measure the effective cohesion and friction angle is ideal but can be slow and expensive and requires careful sampling to minimise disturbance.

In situ strength tests can provide information on the shear strength of unsaturated ground, but they can produce misleadingly high strength estimates.

A common example is the shear vane test. The United States standard (ASTM, 2001) for the Field Vane Shear Test (ASTM D2573-01) "*covers the field vane test in saturated clay and silt soils for determination of undrained shear strength*" noting that the "*test method is used extensively in a variety of geotechnical explorations to evaluate rapid loading strength for total stress analysis of saturated fine-grained clays and silts*" (underlining added for emphasis). However, the procedure in D2573 makes no explicit requirement of the user to assess the saturation of the soil, only to check the soil type being tested, and its permeability. New Zealand's shear vane guidance (NZGS, 2001) makes no reference to the saturation, or otherwise, of the tested soil – indeed, it includes a model clause for use of the shear vane in an earthworks specification, to test the compaction of fill, strongly implying that the authors expected the shear vane to be used in unsaturated soils. Several researchers (DeAlencar et al., 1988; Pamukcu & Suhayda, 1988) state that the shear vane is for the measurement of the strength of saturated soils, whereas other researchers (Veneman & Edil, 1988; Young et al., 1988) carry out shear vane tests on soils with degrees of saturation varying from 85% to 96%, indicating that they consider that useful results can be achieved using unsaturated samples.

In the experience of the writers of this guidance, vane shear testing in New Zealand is carried out in unsaturated soils far more often than in saturated soils. There is little or no awareness that ASTM states that vane shears should be in saturated soils, not helped by a lack of explicit instruction in that standard and no mention of saturation at all in NZGS (2001). Even researchers carry out vane shears in unsaturated soils.

The problem is that, in unsaturated soils, the shear strength is dependent on suction (refer to the Bishop equation, Section 12.1) and hence on the degree of saturation. The higher the degree of saturation, the lower the suction, and hence the lower the cohesive shear strength. If a vane shear is carried out in an unsaturated clay or silt, then that measurement may be unconservative because, were the same test to be done at a higher degree of saturation in the same soil, then the shear strength reading would be lower because of the lower soil suction.

It is common in New Zealand practice to consider that the vane shear strength measured in the field is representative of the soil's shear strength in future rapid loading conditions (such as seismic loading). It is recommended that this is only done if the moisture content at the time of testing is equal to or wetter than the moisture content in the future loading case, and the overburden is expected to be similar. This would be reasonable if the soil is saturated or (perhaps) if the test were carried out during a wet period. It would require explicit measurement of the moisture content at the time of testing and some understanding of the likely moisture content and overburden in the future loading condition being considered.

This effect is true of other in situ tests, too. For instance, in the SPT and the CPT, if carried out in cohesive soils in unsaturated conditions, there is a component of shear strength that is derived from suction. This suction will be less if the test were to be later carried out in wetter conditions, and hence the SPT "N" or CPT "q_c" values may be lower. This effect should be considered if the moisture content in future loading conditions is significantly different from that during the testing.[9]

Those geoprofessionals who use the Scala probe (also known as a Dynamic Cone Penetrometer) regularly will be familiar with this effect. On a silty or clayey subgrade, the blowcounts are higher on a sunny day than during a rainstorm, even though the same soil is being tested.

[9] Other limitations of using SPT to assess shear strength in cohesive soils are noted in Table 5.

As per Figure 44, high-permeability soils such as sands (and, by extension, gravels) are much less likely to develop significant suction, and thus will have much lower values of suction-derived shear strength when unsaturated. Hence, in sands and gravels, the difference in field test shear strength between saturated and unsaturated conditions is typically much less significant than for silts and clays and can be conservatively ignored.

In conclusion, a vane shear strength measurement, SPT "N", or CPT "q_c" in silts and clays represent only that soil's shear strength at the degree of saturation at the time of the test. Shear strengths derived from these tests should only be used in slope stability analysis if the expected degree of saturation of the soil in the loading condition being considered is the same, or less, than at the time of testing.

HOW DO SLOPE/W AND SLIDE2 CONSIDER UNSATURATED SOIL STRENGTH

Both programs can consider suction-induced strength, as follows:

	SLOPE/W	Slide2
Considering negative pore water pressure	The user can choose to assume that there is zero strength increase from suction, or the user can input a ϕ^b value to allow calculation of strength from suction.	Like SLOPE/W, but first a non-zero value must be entered in the "Maximum negative pore pressure" box in Project Settings – Groundwater.
Facility to allow ϕ^b to change with suction or degree of saturation	A volumetric water content function is available to calculate suction derived shear strength, but it doesn't include ϕ^b.	A water content function can be used to determine ϕ^b if a transient analysis is being carried out.

Refer to the user manual (GEO-SLOPE International, 2021) for further information on SLOPE/W and SEEP/W. For Slide2, consult the Rocscience website.

13 LOADING CONDITIONS & FACTORS OF SAFETY

13.1 OVERVIEW

There are multiple approaches to assessing the stability of a slope and to expressing the degree of stability (e.g. LEM and FoS, numerical modelling methods and deformation, observational and risk-based approach). As discussed in Section 1, this document focusses on LEM and the associated measure of stability, FoS. This approach is not always required or appropriate. The limit equilibrium approach is typically appropriate to assess stability of new slopes or when changes are proposed to existing slopes. Where existing slopes are being assessed with no changes to the slope proposed, it is usually more appropriate to use a risk assessment approach.

This section outlines typical loading conditions and an approach to selecting target FoS for those load conditions where limit equilibrium methods are used to assess slope stability.

13.2 DEFINITIONS AND ACRONYMS IN THIS SECTION

Factor of Safety (FoS)[10] - the ratio of stresses resisting soil movement to the stresses driving soil movement.

Level of Engineering (LoE) – categories of the amount and quality of geotechnical investigation, design, analysis, construction monitoring and post-construction monitoring. These range from LoE I (best) to LoE IV (poor) and are defined in Table 7.

Annual Exceedance Probability (AEP) - The estimated probability that an event of specified magnitude will be exceeded in any year (the definition provided by AGS, 2007). This concept has several names. AGS (2007) sometimes just call it Annual Probability. NZS1170.0 and Saunders & Glassey (2007) (sometimes) call it Annual Probability of Exceedance. Justice et al. (2006) call it Annual Probability of Occurrence.

Qualitative Risk Assessments – a process of evaluating the potential risks associated with landslides based on expert judgement and qualitative observations.

Quantitative Risk Assessments – a process of evaluating the potential risks associated with landslides based on more detailed 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.

Tolerable Risk – defined by AGS (2007) as "risks 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." In other words, it is the threshold of risk that is tolerated because the cost to reduce the risk outweighs the benefit of that reduction.

Acceptable Risk - defined by AGS (2007) as "risks which everyone affected is prepared to accept. Action to further reduce the risk is usually not required".

Geometric Mean of AEP - The geometric mean of the annual exceedance probabilities (AEPs) is a statistical measure used to summarize multiple AEP values. It is defined as the n-th root of the product of n individual AEP values. The geometric mean is appropriate for AEPs because it preserves the proportional relationships among probabilities and avoids the dominance of extreme values that can occur when using an arithmetic mean.

13.3 LEM VS RISK ASSESSMENT

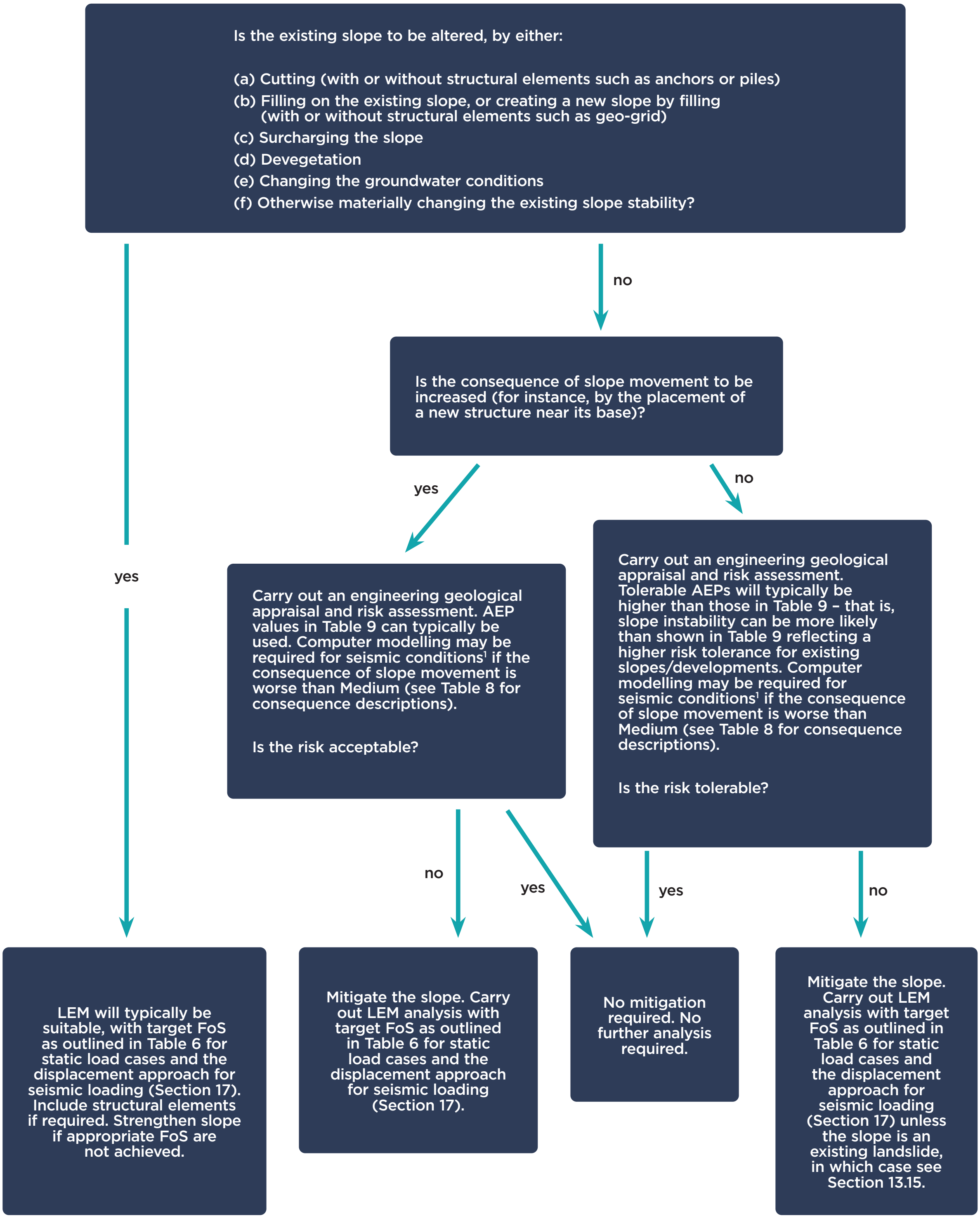
It is not always required or appropriate to assess the stability of a slope using LEM. Figure 46 provides guidance on situations in which a risk assessment is appropriate and situations in which LEM is appropriate.

13.4 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 6. This is not an exhaustive list. Additional or alternative loading cases may require analysis depending on the slope. For instance, dams, which lie outside the scope of this guidance, may require a rapid drawdown analysis.

Commonly, different minimum Factor of Safety (FoS) values are needed for each scenario. This Unit presents FoS minimum values with a clear derivation based on considerations of consequence and geotechnical uncertainty. The derivation of the FoS values is summarised in the sections below, with details in Wightman & Norris (2024).

[10] There are alternative definitions of FoS including the ratio of stabilising moments to destabilising moments and the ratio of soil shear strength to the shear stress required for equilibrium.



[1] Risk assessment of slope instability under static long-term and high ground water conditions for existing slopes can be easier than for seismic conditions because there is more information on the slope's past performance. Hence, computer modelling may be needed to supplement the seismic risk assessment.

Figure 46: LEM versus Risk Assessment

Table 6: Typical Conditions for Analysis

<table>
<tr><th>Condition</th><th>Modelled Soil Shear Strength</th><th>Modelled Pore Pressures</th><th>Reference for Minimum FoS</th></tr>
<tr><td>Long-term Static (long term conditions, typical wet-season ground water conditions, include permanent and temporary surcharges)</td><td>Drained shear strengths related to effective stresses for both free-draining and low permeability soils.</td><td rowspan="2">Based on steady state seepage analysis
or
Calculated from the phreatic surface or piezometric line</td><td>Section 13.10</td></tr>
<tr><td>High Ground Water (Flooding, Saturation of near surface soils, Perched GWT due to high intensity rainfall[1])</td><td>Where instability may be caused by increase of pore pressure within the slope, drained shear strengths related to effective stress apply. Strengths associated with saturation should be applied to near surface soils.</td><td>Section 13.13.1</td></tr>
<tr><td>Undrained loading in low permeability soil
Either the early stages of longer-term loading before excess pore water pressures have dissipated
Or
Short-term (non-seismic) loading such as traffic surcharge</td><td>Low-permeability soils – undrained strengths related to total stresses</td><td rowspan="2">Total stresses with no pore pressure in computations.</td><td rowspan="2">Section 13.10, with notes from Sections 13.13.2 and 13.13.3</td></tr>
<tr><td>Partial Consolidation and Staged Construction</td><td>Low-permeability soils - Consolidation analyses can be used to estimate the increases in effective stress at a particular degree of consolidation. The undrained shear strength at this stage can then be estimated for total stress analysis using relationships between ΔS_u and $\Delta\sigma_v$.[2]</td></tr>
<tr><td rowspan="2">Earthquake Loading (average ground water conditions[3])</td><td>Free-Draining Soils – drained shear strengths related to effective stresses.</td><td>Based on steady state seepage analysis
or
Calculated from the phreatic surface or piezometric line.
Where pore pressure buildup is expected, excess pore pressures related to FoS against liquefaction should be included (Section 17.3)</td><td rowspan="2">Section 17</td></tr>
<tr><td>Low-permeability soils – undrained dynamic shear strengths related to total stresses. Use strain compatible dynamic strength.</td><td>Total stress with no pore pressure in computations</td></tr>
</table>

Table 6: Typical Conditions for Analysis (continued)

Post-Earthquake (residual liquefied soil strengths, average ground water conditions)	Non-liquefied Free-Draining Soils – drained shear strengths related to effective stresses.	Based on steady state seepage analysis or Calculated from the phreatic surface or piezometric line. Where pore pressure buildup is expected, excess pore pressures related to FoS against liquefaction should be included (Section 17.3)	Section 17.4
	Low-permeability soils[4] – undrained dynamic shear strengths related to total stresses. Use strengths that are consistent with the anticipated deformation[5].	Total stress with no pore pressure in computations.	
	Liquefied Soils – residual liquefied undrained shear strengths related to total stresses.		

[1] The definition of the High Ground Water condition is provided in Section 13.13.1.
[2] There is still some debate in the geotechnical community on whether effective stress analysis or total stress analysis is best for partial consolidation stability analysis. Total stress analysis is recommended here as undrained failure is the most likely failure mechanism. For more details a thorough discussion is included in Duncan et al (2014). In addition to the analyses described here for stability of embankments on soft ground, monitoring during embankment construction is critical to verifying stability models. Matsuo plots, which relate measured embankment vertical and horizontal displacements to stability, can be a useful tool in stability control during construction (Matsuo and Kawamura, 1977).
[3] This is consistent with Module 6, which states that the average water table should be used for the earthquake load case (for retaining walls).
[4] If soils are over-consolidated, the drained case may be critical, and a two-stage procedure as described in Duncan et al (2014) is appropriate.
[5] Under seismic loading, some cohesive soils lose strength, which is called "cyclic softening".

13.5 APPROACH TO DERIVING APPROPRIATE FOS

The Factor of Safety (FoS) is the most common quantitative measure of the stability of a slope. An initial target value (under long term conditions) of 1.5 was implied by Terzaghi in 1943 without a detailed discussion of why this value was selected (Schnaid et al., 2020). Over time, typical target values of FoS have become established for other loading conditions (i.e. temporary works, high groundwater etc.) and widely adopted by authorities and geoprofessionals.

It is widely recognised that the minimum FoS should be based not just on the loading conditions, but also the consequence of failure, importance of nearby structures, and/or level of certainty of the input parameters (Duncan, 2000; Schnaid et al., 2020; Adams, 2015; and GEO, 2000). In other words, the selection of FoS should reflect the project-specific risk.

In New Zealand this issue was highlighted by Crawford & Millar (1998, 1999) who drew on results of a questionnaire to Councils and geotechnical consultants. They emphasized the need for consideration of certainty of design assumptions and level of risk when using typical FoS targets. Internationally, recommendations for typical values of FoS are commonly accompanied by commentary that the geoprofessional should evaluate their applicability considering the uncertainties in the model and consequence of failure (e.g. FHWA, 2021; Canadian Geotechnical Society, 2006).

However, our research has not found an author or authority that provides a well-reasoned basis for their selections of minimum FoS.

The approach taken to develop the FoS recommendations for this guidance is outlined below and the following sections provide more detail.

1. Research completed by Silva et al (2008) was used to relate FoS to a slope's probability of failure for various levels of uncertainty in the ground model.
2. Generalised maximum acceptable probabilities of failure were developed for several consequence levels using a range of widely used risk frameworks (see Table 9). This is intended to be broad and reflect common levels of risk tolerance for "routine" low to medium risk projects. It may not be suitable for a specific use and the applicability of the presented risk tolerance to a specific site needs to be considered on a site-by-site basis.

3. For the maximum acceptable probability of failure at each consequence level developed in Step 2, the corresponding FoS from relationships in Step 1 was determined for each level of uncertainty in the ground model.
4. The resulting FoS for each level of uncertainty (termed Level of Engineering in this Unit, see Table 7) and consequence level (see Table 8) provide generalised guidance for geoprofessionals to aid the project team and stakeholders in determining minimum FoS for slope stability assessments.

13.6 FOS VERSUS ANNUAL PROBABILITY OF FAILURE

Silva et al (2008) present a figure which relates the annual probability of failure of a slope to its factor of safety, dependent on the level of engineering. Silva et al (2008) selected engineering projects with well-known design, construction, and operation characteristics from their practice. The data was developed from over 75 projects spanning over 4 decades including zoned and homogenous earth dams, tailings dams, natural and cut slopes, and some earth retaining structures. Stability analyses assessed FoS using the "*best estimate of strength acting in the field and not necessarily the average strength or a conservative value of strength*".

The level of engineering (LoE) is described by categories, with LoE I being "best" and LoE IV being "poor". In Figure 47, these are shown as Category I to IV projects. As expected, for a given FoS, LoE I gives the lowest probability of failure and LoE IV the highest probability of failure, with LoEs II and III in between. Descriptions of the LoE categories are provided in Table 7. These are similar in concept to the Engineering Geological Models described in Part 5 of Unit 1, and comparisons between the two systems are noted in Table 7.

When assessing LoE for a particular project, it is possible that the project may have different LoEs for different aspects of the project. The procedure for this circumstance is described in Section 13.19 and involves weighting numbers (shown in brackets at the bottom of cells in Table 7).

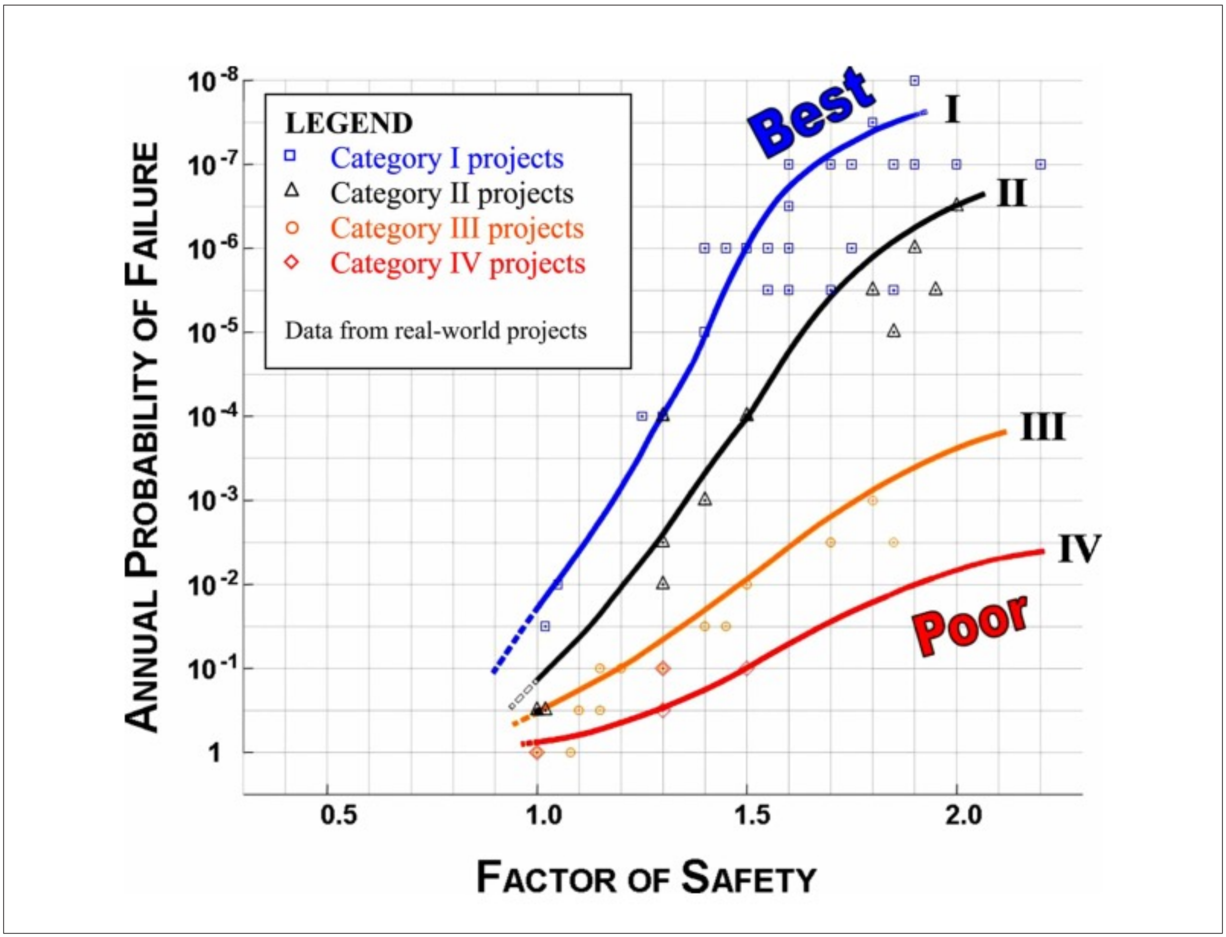


FIGURE 47: Annual Probability of Failure versus Factor of Safety – from Silva et al (2008)

Table 7: Level of Engineering descriptions (after Silva et al, 2008)

Level of Engineering (LoE)	Pre-construction investigation	Testing	Analyses and documentation	Construction observations	Post-construction operation and monitoring[5]
I (Best)	Evaluate design and performance of nearby structures and/or slopes[7] Analyse historic aerial photographs Locate most or all nonuniformities (soft, wet, loose, high or low permeability zones) Determine site geologic history Determine subsoil profile using continuous sampling Obtain undisturbed samples for lab testing of foundation soils Determine field pore pressures Broadly analogous to EGM Level 3 (0.2)	Run lab tests on undisturbed specimens at field conditions[6] Run strength test along field effective and total stress paths Run index tests to detect most or all soft, wet, loose, high or low permeability zones Calibrate equipment and sensors prior to testing program Broadly analogous to EGM Level 3 (0.2)	Determine FoS using effective stress parameters based on measured data (geometry, strength, pore pressure) for site Consider field stress path in stability determination Prepare flow net for instrumented sections Predict pore pressure and other relevant performance parameters (e.g. stress, deformation, flow rates) Have design report clearly document parameters and analyses used for design Peer review (0.2)	Full-time supervision by geoprofessional Construction control tests by geoprofessionals and technicians Construction report(s) clearly documents construction activities (0.2)	Complete performance program including comparison between predicted and measured performance (e.g. pore pressure, strength, deformation) Continuous maintenance by trained crews (0.2)
II (Above average)	Evaluate design and performance of nearby structures and/or slopes[7] Exploration program tailored to project conditions by geoprofessional Broadly analogous to EGM Level 2. (0.4)	Run standard lab tests on undisturbed specimens[6] Measure pore pressure in strength tests Evaluate differences between lab test conditions and field conditions Broadly analogous to EGM Level 2 (0.4)	Determine FoS using effective stress parameters and pore pressures obtained from laboratory data. Adjust for significant differences between field stress paths and stress path implied in analysis that could affect design[1] No matter how sophisticated the analysis, this category cannot be chosen for this aspect if no laboratory testing has been done. (0.4)	Part-time supervision by geoprofessional Construction control tests by geoprofessionals and/or technicians Construction report(s) documents construction activities (0.4)	Periodic inspection by geoprofessional Selected field measurements Routine maintenance (0.4)

Table 7: Level of Engineering descriptions (after Silva et al, 2008) (continued).

Level of Engineering (LoE)	Pre-construction investigation	Testing	Analyses and documentation	Construction observations	Post-construction operation and monitoring[5]
III (Average)	Evaluate performance of nearby structures and/or slopes Estimate subsoil profile from existing data and borings Broadly analogous to EGM Level 1. (0.6)	Index tests[2] on samples from site Broadly analogous to EGM Level 1. (0.6)	Rational analyses[3] using parameters inferred from index tests (0.6)	Informal construction supervision Few or no construction control tests (0.6)	Annual inspection by geoprofessional No field measurements Maintenance limited to emergency repairs (0.6)
IV (Poor)	No field investigation (0.8)	No laboratory tests or index tests (0.8)	Approximate analyses[4] using assumed parameters (0.8)	No construction supervision by geoprofessional No construction control tests (0.8)	Occasional inspection by non-qualified person No field measurements (0.8)

[1] For rain-induced landslides, soil elements follow stress paths characterised by gradual increases in pore-water pressure while total normal stress and shear stress remain constant, with the failure produced by a loss of effective stress, and hence a loss of shear strength. This is different from the stress paths followed in conventional triaxial tests, which involve increasing the normal stress and the shear stress. Farooq et al (2005) provide advice and an equation on how to account for this difference.
[2] Index tests are listed by Silva et al (2008) as field vane and cone penetrometer. It is recommended that Standard Penetration Tests, Plasticity Index, Geological Strength Index, Point Load Tests, and hand estimates of rock strength (for instance, using Table 3.5 of NZGS, 2005) are also considered as Index tests. Dynamic Cone Penetrometer (Scala) tests should not be considered an Index test in this context.
[3] In this table "Rational Analyses" means LEM, Procedures of Slices analysis. The salient point is that this is considered LoE III if the soil strength parameters are inferred from index tests only.
[4] It is inferred that "Approximate analyses" could include Infinite Slope and Stability Charts methods described in Table 12.
[5] If works are temporary, a value of 0.2 can be used for Post Construction Operating & Monitoring in most circumstances - refer Section 13.13.5.
[6] Undisturbed sampling in sand, gravel and sensitive soils can be challenging due to the non-cohesive nature of these materials and/or the potential for sample disturbance during the extraction process. Where undisturbed sampling is impracticable, an LoE I or II may still be achieved in the relevant columns of Table 7 if the undisturbed sampling is substituted by an in situ test that would provide similar quality information to a laboratory test on undisturbed material.
[7] If a back-analysis is carried out to determine the minimum plausible soil strengths on the subject slope then this could form part of an LoE I or II classification under the "Pre-construction investigation" aspect (see Section 18 for more on back-analysis)

13.7 REVIEW OF PUBLISHED RISK THRESHOLDS

New Zealand does not have a regulatory framework setting out acceptable or tolerable risk. Published guidance on risk was used to define ranges of AEPs for a variety of consequence categories, and in different contexts (risk to life, damage to structures and land, damage to roads). The goal was to develop threshold annual probabilities of failure that reflect generally accepted levels of slope stability risk in New Zealand. The references used were:

- AGS (2007), which recommended Annualised Individual Fatality Risks (AIFR) limits of one in 100,000 (1×10^{-5}) for new slopes and developments.
- Draft TS1170.5 commentary, which provides comments on AIFR for buildings.
- Qualitative risk assessments by AGS (2007), Justice et al (2006), Saunders & Glassey (2007) and Saunders et al (2013).
- NZS1170.0 - Although it doesn't consider landslides, it is reasonable to expect that slopes near buildings are designed such that the probability of building collapse due to a landslide is similar to the probability of building collapse due to any other hazard. NZS1170.0 provides implied acceptable risks at ULS and SLS levels.
- MoE (2020) – provides implied acceptable performance at SLS2 for schools.

13.8 CONSEQUENCE CATEGORIES

Each risk assessment system in Section 13.7 uses slightly different descriptions of consequences, and hence the authors of this Unit created a new set of consequence descriptions that was similar to each of them. A new level was added to allow for the possibility of more than 10 fatalities, and descriptions were added on the expected numbers of fatalities. Recommended consequence levels and their associated descriptions are presented in Table 8.

Table 8: Consequence Levels

Consequence Level	Life risk	Expected damage to buildings and property[2]	Expected damage to roads
Catastrophic	Probably 10 - 20 fatalities[1]	Collapse of 20 - 100 single-residence buildings, or collapse of a large multi-storey building or school building.	Undermining or inundation of a major state highway such that it is impassable for months in both directions.
Disastrous	Probably 1 - 10 fatalities	Collapse of up to 20 single-residence buildings or one small multi-storey building.	State highway or arterial blocked for days or weeks in both directions; significant effects to communities for extended periods.
Major	Probably no fatalities	Extensive damage to single-residence building(s) (or its only accessway) requiring major engineering works for stabilisation.	Both lanes of local road impassable for days or weeks; or Arterial route blocked for up to a day in both directions.
Medium	No fatalities	Extensive damage to land but dwelling(s) has moderate or no damage.	Both lanes of local road temporarily blocked/slipped (delay of few hours to a day); or One lane of arterial route blocked with major delays.
Low	No fatalities	Moderate damage to land, no damage to structure.	One lane of local road blocked / slipped; or Several metres of footpath destroyed - no alternative access available.
Minor	No fatalities	Little or no damage to land; no damage to structure.	Half of one lane of road or shoulder blocked for short period of time; emergency works limited to clean up only; Or Footpath destroyed over several metres - alternative access is available.

[1] If the consequence is likely to exceed 20 fatalities a site-specific study is recommended, with a similar methodology to that outlined here, but with specific consideration of the number of buildings at risk, the likely number of occupants per building, the likely proportion of the time those occupants will be present, and the likelihood of fatality if the structures were to be affected by the landslide.
[2] This refers also to neighbouring properties, not just the property on which the works will take place.

Assessing the consequence of a failure can be a difficult exercise. It involves estimating the likely velocity of the failure and runout distance as well as considering the elements (structures, infrastructure, people) that are within that runout distance or could be undermined.

The likelihood of fatalities is dependent on the location of the landslide relative to structures and people. A landslide (including rock fall and debris flows) that descends on structures and people from above is much more likely to result in fatalities than a similar sized landslide that occurs below structures and people. Catastrophic and Disastrous consequences will thus arise mostly from landslides impacting buildings from above, and those with the highest velocities the most likely to cause fatalities. Landslides that undermine structures are, typically, less likely to result in fatalities, except for a cliff collapse undermining a building, where fatalities are more likely. For guidance on assessing consequences of debris flows, refer Unit 1 Section 9, and a future Unit 6 devoted to debris flows. For guidance on assessing consequence of rockfall, see Unit 1 Section 8 and MBIE (2016).

The consequence should be assessed based on considerations of every person, every property and every building that could reasonably be expected to be affected by slope movement – not just on the people, property and buildings associated with the project's client. This means that if it is known that 100 houses are proposed to be developed and that the landslide could affect all of them, then that must be considered by the geoprofessional in assessing consequence.

Table 8 considers only consequences to people, buildings, land, and roads, as those were the subjects of the prior risk assessment methods reviewed. In some projects, other assets could be at risk, such as ports, railways, or animals. In cases where the asset has similarities to those in Table 8 (for instance a railway has similarities to a road) then the recommendations in this guidance could reasonably be used – noting however the intended limitations in Section 1.3. Where there is insufficient similarity with the assets listed in Table 8, then the geoprofessionals may need to carry out their own literature review and/or consult with the asset owner to assess acceptable AEPs (see next section).

13.9 ACCEPTABLE AEP

The recommended values of acceptable AEP were assessed by considering the acceptable AEP values implied, or expressly indicated, by the authors listed in Section 13.7. Where there was a range of AEP for any consequence level, the average was taken, after discarding any clear outliers. The recommended AEP values are presented below with implied return period for easy comparison.

13.10 MINIMUM FACTOR OF SAFETY VALUES – LONG-TERM STATIC CONDITIONS

If the LoE is known, and the acceptable AEP is known, then the associated minimum factor of safety can be determined using the work of Silva et al (2008). This Unit therefore presents minimum factors of safety for a range of levels of engineering and consequence. Recommended minimum factors of safety for new slopes under Long-term Static conditions are presented in Table 10. These are based on acceptable (rather than tolerable) probabilities as this is the appropriate level for new structures. Details on the derivations of these values are presented in Wightman and Norris (2024).

Table 9: Acceptable AEP values.

Consequence Category	Range of Reviewed AEPs	Average[1] of reviewed AEPs	Implied acceptable return period range (average[1] in brackets)
Catastrophic	10^{-7} to 10^{-6}	3×10^{-7}	1 to 10 million years (3.3 million years)
Disastrous	10^{-6} to 5×10^{-4}	6×10^{-5}	2,000 to 1 million years (17,000 years)
Major	5×10^{-5} to 2×10^{-3}	5×10^{-4}	500 to 20,000 years (2,000 years)
Medium	5×10^{-4} to 0.02	4×10^{-3}	50 to 2,000 years (250 years)
Low	0.01 to 0.05	0.03	20 to 100 years (30 years)
Minor	0.1 to 1	0.2	1 to 10 years (5 years)

[1] More precisely, this is the geometric mean (see Section 13.2 for definition).

Table 10: Minimum FoS – Long-term Static loading conditions

Consequence Category	Minimum FOS range (geometric mean in brackets) for each Level of Engineering Category			
	Category I Best	Category II Above Average	Category III Average	Category IV Poor
Catastrophic	1.5 to 1.7 (1.6)	1.8 to 2.2 (2.0)	N/A*	N/A*
Disastrous	1.2 to 1.5 (1.4)	1.4 to 1.8 (1.6)	N/A*	N/A*
Major	1.2 to 1.3 (1.2)	1.3 to 1.6 (1.4)	1.7 to 2.1 (1.9)	N/A*
Medium	1.2	1.2 to 1.4 (1.3)	1.4 to 1.9 (1.6)	1.8 to 2.2 (2.2)
Low	1.0 to 1.1 (1.1)	1.1 to 1.2 (1.2)	1.3 to 1.5 (1.4)	1.6 to 1.9 (1.7)
Minor	1.0	1.0	1.0 to 1.2 (1.1)	1.0 to 1.5 (1.4)

*Not appropriate – must increase the Level of Engineering (LoE) to improve LoE Category

The values in brackets (where present) should be used in most cases. If there is some significant aspect of risk in the project not adequately represented by the LoE and Consequence selections, another value within the range may be used.

Legend

- (Yellow) Additional Investigation Required to Improve LoE Category.
- (Orange) FoS values higher than "typical" target values are required, or additional investigation required to improve LoE.
- (Blue) Typical FoS values (~1.5) for Long-term Static conditions can generally be adopted.
- (Green) A FoS lower than "typical" values is suitable. Appropriate justification is required, and consequences should be communicated to stakeholders.

The FoS values presented in Table 10 aim to reflect general levels of slope stability risk acceptance for new slopes within New Zealand. An interesting and useful conclusion that can be drawn from these values is that where the level of investigation, design and oversight for slope stability assessment reflects the magnitude of the consequence (i.e. higher level of engineering where the consequence is higher, and lower where the consequences are low), the "typical" values of FoS that have been commonly used for decades generally achieve a broadly acceptable level of risk. While this conclusion is not unexpected, it provides confidence in the use of these values provided the appropriate level of investigation and oversight is carried out.

In general, we consider that if the recommendations in IAEG Commission 25 (Baynes & Parry, 2022) on the level of ground model development relative to the project and geology complexity are followed (Figure 5.5 and 5.6 in Unit 1 of the Slope Stability Guidance), "typical" values of FoS are appropriate to achieve broadly acceptable levels of slope stability risk in New Zealand.

A thick black line has been added to Table 10 to distinguish between Medium to Catastrophic consequences (in which geoprofessionals are most often consulted) and Low to Minor consequences (in which geoprofessionals are infrequently consulted).

13.11 PROCESS FOR SELECTING THE APPROPRIATE FOS AND LOE FOR NEW SLOPES

This process should begin with considering the consequences of failure and selecting an appropriate Level of Engineering investigation as described above. The approach should encourage the geoprofessional to consider the elements of risk and communicate the risk to the project stakeholders.

1. **Estimate the consequence (Section 13.8).** The geoprofessional should estimate the consequence that could occur if the slope failed and then choose the corresponding consequence category (Table 8). The failure with the highest consequence might not have the greatest risk. It may be that the failure with the highest likelihood has the most risk. Geoprofessionals might need to try several surfaces to see which has the most risk.
2. **Determine LoE (Section 13.6).** By targeting a Level of Engineering to be within the blue region of Table 10 and Table 11, typical FoS values can be used. Alternatively, if investigations have already been carried out and further investigations are not proposed, the LoE should be assessed using Table 7, and the geoprofessional should determine which region in Table 10 and Table 11 applies and proceed as indicated. The light orange region indicates that the consequence is too large for the level of investigation and that additional certainty in the ground model and

slope stability results is required. This could be achieved through additional investigations or design work. Within the dark orange coloured region, a higher than typical FoS should be targeted and recommended values are provided in Table 10 and Table 11. Within the blue region, typical values can be adopted, and within the green region, it may be acceptable to adopt a lower than typical minimum FoS.

Many of the cells in Table 10 and Table 11 provide a range of possible minimum FoS values, reflecting the range of acceptable AEP values for any given consequence. The value in brackets should be used, unless there is some aspect of risk in the project not adequately represented by the LoE and Consequence selections, in which case the geoprofessionals may select another value within the range provided.

The geoprofessional shall provide, in their design report, an appropriately detailed evaluation of the consequence selected, the LoE selected, and hence the selected minimum FoS. Where the calculated FoS is lower than typical values, it may be that territorial authorities (and others) may be wary, and hence the justifications in the design report should be especially clear and prominent for this case. A lower than typical FoS may mean a higher-than-usual likelihood of slope movement (offset by a relatively low consequence) and the site owner should be advised of this in writing.

Examples of calculations of FoS are presented in Section 13.19.

13.12 OTHER CONSIDERATIONS

13.12.1 Soil Shear Strengths

To be consistent with Silva et al (2008), from whose work the values in Table 10 are derived, it is recommended that soil and rock "*strength determination corresponds to the best estimate of the strength acting in the field and not necessarily the average strength or a "conservative" value of strength*".

What is the "best" estimate of soil strength? If the investigation has provided many measurements of soil strength, then the best estimate may be the mean value, if the anticipated failure surface is long, and hence a large amount of soil will be mobilised. But if the anticipated failure surface is short, the best estimate may be the lower quartile or possibly even the lowest value. If the investigation has yielded few measurements of soil strength, then usually the best value would be the lowest value or one near the low end of the range measured.

To consider the possibility that field values of strength might be significantly lower than those assumed in analysis, sensitivity studies should be carried out, as described in Section 19.

13.12.2 Multiple failure surfaces

It may be that different plausible failure surfaces on the same slope have different consequences. For instance, a shallow failure might have a Medium consequence but a deep-seated failure would have a Major consequence. In this instance, it is appropriate for the two failure surfaces to be assigned different minimum FoS values.

13.12.3 Project-specific risk thresholds

In unusual circumstances, some clients and/or authorities may have risk appetites different from those implied by Table 9 and where this is so, the geoprofessional should work with the project team and stakeholders to define acceptable AEPs. The Silva et al (2008) chart (Figure 47) can be used to determine project specific FoS values for the project specific AEPs.

13.12.4 Lower than typical FoS

Some of the values in Table 10 are less than the previously common FoS value of 1.5 for Long-term Static conditions. This includes all the values in the green boxes and some of the values in the blue boxes. It is expected that only in unusual circumstances would geoprofessionals assess that a FoS value from a green box is appropriate. This is because commonly geoprofessionals instinctively match LoE to consequence and hence the blue box is appropriate. Examples of green box circumstances are provided in Section 13.20.

Where the FoS is lower than 1.5 for Long-term Static conditions, it may be that territorial authorities (and others) are wary, and hence the justifications in the design report should be especially clear and prominent for this case. Should territorial authorities still be concerned by low FoS, they may request a peer review.

Where low minimum FoS values are selected, the geoprofessional shall advise their client (and any other potentially affected party, if possible) in writing that slope movements may occur in future and advise them of the possible consequences of these slope movements. If, for example, a FoS of 1.2 appears justified under Long-term Static conditions, then a concept design could be carried out for both an FoS of 1.2 and an FoS of 1.5, with the physical works costs and slope movement frequency and consequence of each concept design assessed and the best solution agreed upon.

13.13 OTHER LOADING CONDITIONS AND SCENARIOS

Table 10 provides FoS recommendations for Long-term Static conditions for new slopes and developments without previous instability. As outlined in Table 6, in a typical project, assessment of other load cases will likely be required, or the geoprofessional may be interested in an existing slope or an existing landslide. FoS recommendations for these conditions are provided below.

13.13.1 Ground Water Conditions

Long-term static analyses, using the FoS values in Table 10, should consider the typical wet-season ground water conditions – that is, the ground water conditions that would typically occur each year in the wettest season of the year (usually in winter in New Zealand). It would be expected that this seasonal ground water condition would last for 1 to 3 months – therefore, up to about a quarter of the time. This is not the worst ground water condition expected in any given year, but rather the ground water condition expected each year for much or most of the wet season.

Analyses should also consider adverse ground water conditions that would not be encountered every year. It is recommended that a High Ground Water analysis consider the ground water conditions that are expected to occur during a 5-year to 10-year storm.

In this guidance the “High Ground Water” condition is defined as:
The ground water pressure distribution in the slope that is likely to cause the factor of safety to reduce to a value that is only reached once every five to ten years.

The assessment of ground water conditions should include (see also Section 11):

(a) The likely location of the ground water table, including any perched water tables, both in the Long-term Static case and the High Ground Water case.

(b) Consideration of the potential for saturation of the near-surface soils during the High Ground Water case, and the consequent loss of cohesion due to a loss of suction.

(c) Considerations of climate change - for instance, is the ground water table expected to be higher in future storms, or are more intense storms expected that may saturate the surface soils? Is the slope close enough to the coast for sea-level rise to be significant?

Subsurface drainage is a common method used by geoprofessionals to improve the FoS of a slope, particularly under storm conditions. However, subsoil drains can over time become less effective, or in extreme cases become blocked. The potential for blockage depends on several factors, some of which can be controlled during the design stage (e.g. use of filters, good specification, type of drain suited to the ground conditions and permeability) and other post-construction factors where there is little control from the designer (e.g. algae growth, reliance on maintenance, protection from future development) (Tonkin & Taylor Ltd, 2018). It is therefore recommended that drains are modelled as ineffective when considering High Ground Water conditions, unless a detailed maintenance plan for the drains is prepared and there is high confidence that it will be followed.

Recommended FoS for the High Ground Water case is shown in Table 11.

Table 11: Minimum FoS – High Ground Water conditions

Consequence Category (this study)	Minimum FOS range (geomean in brackets) for each Level of Engineering Category			
	Category I Best	Category II Above Average	Category III Average	Category IV Poor
Catastrophic	1.3 to 1.4 (1.4)	1.6 to 1.7 (1.6)	N/A*	N/A*
Disastrous	1.2 to 1.3 (1.2)	1.2 to 1.6 (1.3)	1.4 to 1.8 (1.6)	N/A*
Major	1.2	1.2 to 1.3 (1.2)	1.2 to 1.7 (1.4)	1.5 to 1.9 (1.7)
Medium	1.2	1.2	1.2 to 1.4 (1.2)	1.2 to 1.7 (1.4)
Low	1.0	1.0	1.0	1.0 to 1.2 (1.0)
Minor	Not required**	Not required**	Not required**	Not required**

*Not appropriate – must increase the Level of Engineering (LoE) to improve LoE Category.
**This case may not be required to be analysed where the consequence is minor. It is implied that failure every 5 - 10 years is acceptable (to the owner and regulatory authority) if the consequence is minor.

The values in brackets (where present) should be used, unless there is some aspect of risk in the project not adequately represented by the LoE and Consequence selections, in which case another value within the range may be used.

Legend

Additional Investigation Required to Improve LoE Category.

FoS values higher than “typical” target values are required, or additional investigation required to improve LoE.

Typical FoS values (~1.2 or 1.3) for high groundwater conditions can generally be adopted.

A FoS lower than “typical” values is suitable. Appropriate justification is required, and consequences should be communicated to stakeholders.

13.13.2 Undrained Loading of Low Permeability Soils

When low permeability soils are present, analyses should be carried out to assess the FoS of slopes under short-term loading, using undrained strengths for low permeability soils. Short term loading assessments should be carried out to consider:

- The early stages of longer-term loads before excess pore water pressures have dissipated (and this may include the early stages of temporary works or staged construction)
- Short-term (non-seismic) loading such as occasional traffic surcharge
- Partial consolidation of low permeability soils

When assessing LoE to determine the FoS, practitioners may select a 0.2 rating when considering the "post-construction operation and monitoring" aspect, because the short duration of the loading means there will be no "post-construction" phase.

13.13.3 Traffic surcharging

Traffic surcharges should be included in a Long-term Static analysis, using drained parameters because most traffic surcharges are repetitive and frequent. If there are cohesive soils below the traffic loading, then an undrained analysis should be also carried out (see also Table 6).

Guidance on application of surcharge loads can be found in Section 16.8 with the appropriate load factors given in Table 13.

13.13.4 Factors of Safety under seismic conditions

A detailed discussion of the FoS approach under seismic conditions is provided in Section 17. In general, the approach is to assess the deformation likely to occur in an earthquake and the consequences of slope movement considering how much damage, if any, this would imply in the structures. It is acknowledged that there is no explicit consideration of LoE in this approach, but there is no mention of seismic performance in Silva et al and thus we consider using Silva's probability of failure vs. FoS relationships for seismic analysis would not be valid.

13.13.5 Temporary works

The stability of temporary slopes should be analysed when they pose a significant risk to either construction workers, the structure being built, or nearby structures, people, or property. Any of the conditions listed in Table 6 may apply to temporary works, and the geoprofessional shall analyse all applicable loading conditions.

In practice, it has been common to use lower factors of safety for temporary works – for instance, in Table 5 of CIRIA Report 104 (Padfield & Mair, 1984) or Table 5 of Adams (2015). However, this Unit does not recommend an explicit reduction in target FoS for temporary works. This is because it is not clear that duration of project should be a factor in choosing FoS [11]. If slope failure has, for example, a Medium consequence in both the permanent and temporary case, then the acceptable AEP is about 4 x 10^{-3} in both cases, no matter if the duration of the works is 10 months or 100 years, and hence the FoS should be the same, everything else being equal. The previous rationale for reducing the target FoS for temporary works may have been at least partly because the consequence of failure may be lower during construction, or at least the first part of the construction, because there is less of the valuable building to damage.

When assessing temporary works projects, practitioners should select the consequence as per Table 8 based on what the effects will be of a slope failure during construction, noting that it is possible that the consequences during construction may be higher than in the permanent case, especially if construction workers are directly exposed to slope movement without the sheltering benefit of the permanent structure. When assessing LoE, practitioners may select a 0.2 rating when considering the "post-construction operation and monitoring" aspect, so long as there is no viable possibility that the temporary works will become "permanent", and this will go some way towards reducing the target FoS.

13.14 ASSESSING EXISTING SLOPES

If new structures are to be placed near existing slopes, such that there is a significant change to the consequence of failure (but no change to the likelihood of failure) then a sound engineering geological appraisal of existing and past slope performance (including investigations as required) should be carried out, followed by a risk assessment (see also the flow chart in Figure 46).

If there are no new structures to be placed near, but a risk assessment shows that the existing slope poses an intolerable risk to existing structures, then mitigation will be required. The minimum design values of FoS for the mitigated slope may be set as per Table 10 and Table 11.

11 Published guidance on risk (examples in Section 13.7) assess acceptable risk based on likelihood per year, not likelihood per design lifetime. Therefore, acceptable risk, and hence FoS, should be independent of design life.

Computer-based slope stability analyses under Long-term Static and High Ground Water conditions should be carried out on existing slopes if a substantive change to the slope is proposed, such as:

- The slope is to be altered in a way that increases the probability of failure (such as cutting the slope's toe, or de-vegetation).
- A substantial new load (for instance, from a new structure or railway line) is being added above an existing slope. However, if the new load is small relative to the size of the slope, such as a new driveway or single storey timber house above a high slope, then a new analysis may be of little value.

A seismic analysis should still be carried for existing slopes unless the site has been subjected to enough previous earthquakes to have satisfactorily demonstrated stability at or above design seismic accelerations.

13.15 ASSESSING EXISTING LANDSLIDES

If the existing slope is a landslide, then the same process should be followed as for existing slopes that aren't landslides, as described in the previous section and in the flowchart on Figure 46. With existing landslides, the following useful information is available that would not be for non-landslide slopes:

- It is known that the existing FoS lies near 1.0 or has approached 1.0 in the past, and hence back analysis can be used to assess the soil parameters.
- There may be records and/or aerial photographs that show previous periods of landslide movement, and hence it can be easier to assess future likelihood and consequence of slope movement.

If no new structures are proposed, but an engineering geological appraisal and risk assessment indicates that the risk posed by the landslide is intolerable, then mitigation is required.

The FoS approach set out in Table 6 (the same as for non-landslides) should be followed for landslide mitigation if reasonably practicable.

Caltrans (2020), however, observe that landslides may be complex features with large dimensions that often extend well beyond the road boundaries. Geographic features such as mountains, rivers, and oceans may limit or preclude investigation and available mitigation strategies. Landslides may occur along remote highways that act as lesser or greater transportation links. Numerous stakeholders with competing interests and viewpoints may be involved in all aspects of remedial activities. Funding for landslide mitigation may be limited.

In these circumstances, where the approach in Table 6 is not reasonably practicable, the goal of landslide stabilisation should be to attain the highest achievable factor of safety while working to satisfy stakeholders and working within geographic and budgetary constraints imposed (as per Caltrans). The consequences of future landslide movement must be considered when setting the FoS target. When the consequences are relatively low, and where correspondingly low values of FoS are adopted, the asset owner and other stakeholders must agree to and understand that future landslide movement remains possible despite the mitigation works (for example, a formal NZTA Departure from normally accepted standards could be obtained). This approach is termed "marginal stabilisation" by Cornforth (2005), who cautions that "this approach is not an option where there is a risk of a rapid or catastrophic failure because it would pose a high threat to life".

Any landslide stabilisation strategy should strive for a minimum 10% increase in stability under Long-term Static conditions (as per Caltrans, 2020).

A seismic evaluation, as per Section 17, should be conducted for all proposed landslide stabilisations, to provide estimates of likely slope movements in future earthquakes.

If stabilisation of the landslide is not practicable, there are alternative, non-engineered, methods of mitigation, such as avoidance and monitoring. These are described in Section 10.5 of Unit 1.

13.16 COMPARISON OF VALUES WITH THE WORK OF OTHERS

The recommended FoS values for new slopes under Long-term Static conditions (Table 10) was compared to similar work presented by Adams (2015) and Schnaid et al (2020). The recommended values in a 5–10-year storm (Table 11) were compared to those presented in the Hong Kong Highway Slope Manual (GEO, 2000).

Both Adams (2015) and Schnaid (2020) presented target FoS values that are typically lower (and sometimes much lower) than those recommended in this Unit, which may be because operations including open cast mining and tailings dams are more accepting of risk than other common civil works such as housing and infrastructure.

The recommended FoS values in GEO (2000) have reasonable agreement with those in this Unit. Overall, we consider that these comparisons provide encouragement that the values recommended in this Unit are reasonable.

Details on these comparisons are provided in Wightman and Norris (2024). .

13.17 FACTORS OF SAFETY FOR RETAINING WALLS AND REINFORCED SLOPES

When undertaking slope stability analysis of deep-seated surfaces when designing retaining walls, practitioners should use the appropriate target factors of safety indicated in existing design guidance (examples include FHWA, 2009 and FHWA, 2015) where the slope analysis includes structural elements (ground anchors, synthetic reinforcement, piles, etc). This is because the strength parameters of the structural elements used in the slope stability program should have due regard to the recommendations of the retaining wall guidance documents, and hence the target FoS should also be in accordance with that guidance. However, when considering deep-seated failures that either don't include the structural elements or only include those structural elements to a minor degree (for instance, the critical failure surface intercepts only the last metre of a 10-metre-long ground anchor) then the FoS values provided in this Unit are appropriate and should be used.

More information on the use of structural elements in slope stability modelling is presented in Section 16.

13.18 PROBABILITY OF FAILURE APPROACH

On high risk or high-cost projects, where the investigation and laboratory program has been extensive enough to characterise the distribution of soil strength parameters and pore water pressures, a probabilistic analysis approach may be used during slope stability modelling, rather than Factor of Safety.

Acceptable AEP values listed in Table 9 can be used for probabilistic analyses, with the consequence assessed as per Table 8. In a probabilistic analysis, there would be no explicit assessment of LoE; instead, the uncertainties would be reflected in the choices of soil strength and pore water distribution, with greater uncertainty meaning that wider distributions should be used.

For more information on probabilistic analyses, refer to Section 19.

13.19 WORKED EXAMPLES – FOS CALCULATION

In the following examples, the consequence of slope movement and the level of engineering are assessed, enabling the determination of the minimum FoS. These examples are closely based on real projects carried out in recent years.

When assessing LoE for a particular project, it is possible that the project may have different LoEs for different aspects of the project. Silva et al (2008) describe the approach to be taken as follows:

For example, if a particular structure meets most of the Category I criteria but only benefited from part time supervision by a qualified engineer during construction (a Category II attribute), we use the weighting number in brackets (in Table 7) to compute the interpolated value as shown below:

Investigation	*0.2*
Testing	*0.2*
Analysis & Documentation	*0.2*
Construction	*0.4*
Operation	*0.2*
Interpolated Category	*1.2*

Silva et al (2008) say that in this case they would use a linearly interpolated curve location 20% of the distance between curves for I and II in Figure 47. But in practice, it is recommended that practitioners use a FoS value that is 20% of the distance between the appropriate FoS for LoEs I and II in Table 10 and Table 11.

For temporary works and short-term loading, the post-construction operation and maintenance (just "Operation" in the Silva et al, 2008 example above) value should be 0.2.

13.19.1 Example 1 – cut above access road

Matthew is designing a cut above a two-lane access road for a new retirement village. It is not the only access road into the village. The cut is 18 m high and 100 m long, and Matthew intends to specify a 22° slope to avoid crossing the boundary at the top of the cut. On the other side of the boundary is a nature reserve with no structures. The ground conditions are 1.5 m of loose natural sand overlying medium dense sand. What factor of safety should Matthew use for Long-term Static analysis?

Consequence – Low. The most plausible type of ground movement, based on an assessment of nearby slopes, would be shallow and could result in one lane of the access road being blocked for a short time. There are other access roads, so this access road is not critical.

Aspect	LoE	Value	Comment
Pre-construction investigation	Between I and II	0.3	A large site with multiple test pits, CPTs, boreholes, and shear box tests throughout the site, which indicated homogenous sand conditions. At the specific area of the cut there was one borehole and two CPTs.
Testing	III	0.6	Only index testing at the site, although there have been shear box tests done at the wider site.
Analysis	Between II and III	0.5	Standard computer slope stability modelling using effective strength parameters inferred from index tests and shear box testing
Construction	II	0.4	Part-time monitoring by geoprofessional
Operation & Maintenance	IV	0.8	No programme specified
Sum		2.6	

For low consequence, the minimum FoS under Long-term Static conditions for LoE II is 1.2 and for LoE III is 1.4. As the assessed LoE is 2.6, a simple linear interpolation can be used to calculate the minimum FoS = 1.2 + (0.6) x (1.4 – 1.2) = 1.3.

13.19.2 Example 2 – Fill below house

In another part of the same retirement village, Aria is designing a fill slope on which houses will sit. The fill is 10 m high, and the houses will lie 3 m from crest of the fill slope. The proposed house foundations are concrete rib-rafts. There is a footpath at the base of the fill. The proposed fill is site-won well-compacted sand (engineered fill) with no geogrid. The proposed fill slope is 18°.

Consequence – Medium. Slope instability extending under the houses is plausible, but because of the rib-raft foundations the damage to the houses is expected to be moderate.

Aspect	LoE	Value	Comment
Pre-construction investigation	Between II and III	0.5	A large site with multiple test pits, CPTs, boreholes, and shear box tests throughout the site, which indicated homogenous sand conditions. Within the subgrade for the proposed fill, there was one CPT and one test pit.
Testing	III	0.6	Only index testing at the proposed fill, although there have been shear box tests done at the wider site.
Analysis	Between II and III	0.5	Standard computer slope stability modelling using effective stress strength parameters inferred from index tests and shear box testing
Construction	Between I and II	0.3	Part-time monitoring by geoprofessional and a high quantity of compaction testing.
Operation & Maintenance	IV	0.8	No programme specified
Sum		2.7	

For medium consequence, the minimum FoS under Long-term Static conditions for LoE II is 1.3 and for LoE III is 1.7. As the assessed LoE is 2.7, the minimum FoS = 1.3 + (0.7) x (1.7 – 1.3) = 1.6.

13.19.3 Example 3 – temporary cut near existing retaining wall

Georgia is designing a temporary cut that is proposed to create a new basement under an existing house. The cut is about 5 m high and an existing concrete retaining wall, 3 m high, sits above the cut and 1.5 m from the cut's crest. A neighbouring house sits above and 5 m away from the top of the retaining wall. The cut is into weathered greywacke rock. The existing house will be sitting on temporary props while the cut is open because the excavation will be under the house and hence will remove the house's foundations. Little is known about the existing retaining wall except that it appears to be in good condition and is not obviously leaning forward.

In the permanent case, the new basement will be formed with a masonry block retaining wall and will be backfilled, so the cut is considered temporary.
What factor of safety should Georgia consider under Long-term Static conditions when deciding if temporary propping is required?

Consequence – Major. Failure of the cut slope could cause the existing concrete retaining wall to be undermined and/or strike the temporary props holding up the house and cause substantial damage to the house. A fatality of a construction worker is considered possible but not likely because the cut is most likely to fail during wet weather and works would not be permitted under the house in wet weather if no retention is provided. The house on the neighbouring property is too far away to be damaged.

Aspect	LoE	Value	Comment
Pre-construction investigation	Between I and II	0.3	Three boreholes which proved rock. Site logging of the rock cuts including index tests (GSI and hand categorisation of rock strength)
Testing	III	0.6	Only index testing.
Analysis	III	0.6	Standard computer slope stability modelling using effective stress strength parameters and Hoek-Brown parameters inferred from index tests.
Construction	II	0.4	Part-time monitoring by geoprofessional.
Operation & Maintenance	I	0.2	Temporary works.
Sum		2.1	

For a major consequence, the minimum FoS under Long-term Static conditions for LoE II is 1.4 and for LoE III is 1.8. As the assessed LoE is 2.1, the minimum FoS = 1.4 + (0.1) x (1.8 – 1.4) = 1.44, rounded up to 1.5.

13.19.4 Example 4 – fill under house

Hassan is designing a new fill slope that will sit close to a proposed house. The fill slope is about 10 m high, is proposed to lie at 34° and will comprise engineered fill. The house lies 3 m from the crest of the fill slope but the house is founded on rock. The fill slope is thus being added just to create more flat land for the use of the house residents. There is only bush at the base of the proposed fill.

What FoS should Hassan use for the design of the fill slope?

Consequence – Low or Medium. Were the fill slope to fail, the consequence would be moderate or extensive damage to the land, but as the house is founded on rock there would be no damage to the house.

Aspect	LoE	Value	Comment
Pre-construction investigation	Between II and III	0.5	Investigation comprised observations of soil exposures, and hand augers in the surficial soils, which will be removed to found the fill on rock. The depth to rock is well understood, but the rock strength is assumed.
Testing	IV	0.8	Index testing has been carried out, but in soils that will be removed. Rock has been observed on site, but no index testing has been carried out.
Analysis	Between III and IV	0.7	Standard computer slope stability modelling using effective stress strength parameters using assumed parameters.
Construction	Between I and II	0.3	Part-time monitoring by geoprofessional and high quantity of compaction testing.
Operation & Maintenance	IV	0.8	No programme specified
Sum		3.1	

The consequence is assessed as low to medium.

For medium consequence, the minimum FoS under Long-term Static conditions for LoE III is 1.7 with no value being available for LoE IV as it is deemed inappropriate for LoE IV to be used with Medium consequences.

For low consequence, the minimum FoS under Long-term Static conditions for LoE III is 1.4 and for LoE IV is 1.7. As the assessed LoE is 3.1, the minimum FoS for low consequence = 1.4 + (0.1) x (1.7 – 1.4) = 1.43, rounded up to 1.5.

As the consequence is marginal between low and medium, a minimum FoS of 1.6 is appropriate.
A separate calculation would need to be made to assess a minimum FoS for deeper-seated failure in the rock that could undermine the house. It is appropriate to have different target FoS values for different events within the same slope.

13.19.5 Example 5 – cut beneath a shotcrete wall

Geraldine is designing a 3m high shotcrete wall that will sit behind a proposed house. Beneath the shotcrete wall will sit a 3m high concrete block wall, to form the rear wall of the house. After discussion with the project team and contractor, it has been decided that the best approach is to make a cut to form the shotcrete wall, construct the shotcrete wall, and then cut beneath the shotcrete wall to allow construction of the block wall. Geraldine realises that she should assess the temporary stability of the shotcrete wall with a near-vertical, 3m high, unretained cut underneath, and wishes to know the appropriate factor of safety.

The ground conditions are expected to be moderately weathered greywacke rock. The rock has been exposed in several places on site, and the rock level has been confirmed by a few shallow boreholes.

Consequence – Medium. Were the slope to fail, the land and shotcrete wall could be extensively damaged but there would be no damage to the house as it had not yet been built. Fatalities are unlikely.

Aspect	LoE	Value	Comment
Pre-construction investigation	II	0.4	Investigation comprised observations of rock exposures, and shallow boreholes. The depth to rock is well understood, but the rock strength is assumed.
Testing	IV	0.8	Rock has been observed on site, but no index testing has been carried out.
Analysis	Between III and IV	0.7	Standard computer slope stability modelling using effective stress strength parameters using assumed parameters.
Construction	II	0.4	Part-time monitoring by geoprofessional.
Operation & Maintenance	IV	0.2	Temporary works.
Sum		2.5	

For a medium consequence, the minimum FoS under Long-term Static conditions for LoE II is 1.3 and for LoE III is 1.7. As the assessed LoE is 2.5, the minimum FoS = 1.3 + (0.5) x (1.7 – 1.3) = 1.5.

13.20 EXAMPLES OF SITUATIONS WITH LOW FOS

In Table 10, some of the recommended FoS values are lower than the traditionally common value of 1.5 for Long-term Static conditions. In early drafts and discussions regarding this guidance, these low values of FoS have raised concerns. Below are presented some examples where a low minimum value of FoS is recommended by this guidance, and it is hoped that readers can see that there are situations where a low FoS would be of little concern. Indeed, in some cases, the situation would be of such low risk that a geoprofessional would seldom be engaged.

13.20.1 Example 6 – landslide above an access road

A small cut is proposed as part of a major earthworks project. An intense program of ground investigations has been carried out, plus laboratory testing such that the ground conditions and strength near the cut are well known. Full time professional supervision during construction is planned, and there will be regular long-term maintenance. The cut proposed is a minor part of these major works and is sitting above an access road (with alternative access available). Failure of the cut may temporarily block the access road, and the consequences are assessed as medium.

Medium Consequence, LoE I – recommended minimum FoS of 1.2

If instability of the cut would block only one lane of the access road, or less than one lane, then this would be a Low or Minor consequence, and the recommended minimum FoS would be 1.0.

Low Consequence, LoE I – recommended minimum FoS of 1.0.

13.20.2 Example 7 – landslide distant from structures and roads

If a landslide, be it in a fill, cut or natural slope, were to occur far enough away from structures and roads that it was implausible that the structure or road would be affected, or that people would be injured, then the consequence in assessed as minor. If very little investigation had been carried out, and the slope stability calculations were carried out based on assumed parameters based on observations of surface soils or nearby investigations, then the LoE would be IV, and the minimum FoS would be 1.4.

Minor Consequence, LoE IV, recommended minimum FoS = 1.4.

With increasing investigations, the factor of safety would reduce, with, at LoE I:

Minor Consequence, LoE I, recommended minimum FoS = 1.0.

Such minor consequences would seldom be assessed in any detail by a geoprofessional and hence it would be rare for the bottom row of Table 10 to be used.

13.20.3 Example 8 – small fill supporting local road

A low-height fill is proposed that will form the outside lane of a new local road, that slopes down at 26 degrees. An LoE II program of investigations has been carried out on the subgrade, including laboratory testing. Performance of nearby similar fills has been assessed and considered to be satisfactory. Part-time geoprofessional site observations are proposed in construction, with NDM testing by a geoprofessional on compacted fill. Periodic observations of the works after construction by a geoprofessional are proposed.

Instability of the fill could result in the outside lane of the local road becoming impassable to traffic – this is a Low consequence.

Low Consequence, LoE II, recommended minimum FoS = 1.2.

In most circumstances, for such a low consequence, a lesser level of engineering would be more common (LoE III or IV), resulting in more familiar minimum FoS values (1.4 – 1.7).

It is hoped that these examples demonstrate that cases where the FoS is less than the familiar values (FoS ~ 1.5) will be unusual. Commonly, experienced geoprofessionals intuitively calibrate their Level of Engineering to match the Consequence, and it is expected that, when Table 10 is used in practice, FoS values near 1.5 will commonly result.

14 METHODS OF ANALYSIS FOR SOIL

14.1 LEM VERSUS NUMERICAL METHODS

Once the slope geometry, soil strength, and pore pressures have been estimated, stability analyses can be carried out. Analyses generally utilise either limit equilibrium methods (LEM) or numerical methods[12]. Table 12 provides a 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.

14.2 MECHANICS OF LEM - PROCEDURES OF SLICES

Procedures of slices are a group of limit equilibrium methods in which an assumed failure mass is divided into a series of (typically) vertical slices and equations of static equilibrium are resolved for each slice to determine the factor of safety. These methods are applicable over a wide range of slope conditions and are incorporated into proprietary software packages making them the most common technique for stability analysis. Various procedures of slices have been developed and while they are all based on the same fundamental principles of static analysis, they make different simplifying assumptions. It is important to understand the underlying mechanics and simplifying assumptions of these procedures to grasp the implications of the necessary simplifications to the ground model and the representation of any slope reinforcement.

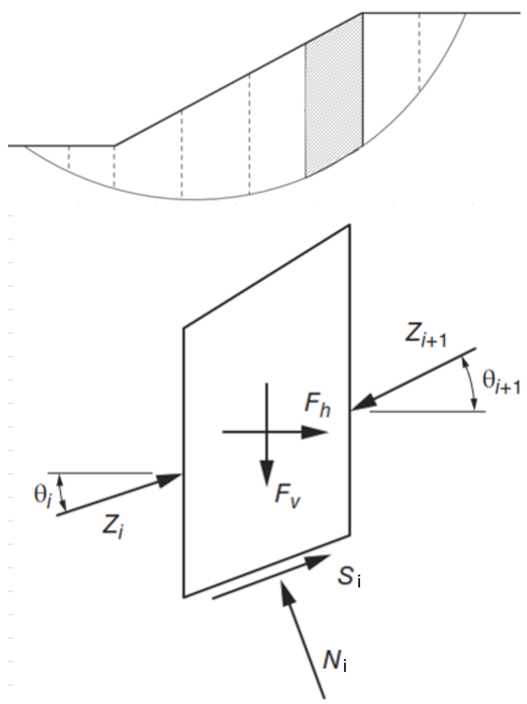
The calculations of procedures of slices are largely performed automatically by software with the user inputting boundary conditions, slope geometry, and inferred geology, groundwater conditions and geotechnical engineering material properties.

Procedures of slices include the following basic steps:

- Identify potential failure mechanisms and associated failure surfaces. Automatic search routines built into software packages search for the lowest FoS but the geoprofessional should guide and vet the results of these search routines based on a thorough understanding of the ground model. Section 15.1 provides some discussion.
- For each failure surface, divide the postulated failure mass into multiple vertical slices.
- Define the forces on each slice and using the equations of static equilibrium solve for the unknown forces and the overall factor of safety. The forces acting on each slice are shown in Figure 48.[13]

[12] Finite Element or Finite Difference based software like Plaxis or FLAC.

[13] Throughout this section, the subscript "i" is used to denote moments and forces on the ith slice.



F_v =	The sum of known vertical forces acting on the slice. This will include the slice weight and the vertical component of any surcharge loading or reinforcement that crosses the slice boundary.
F_h =	The sum of known horizontal forces acting on the slice. In the absence of surcharge, reinforcement or seismic forces, this value is zero.
S_i =	Resisting shear force along the base of the slice.
N_i =	Normal force at the base of the slice
Z_i	Magnitude of interslice force on the left side of slice
Z_{i+1}	Magnitude of interslice force on the right side of slice
Θ_i	Inclination of interslice force on the left side of slice
Θ_{i+1}	Inclination of interslice force on the right side of slice

Figure 48: Forces on a Slice (adapted from Duncan et al, 2014)

Table 12: 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 to derive a unique factor of safety. The critical failure surface location which has the lowest factor of safety is often identified using inbuilt search routines within limit equilibrium software.	**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. The upper groundwater surface is assumed to flow parallel to the ground surface. 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 Unit.	• They have been incorporated into powerful computer software capable of 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 relatively 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 or easy as stability charts or infinite slope methods. • Unlike more sophisticated numerical analyses, they don't produce an estimate of slope displacements. • Earthquake loading is simplistic.

Table 12: 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 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. **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.	• 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. • Allows for the prediction of failure mechanisms and deformations. • Has potential to better capture the slope response to seismic shaking.	• 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.

[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.

In the steps below and illustrated in Figure 49, we have used the Ordinary Method of Slices as an example of this resolution of forces to determine FoS. The Ordinary Method of Slices is one of the most straightforward procedures of slices and assumes a circular failure surface, neglects interslice forces, and satisfies only moment equilibrium. While useful for demonstrating the basic mechanics of procedures of slices, it is not recommended for general use (see Section 14.3 for preferred methods).

1. Determine the driving forces on the postulated failure surface for each slice (the total weight of material in the slice, W_i). This force is multiplied by the horizontal distance between the centre of the slice and the centre of the circular failure surface (a_i) to provide the slice's driving moment M_{Di}.

$$M_{Di} = W_i\, a_i \qquad \text{Equation 22}$$

$$a_i = r\sin(\alpha_i), \qquad \text{Equation 23}$$

where
α_i = inclination of base of slice from the horizontal.
r = radius of circular failure surface

2. Determine the available shearing resistance force on the postulated failure surface for each slice (S_i). Where there are no reinforcement elements, this is the shear strength of the soil (τ) as discussed in Section 10 multiplied by the slice thickness (l_i). This force, multiplied by the moment arm (the radius of the circular failure surface, r), represents the slice's resisting moment (M_{Ri}).

$$M_{Ri} = S_i\, r = \tau_i\, \ell_i\, r \qquad \text{Equation 24}$$

where

$$\tau_i = c'_i + (\sigma'_i)\tan\varphi'_i \qquad \text{Equation 25}$$

for effective stress soil strength as defined in Section 10.1

3. The FoS for the presumed failure surface is the sum of the resisting moments (for all the slices) divided by the sum of the driving moments. This process is repeated for multiple presumed failure surfaces to identify the "critical" failure mechanism (the failure surface with the lowest FoS). The FoS for any given surface is:

$$FoS = \frac{M_R}{M_D} = \frac{\Sigma M_{Ri}}{\Sigma M_{Di}} = \frac{\sum \tau_i \ell_i\, r}{\sum W_i \sin(a_i)} \qquad \text{Equation 26}$$

Additional points on the mechanics of LEM to keep in mind include:

- Some procedures satisfy all conditions of equilibrium while others do not. Procedures that satisfy all conditions are discussed in Section 14.3 and are recommended when performing analysis using computer software.
- There are more unknowns than equations when resolving forces for each slice. Assumptions must be made to obtain a statically determinate solution for the factor of safety. Different procedures make different assumptions, typically around the interslice forces and their inclination.
- The example given in this section assumes that the force driving instability is due only to the soil weight while the resisting force is due only to soil strength. Real-world scenarios often involve additional forces.

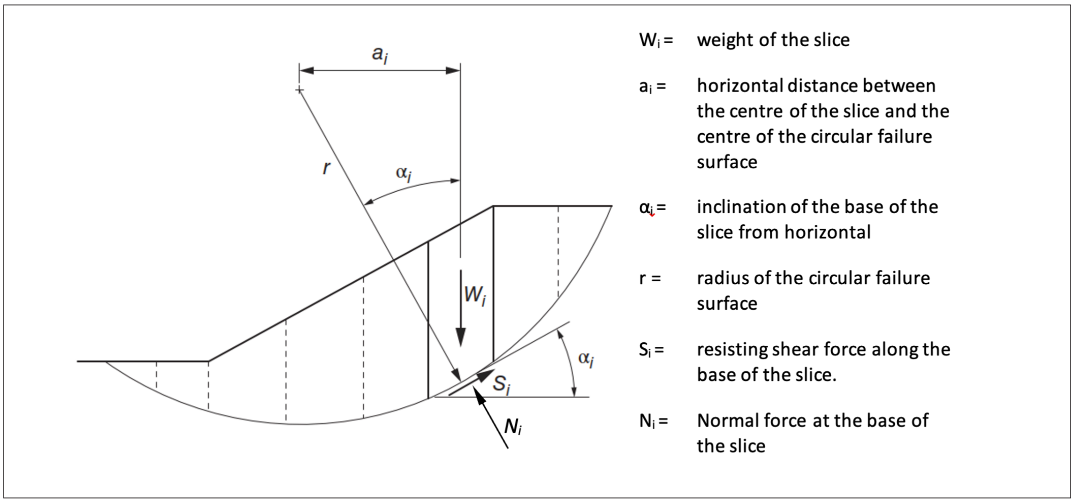


FIGURE 49: Ordinary Method of Slices – Forces on a Slice (adapted from Duncan et al, 2014)

These can include:

- External Loads: Such as water pressure, traffic, or stockpiled materials.
- Seismic Forces: Represented by horizontal body forces in pseudostatic analyses.
- Reinforcement Forces: Such as geogrid, soil nails, or stabilising piles.

These additional forces must be included in the equilibrium equations to accurately compute the factor of safety. Since these forces are known and defined as part of the problem, they don't require extra assumptions for a statically determinate solution (Duncan et al, 2014).

Fundamentally, the LEM analysis is determining how close to a state of limit equilibrium (FoS = 1.0) a presumed failure mass is based on the sum of all the available shear resistance that can be mobilised over the total length of the defined failure surface. This is important to remember when introducing stabilising elements (e.g. geogrid, soil nails retaining walls, etc.) into the analysis as part of remedial measures. The geoprofessional needs to understand how the stabilising contribution will be mobilised and then also accurately reflect this contribution in the stability model. This will be discussed in Unit 4.

14.3 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 are recommended and include:

1. Spencer's Method – assumes interslice forces are parallel to each other.
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 (1968) 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 & Schuster, 1996).

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

15 DETAILS OF LEM STABILITY ANALYSES

15.1 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[14].
- 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.
- There may be multiple significant surfaces that need to be considered. The critical surface is the significant surface with the lowest FoS.
- The critical failure surface can form the basis for determining, using back analysis, the shear strength value(s) needed to bring the slope to a point of limit equilibrium (i.e., a factor of safety = 1.0).

15.2 THREE DIMENSIONAL EFFECTS

15.2.1 Difference between 2D and 3D Analysis Methods

Most stability analysis of slopes in standard engineering practice is two dimensional. The 2D assumption in slope stability analysis is 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 (or sometimes multiple cross-sections). 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 discontinuity 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 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., 2D is conservative).

Where the slope is curved, or short, and hence is not well represented by a plane strain model, there may be significant benefit in accounting for 3D effects.

Where soil strengths are calculated in a back-analysis from 2D failure and these biases are not compensated for (see also Section 18 on back-analysis), then 3D analysis is required.

15.2.2 Three Dimensional LEM

The 2D LEMs discussed in earlier sections can be extended to provide similar 3D analysis. Slope failures always have a 3D shape, and hence 3D LEM can provide more insight into a slope problem than 2D.

Vertical columns are used to discretise the 3D slope (see Figure 50), with the base of each column representing the failure surface. There are several commercial software packages that offer 3D LEM capability.

The geometric inputs for a 3D LEM are readily obtained from LiDAR, GIS or CAD. A complex stratigraphic and/or structural model may be required to represent the slope, and appropriate material properties can be applied to surfaces, layers, or volumes. The 3D failure surface can be explicitly input into the model, or a simple search procedure can be used to find the critical surface.

In 2D analysis, there are several assumptions that need to be made when balancing force and moment equilibrium to calculate the FoS. There are several possible reasonable answers for each, and that is why there are so many different methods for calculating FoS (Spencer and Bishop are two examples). In 3D, even more assumptions are required. Depending on

[14] There are many algorithms that can search for non-circular failure surfaces, including cuckoo, auto-refine, and particle swarm searches. Designers should apply several different non-circular algorithms until they are satisfied that they have found the critical feasible surface.

the analysis procedure used, all conditions of static equilibrium may or may not be satisfied. Different procedures make different assumptions even when they satisfy the same equilibrium equations. As with 2D analysis, it is important in 3D analysis to use a procedure that satisfies full force and moment equilibrium, with only a few simplifying assumptions required to obtain a solution (e.g. Spencer's method).

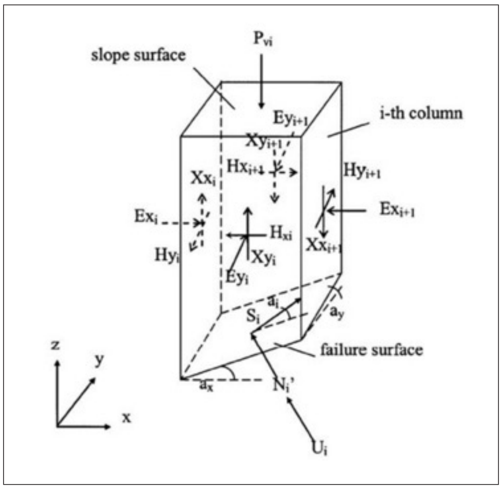


FIGURE 50: 3D view of forces acting on a column (Cheng & Yip, 2007)

In most cases the 3D FoS is greater than the 2D FoS, especially with slopes that are short (perpendicular to the direction of movement) or have a failure surface along a deep weak layer. In some cases, the 3D FoS may be lower – an example is the landfill slope failure at Kettleman Hills, California (Seed et al., 1988).

The benefits of 3D analysis are best shown in cases where slope reinforcement is utilised, or there are irregular point or surface loads on the slope. With 2D analysis it is difficult to model out of plane elements, and various assumptions are needed. Examples of the difference in FoS that result from consideration of slope reinforcement are given by Brown (2022).

3D pseudo-static analyses can be undertaken as part of screening for potential slope stability problems during strong earthquake shaking. Brown (2022) showed that using 3D analysis can reduce conservatism in seismic analysis, resulting in more economic design of slopes.

15.3 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 and/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. The existence of tension in analyses can be identified by examining the forces on the slices at the top of the slope. Tension is indicated where (1) interslice forces are negative (2) normal forces at the base of slice becomes negative or (3) the "line of thrust" (i.e. the line that connects locations of the interslice forces on slice boundaries) is located outside the slice. In slopes with cohesive soils at the crest, the geoprofessional should check the critical failure surface for tensile forces and introduce tension cracks where these forces affect results. Further details can be found in Duncan et al (2014).

15.4 ISSUES IN THE PASSIVE ZONE AT THE TOE OF THE SLOPE

Problems near the toe of the slope occur when the direction of the resultant force on the base of the last slice is a similar inclination to the interslice force leading to very large or negative forces. Where this occurs in analyses (1) the trial-and-error solution for FoS may not converge, (2) forces may become either very large producing very high shear strength in frictional soils, or (3) forces may become negative producing negative shear strength and much smaller than reasonable FoS (Duncan et al, 2014). There are multiple ways to address the issue including changing the slip surface inclination near the toe and using Ordinary Method of Slices procedure. Duncan et al (2014) provides further advice.

15.5 VERIFICATION OF RESULTS

Slope stability analysis results should be checked to ensure sensible 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 ask themself *"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.

16 MODELLING OF STRUCTURAL ELEMENTS IN SLOPES

To mitigate existing or potential slope instability, it is common to place structural elements within a slope. These elements could include plastic reinforcing, ground anchors, or piles. The modelling of these elements within a slope stability program is discussed in this section. In most cases, slope stability modelling is carried out to check for the possibility of deep-seated failure extending beyond most or all the structural elements. The detailed design (including durability) of these elements, the facing, and the connections between the elements and facing, is carried out separate to slope stability modelling, and is largely outside the scope of this guidance (but will be covered in Unit 4).

16.1 STRIP REINFORCEMENT

Strip reinforcements comprise horizontal (or nearly horizontal) layers of steel or plastic placed at vertical intervals within fill. They provide strength to Mechanically Stabilised Earth (MSE) walls or Reinforced Soil Slopes (RSS). MSE and RSS are usually designed using proprietary software developed by the manufacturer or distributor of the product. Such products are designed to check a suite of potential modes of failure, but it is recommended that geoprofessionals check deep-seated instability using a specialist slope stability program. Discussion on appropriate inputs to a slope stability program are described below. Many other matters are involved in MSE and RSS design, but these are typically covered in the propriety software, and the geoprofessional should confirm that this has been appropriately done. Design of MSE and RSS is described in FHWA (2009) and BS 8006-1:2010.

Plastic reinforcement, often called ‘geo-grid’, comprises high density polyethylene or polyester (FHWA, 2009). Geotextiles can also be used for strength, although it is more common to use them only for separation and drainage, and to disregard any strength benefit they may provide.

Steel reinforcement can comprise either steel strips or steel grids (FHWA, 2009).

The choice of which reinforcement to use in which situation is a wide topic not covered by Unit 3, but FHWA (2009) contains some guidance. This section assumes that an appropriate choice of reinforcement has been made, and that designers wish to know how to appropriately model that reinforcement in a slope stability model.

The choice of reinforcement used can be dependent on the facing type used – for example, steel reinforcement is usually used with concrete facing panels on MSEs, whereas geo-grid is commonly used to support soil-filled wire-mesh facing systems.

Strip reinforcements are modelled with the following:

- Allowable tensile strength - for plastic reinforcement, this is the ultimate capacity divided by several factors to account for loss of strength due to creep, installation damage, temperature and physical or chemical degradation. The calculation of allowable tensile strength for plastic reinforcement is provided in FHWA (2009), equation 3-12. The factors are usually provided by the manufacturers based on laboratory testing, although FHWA has some default values. The creep factor is particularly important, with a significant loss of strength likely to occur in plastics under long-term loading. The creep factor is not applied for seismic loading, meaning that the allowable seismic strength is usually higher than the allowable long-term strength. FHWA also provides a method of calculating the allowable tensile strength in steel, which accounts for the corrosion that may occur.
- Anchorage –strips are normally anchored to the slope face, using some type of anchoring system. In addition, the embedded end of the strip could be anchored within the slope, although this is not common. Which type of anchorage is being used should be defined within the slope stability program. If the strips are modelled as anchored to the face, a connection strength should be provided. Advice is provided in FHWA about the calculation of connection, but this is rarely a significant issue when considering deep-seated stability.
- Shear strength of reinforcement to soil interface – this can be derived from cohesion or friction, but FHWA recommends that only coefficient of friction is considered. Therefore, the cohesion within the reinforced block in the slope stability model must be set to zero under Long-term Static conditions. FHWA note that, in the absence of test results, the interface friction coefficient can be conservatively taken as 2/3 of the tangent of the friction angle (equation 3-9) for geotextiles, geogrids and ‘geonet type drainage composites’.

16.2 GROUND ANCHORS

Ground anchors are often used to stabilise slopes and new cuts. They are often used in conjunction with a facing, either a soft facing such as rock-fall mesh, or a hard facing such as reinforced shotcrete. Detailed information on the design of such systems is outside the scope of this guidance, and can be found in several publications, including FHWA (2015) and CIRIA report C637 (Phear et al., 2005; referred to in this document as CIRIA, 2005). Detailed information on ground anchor design, construction and testing is provided in NZGS (2023) which has been issued as a draft for public comment at the time of writing of Unit 3.

Some people and some publications distinguish between anchors and nails. It is sometimes said that an anchor is tensioned, and a nail isn't. Or sometimes that anchors are there to provide tension while nails are closely spaced to create a reinforced soil block. Sometimes the term "rock bolt" is used for ground anchors or nails in rock.

In the vernacular, many of these terms are used loosely and interchangeably[15]. Some people don't bother to distinguish between them. In this section, no distinction is made, and everything is called an anchor.

16.2.1 Modelling anchors in slope stability programs

Although design of anchored walls can be carried out in spreadsheets or specialist software (such as SNAILZ), much of the design, including the check for global stability, can be carried out in a slope stability program. To do so, the designer needs to assign the following:

- Bond stress, or adhesion, between the grout of the anchor, and the soil or rock providing resistance. The allowable value should be input to slope stability program, which is the ultimate bond strength multiplied by a capacity reduction factor. Ultimate bond strength should be established and checked by load testing of anchors on site. FHWA (2015) and NZGS (2023) provide guidance on the capacity reduction factor, suggestions for possible ultimate bond strength, and appropriate testing regimes.
- Tensile capacity of the tendon in the anchor. The tendons used in the anchors to provide tensile strength can be either steel bar, steel strands, or glass reinforced plastic. Again, the allowable value should be input, which is the ultimate yield strength of the tendon (provided by the manufacturer) multiplied by a capacity reduction factor from FHWA (2015) or NZGS (2023).
- For hard facings, the capacity of the head should be included – this needs to be calculated structurally, depending on the thickness of the facing and its reinforcement – FHWA has guidance on this. If the facing is soft, a head capacity can still be provided – this is dependent on the mesh and anchor plate being used – CIRIA (2005) has guidance.

Usually, the strength of the facing, be it shotcrete or steel mesh, is not included in the slope stability model. The assumption in slope stability modelling is thus that the facing is strong enough to resist small failures between the anchors, and this assumption should be checked using facing calculation methods provided in FHWA (2015) or CIRIA (2005).

16.2.2 Bond lengths and unbonded lengths

An anchor has a bond length and a free length (the free length is also called the unbonded or de-bonded length). The bond length is the length of tendon that is bonded to the grout and can transmit the applied tensile load to the surrounding soil or rock. The free length is the length of tendon that is not bonded to the surrounding grout. The free length may have several purposes, including:

- Corrosion protection. The materials providing the corrosion protection (for instance, plastic sheath) also de-bond the tendon from the grout. See further advice on corrosion protection in NZGS (2023).
- To soften the tendon's load-deflection behaviour. This may be desired because the anchor is connected to a retaining wall, and the designer wants the retaining wall to deform enough to engage active pressures, thereby reducing the forces on the wall.
- To make load testing easier. Having the top part of the tendon unbonded means that it is not critical if the test reaction frame is close to the grout.
- Often the anchor will go through a weaker stratum before embedding into the target stratum. The length of tendon through the weaker stratum isn't providing much resistance, and in the context of a slope stability model, it may be convenient to refer to this as the free length.

Bonded length limits:

- If there is no lock-off load, BS 8081: 2018 states that the bonded length should not be less than 3 m, unless the anchor is in rock and the design load is less than 200 kN, in which case the bonded length can be 2 m. This is because for short bond lengths, a sudden drop in rock or soil quality can induce a serious decrease in anchor holding capacity (BS 8081: 1989, Clause 6.2.3.4, there seems to be no equivalent clause in BS 8081: 2018). If, due to space constraints or other reasons, a shorter bonded length is required, then a high proportion of load tests should be carried out.
- If there is a lock-off load, FHWA (1999), page 74 suggests a minimum bonded length of 3 m.
- NZGS (2023) recommend a minimum bond length of 3 m and a maximum bond length of 10 m.

Minimum free length / unbonded length / de-bonded length:

- If there is no lock-off load, a free length of at least 1 m is recommended – FHWA (2015, p247) suggests 3 feet to allow for testing without the reaction load transferring to the bonded length.

[15] FHWA (2015) differentiates between them by saying that an anchor is post-tensioned and a nail isn't. NZGS (2023) is confusing because it states that soil nails are excluded from the document (Section 2) but also allows anchors to not be post-tensioned (Section 4.1 and footnote 1 on page 6). A non-tensioned anchors is a nail, so it is not clear what NZGS (2023) is excluding when it says it is excluding soil nails. Hopefully this confusion is cleared up in the final version.

- If there is a lock-off load, FHWA (1999) suggests a minimum unbonded length for rock and soil ground anchors of 4.5 m for strand tendons and 3 m for bar tendons. These minimum values are intended to prevent significant reductions in load resulting from seating losses during transfer of load to the structure following anchor load testing. Seating losses occur during load lock-off so are only relevant for post-tensioned anchors (FHWA, 1999, Section 7.5).
- NZGS (2023) recommends a minimum unbonded length of 3 m for bar tendons and 5 m for strand anchors, although it appears that these recommendations are intended for tensioned anchors only.

16.2.3 Shear strength of anchors

It is possible to model the shear strength of anchors in a slope stability model, in which case the resistance of the failure surface will be increased by each tendon through which it penetrates. This is not normally done and should only be undertaken with caution. To use the shear strength of anchors, designers would need to consider:

  - Do the anchors have enough bending capacity? Anchors are thin (relative to shear piles) and, under significant shear loading, may fail in bending at much lower loads than their shear capacity. Typically, only the bending capacity of the bar could be relied upon in a bending capacity calculation, with the grout's contribution not being reliable.
  - How much would the anchors deform laterally under the imposed shear force?
  - Does the anchor have sufficient passive resistance below the failure surface to mobilise the shear resistance that is being relied upon? A rule-of-thumb is that the anchor should penetrate at least as far beyond the failure surface as it does within the failure surface. A more accurate calculation of shear resistance of the anchor can be considered using a passive pressure equation considering the length beyond the failure surface.
  - Are the anchors close enough together (out-of-plane) to be providing genuine shear resistance? If the anchors are far apart, then soil could slip between them.

TRL Report TRL537 (Johnson et al., 2002) provide a meta-analysis of previous research on this topic, with the consensus being that bending resistance does not contribute significantly to the strength of a nailed slope. The contribution from bending stiffness is small unless the nails are oriented approximately normal to the failure plane; their stiffness is similar to that of the surrounding ground; a narrow, well-defined shear band forms; and significant soil movement occurs. If a geo-professional is considering using shear in anchors or nails, then they should refer to TRL537 and the numerous researchers and documents referenced in TLR537 Section 3.2.

16.3 SHEAR PILES

The use of vertical piles to stabilise slopes is a widely accepted and successfully applied method (Kourkoulis et al., 2011). These piles are called 'shear piles' or 'dowelling piles' with the resultant structure called an 'in-ground wall' or 'palisade wall.' Bored piles are the typical structural component, comprising concrete with bending capacity provided by either a steel reinforcing cage, structural steel post (for instance, Universal Beam) or timber post.

The shear piles provide a resistance (in the form of a shear capacity) that increases the factor of safety of the slope from an unsatisfactory value to a satisfactory value.

To model shear piles in a slope stability program, the user must enter a shear capacity (in kN/m) representing the shear capacity of each pile (kN) divided by the spacing of the piles out-of-plane (in metres). While this document is not a formal guidance on palisade wall design, it is noted that consideration should be given to:

- The point of application of the shear load above the failure surface.
- The bending capacity of the piles, considering that the point of maximum bending is likely to be at, or slightly lower than, the failure surface. Capacity reduction factors, as per the appropriate structural standard, should be used.
- The piles having sufficient depth below the failure surface, such that they have enough lateral capacity to resist the shear capacity entered in the model. This can be calculated by the method presented in Kourkoulis et al. (2012) with the following guidelines provided by Kourkoulis et al. (2011)– that the required embedment depth below the failure surface to achieve fixity conditions at the base of the pile is found to range from 0.7 – 1.5 times the depth of the failure surface. The 0.7 end of the range applies if the strength of the stable ground is three times the strength of the unstable ground, and the 1.5 end applies if the strength of the stable ground is equal to the strength of the unstable ground.
- The ratio of centre-to-centre pile spacing to pile diameter. A ratio of 2 - 2.9 has been used successfully on some documented projects (Edwards & Fairclough, 2018; Vessely et al., 2007; Rollins & Rollins, 1992). Kourkoulis et al. (2011) state that the spacing ratio is related to soil arching, that soil arching is guaranteed at a ratio of 2, and that piles behave almost as single isolated piles at ratios of more than 5. Kourkoulis et al. (2011) suggest that a ratio of 4 can be thought of as the most cost-effective arrangement because it has the largest spacing that will produce soil arching between the piles such that the inter-pile soil is adequately retained. However, a ratio of between 2 and 3 is

recommended unless previous projects in similar soil or rock have demonstrated success at higher ratios.
- The most cost-effective solution is to install the least number of piles with the maximum practically attainable reinforcement (Kourkoulis et al., 2011).
- For deep slides, if room allows, it may be beneficial to have lines of piles running parallel to landslide movement. If these piles are connected with a pile cap, then a significant amount of resisting force is provided by the axial load (tension and compression) and the pile design can be more efficient (Kourkoulis et al., 2011). In such cases, numerical modelling should be considered.

16.4 SMALL RETAINING WALLS

Retaining wall bases shallower than the critical failure surface will have no significant effect on the stability of the slope. Thus, when modelling existing slopes, small walls can be disregarded if they are considered unlikely to extend below the critical failure surface. However, if construction documentation or on-site testing suggests that they may extend below the critical failure surface, then they can be modelled as shear piles, using the methodology discussed above.

16.5 DEADMEN

A deadman is a vertical or horizonal structural member, such as a timber pole or concrete beam, embedded in the ground, and connected to a retaining wall or other slope facing using horizontal (or sub-horizontal) steel tendons. Slope stability modelling aspects of deadmen include:
- They provide lateral restraint to a retaining wall by passive pressure.
- They can be modelled in a slope stability program by choosing an 'end anchored' support type.
- Their capacity is entered on a per-metre basis. So, if they are vertical, the user should enter the passive resistance of one deadman divided by the horizontal spacing of the deadmen. If they are horizontal, the user should enter their passive capacity per metre length out of plane of the model.
- With the 'end anchored' support type chosen, the model will realise their full resistance, no matter where the failure surface intercepts the tendon. Designers must locate any deadman far enough from the wall or slope facing such that the passive wedge does not intercept any failure surface with a factor of safety less than the target value.

16.6 VEGETATION

The presence of trees and other vegetation can reduce the susceptibility of steep slopes to shallow landslides. This is an attractive mitigation measure for aesthetic reasons, and because it is low cost. A study in a large area of hilly Wairarapa farmland susceptible to shallow landslides found that although poplars and willows have the greatest positive influence on slope stability, an adequate plant density is more important than tree species (Spiekermann et al, 2021, 2022).

Although some researchers have calculated the effective cohesion that vegetation can provide, this will only apply within the depth of the roots and hence will only be, at most, 1 m or 2 m deep. Vegetation is thus only significant when considering shallow landslides which, as discussed in Section 11.4.1, are difficult to analyse in a slope stability model, and hence there will be limited benefit in modelling the vegetation as a small cohesion increase.

16.7 ACTIVE AND PASSIVE SUPPORT

When defining anchors or synthetic reinforcement in Slide2, the user must specify whether the supports are active or passive. SLOPE/W does not require the user to distinguish between anchor and passive supports.

Active supports are assumed by the slope stability program to decrease the driving force in the factor of safety calculation. Tensioned anchors, which exert a force on the sliding mass before any movement has taken place, can be considered as active support.

Passive supports are assumed to increase the resisting force in the factor of safety equation. Geo-textiles, deadmen, or un-tensioned anchors, which only develop a resisting force after some movement within the slope has taken place, can be considered as passive support.

16.8 MODELLING VERTICAL SURCHARGES

If there is a road, railway, building or other heavily loaded item near the crest of a slope, or anywhere else that it might destabilise the slope, then this should be modelled as a vertical surcharge in the slope stability program.

It may be, that for high slopes or deep-seated landslides, the effects of traffic, particularly if it is light, can be disregarded.

16.8.1 Pressures and loads

For light and medium traffic areas, such as driveways and car parks, with vehicles up to 10 tonnes, NZS 1170.1:2002 Table 3.1 provides guidance on appropriate surcharge pressures and loads.

For highways, the traffic loads and pressures are provided in Section 3 of the Bridge Manual (NZTA Waka Kotahi, 2022).

For railways, consult KiwiRail Standard C-ST-RW-4104 – Retaining Walls, Section 6.9 - this document is currently in preparation, but publication is expected to be soon.

For buildings, the recommended approach, for most situations, is to model the building as a uniformly distributed pressure, being the total weight of the building (including live loads) divided by building area. If it is a new building being considered, then the geoprofessional should ask the structural engineer for estimates of surcharge pressure. If it is an existing building, then some rough estimates of surcharge pressure will need to be made. As a preliminary estimate, it is suggested that timber framed structures are modelled at 4 – 6 kPa per storey and concrete framed structures at 10 – 12 kPa per storey. If it appears that the surcharge pressure from the existing building is having a significant effect, then a structural engineer should be included in the project to provide the best possible estimates of surcharge pressure – although, unless there are as-built plans available, these will still be approximations.

If there is a heavily loaded foundation near the crest of the slope, it would be appropriate to also model individual footings and their applied pressures (which will be much more than the uniformly distributed building pressure) to assess their effect on the slope stability.

16.8.2 Load Combinations and Surcharge

The most authoritative New Zealand systems for load combinations, NZS 1170.0:2002 and the NZTA Bridge Manual, use partial factors on loads. As any slope modelling of surcharges is in effect a load combination (being a combination of, at least, surcharge and earth pressure) then a partial factor approach is required. This section presents a method of applying partial factors to slope stability modelling.

For structures, NZS1170.0:2002 Clause 4.2.1 (b) (vi) states that, for combinations that produce net destabilising effects (such as slope instability):

$E_d = [1.2G, S_u, \Psi_c Q]$

Where:

E_d = design action effect.

G = permanent action (self-weight or 'dead' action).

S_u = the action from snow, liquid pressure, rainwater ponding, ground water or earth pressure. For the purposes of this guidance, S_u is earth pressure.

Ψ_c = combination factor, as per Table 4.1 of 1170.0.

Q = imposed action (due to occupancy and use, 'live' action).

Clause 4.2.3 (f) states that, for earth pressures:

$S_u = 1.0\ F_{e,u}$ when $F_{e,u}$ is determined using an ultimate limit states method.

$S_u = 1.5\ F_e$ when determined using other methods.

Where:

F_e = earth pressure action.

$F_{e,u}$ = ultimate earth pressure action.

Now, compare the equations of factor of safety with those of a system of partial factors. In a factor of safety approach, for the stability of the slope to be satisfactory:

$RF / DF \geq FOS_T$

Where:

RF = resisting forces.

DF = driving forces.

FOS_T = target factor of safety.

Whereas in a partial factor system:

$(\phi\ RF) / (LF\ DF) \geq 1.$

Where:

ϕ = reduction factor on resisting forces.

LF = load factor on driving forces.

Combining these two equations, and re-arranging, it follows that:

$LF / \phi \geq FOS_T$

If there is no surcharge or earthquake, then LF and ϕ relate solely to earth pressure. NZS 1170.0:2002 Clause 4.2.3 (f) (written above) implies that the load factor on earth pressure is 1.5. Module 6, equation 6-4, also states that the load factor on earth pressure when designing retaining walls is 1.5 – indeed, its derivation seems to come from NZS1170.0:2002. Therefore, it appears that, if slope stability were considered as a partial factor system, then LF for earth pressure would be 1.5, and hence for the common case where $FOS_T = 1.5$, ϕ must be 1 – that is, there is no reduction factor on soil strength.[16]

As the load factor for earth pressure can be taken as 1.5, then it follows that, for Long-term Static analyses, the load factors on other driving force types should also be as per NZS1170.0:2002 Clause 4.2.1 (b) (vi), with permanent loads multiplied by 1.2 and temporary loads by Ψ_c. Because most traffic loads are repetitive and frequent, it is recommended that, when including the temporary surcharges in the Long-term Static analysis, the modelling is with drained analysis.

When the surcharge is large, and the slope is low or of marginal stability, the surcharge could be significant. It is noted that Ψ_c rarely exceeds 1 in NZS1170.0:2002, and hence the possibility that the surcharge might be greater than expected would seldom be considered in the relationship $Ed = [1.2G, S_u, \Psi_c Q]$. More generally, NZS 1170.0:2002 allows for unexpectedly high surcharge in the load combination [1.2G, 1.5Q] (Clause 4.2.1 (b) (ii)) but not when in combination with earth pressure (S_u). However, NZS 1170.0:2002 is primarily for the design of buildings not slopes, and hence earth pressure could be

[16] In the opinion of the Unit 3 authors, this is the wrong way around. It is slope geometry and soil density that is usually quite well known, hence the driving force is fairly well understood. It is the resisting force that is most prone to uncertainty.

Table 13: Load factors for vertical effects of surcharges.

Loading Condition	Load factor on permanent vertical surcharge	Load factor on temporary vertical surcharge	Minimum FoS
Long-term Static (drained conditions)	1.2	Ψ_c as per Table 4.1 of 1170.0.	As per Table 10
High Ground Water conditions	1.0	Ψ_c as per Table 4.1 of 1170.0.	As per Table 11
Traffic loading (drained conditions, with undrained conditions checked for low permeability soils)	1.0	1.5	1.2
Earthquake loading	1.0	Ψ_E as per Table 4.1 of 1170.0.	See Section 17 for approach for earthquake cases.

seen as, in essence, a building dead weight in the [1.2G, 1.5Q] combination. So, in a case where surcharge effects might be significant, it is recommended that designers apply a load factor of 1.5 on temporary surcharges, and 1.0 on permanent surcharges (if present) and because the load factor on G is 1.2, and the earth pressure is G in this instance, design for a factor of safety of 1.2. As this case is expected to occur rarely, it should be carried out in association with undrained conditions for low-permeability soils.

Similarly, there is no consideration of a combination involving earthquake forces and earth pressure in NZS 1170.0:2002. However, in the combination [G, Eu, ψ_EQ], earth pressure could act as a surrogate for dead load (G), allowing for a combination of earthquake and surcharge. In the earthquake case, permanent surcharge should remain unfactored, and temporary surcharge multiplied by ψ_E. When temporary surcharges are small, and the slope is high, temporary surcharges can usually be deleted from seismic models without a significant effect on the factor of safety.

In conclusion, when modelling surcharges in slope stability programs, the load factors for surcharges should be as per Table 13. The appropriate shear strengths, drainage conditions and pore pressure conditions for each loading condition should be as per Table 6.

For highways, load combinations are provided in Section 3.5 and Table 3.3 of the Bridge Manual. A similar approach could be taken for slope stability partial factors as that discussed above, noting that the partial factor on traffic can be as high as 2.25 and that there need be no traffic consideration in seismic conditions.

17 SEISMIC SLOPE STABILITY

Seismic performance of both natural and manmade slopes is a critical consideration to the design of the built environment in much of New Zealand due to the country's high seismicity and rugged terrain. Assessment of seismic slope stability is introduced in Part 7 of Unit 1; the aim of this section is to provide more in-depth discussion and guidance on the topic.

17.1 BACKGROUND

17.1.1 Observations of Slope Performance

Earthquakes are a long-recognised cause of landslides in seismically active regions worldwide. In many cases, coseismic landsliding accounts for a significant portion of total earthquake damage (Jibson, 2007). These historic failures improve our understanding of the drivers and characteristics of these instabilities and are useful for validating the results of seismic slope stability modelling.

In natural slopes, which commonly consist of a layer of highly jointed/weathered rock or colluvium over more competent material, by far the most common seismically induced type of slope failure is shallow, disaggregated[17] slides (Keefer, 2002). These are often observed to be concentrated in the middle to upper parts of hillslopes, likely due in part to topographic amplification (Brabhaharan et al., 2018). These slides can lead to rock avalanches and, depending on the saturation of the slope materials, debris flows and mud flows. While generally shallow they can cover large areas, and therefore produce high volumes of debris and cause extensive damage. Rock falls are also frequently observed in natural slopes but are not as common as shallow sliding. Deep-seated slides (rotational and translational) are rarer and highly dependent on the underlying geology (presence of lower strength sliding surface) but can be very destructive. Where slope stabilisation measures have been undertaken, they tend to perform well relative to areas where no stabilisation has been carried out (Brabhaharan et al., 2018).

Fill slopes tend to be constructed of relatively ductile materials and are typically subject to deeper modes of seismic shear failure (Jibson, 2007). These types of failures may be able to accommodate limited displacement before complete mobilisation of the basal rupture surface and catastrophic ground failure occurs (Murphy & Mankelow, 2004). Performance of well compacted fill or embankment slopes during earthquakes has generally been good except where liquefaction of the foundation soil occurs (FHWA, 2011).

Within New Zealand many thousand earthquake-induced slope failures have been documented since 1840. These failures are primarily influenced by earthquake magnitude, slope angle, and ground conditions, with significant landsliding occurring at magnitudes of 6 or greater. The most common types of coseismic landslides in New Zealand are shallow disrupted falls, slides, and avalanches of rock, debris, and soil. These typically involve translational sliding on the soil to rock interface or sliding and release on rock discontinuities. Deeper, more coherent coseismic landslides with limited displacements often occur on slopes of fine, cohesive soils in areas of Tertiary mudstone and weathered volcanic tephra deposits (Brabhaharan et al., 2018). A study of coseismic landsliding in New Zealand prepared by GT Hancox of GNS Science is provided in Appendix A of Brabhaharan et al. (2018).

Slope performance and landsliding in the 2011 Christchurch Earthquake and the 2016 Kaikōura earthquake have been extensively documented and are summarised below.

<u>February 2011 Christchurch Earthquake</u>

A series of large earthquakes occurred in 2010 and 2011 in the Canterbury region, the most significant of which was a M6.2 Christchurch Earthquake that occurred in February 2011. This event triggered several types of mass movements in the Port Hills, an eroded remnant of the Lyttelton Volcano on the south flank of Christchurch. These failures as outlined in Dellow et al. (2011) include:

- Rockfalls – boulder rolls of joint-controlled, dislodged lava blocks from lava-flow outcrops (Figure 51).
- Collapses of steepened sea cliffs.
- Large landslides with limited deformations (typically less than one metre) in the loess deposits overlying volcanics and in deeper loess interbedded with marginal marine sediments at the base of the hills (Figure 52).
- Minor but widespread failures of retaining walls and settlement of poorly compacted fill.
- Details of individual mass movement areas are documented in a series of reports prepared by GNS available on the Christchurch City Council website (Port Hills GNS reports : Christchurch City Council (ccc.govt.nz)). A summary of these mass movement areas is provided in Massey et al. (2013).

[17] Disaggregated or disrupted slides refer to those with a landslide mass that breaks up once mobilised.

FIGURE 51: Rolling boulder damage to a house in Rāpaki (Dellow et al, 2011).

FIGURE 52: Tension crack on the Kinsey Terrace landslide (Dellow et al, 2011).

2016 Kaikōura Earthquake

The 14 November 2016 M_w7.8 Kaikōura Earthquake generated tens of thousands of landslides and more than 200 significant landslide dams (Dellow et al., 2017). The most intense landsliding was concentrated in the area around fault rupture zones and the large majority of landslides were disrupted slides and falls in rock and debris of shallow to moderate depth (~1–10 m) (Figure 53 and Figure 54). Dozens of large, deep landslides which remained more coherent were triggered primarily in the weaker Neogene[18] sedimentary rocks (Jibson et al., 2018; Dellow et al., 2017) (Figure 55 and Figure 56). Many of these slides blocked valleys and dammed rivers creating downstream hazards (Figure 55). Widespread ground cracking on steep slopes, likely related to incipient landsliding, was notably concentrated at the ends of ridges, probably due to topographic amplification effects (Jibson et al., 2018).

Large and small landslides blocked road and rail corridors in many places, most consequentially along State Highway 1. Rock slides, disaggregated rock mass failures and debris avalanches on high greywacke hillslopes caused most of the prolonged closure of the transport corridors (Mason et al., 2023). Fill embankments experienced widespread seismic compression and shear displacement resulting in varying degrees of deformation (Mason et al., 2023) (Figure 57 and Figure 58).

The landslides created significant post-earthquake hazards predominantly related to debris flow and additional landsliding of earthquake-damaged slopes during post-earthquake rainfall (Dellow et al., 2017; Mason et al., 2023).

[18] A geologic period comprising the Miocene and Pliocene epochs, spanning between approximately 23.03 million years ago (mya) and 2.58 mya.

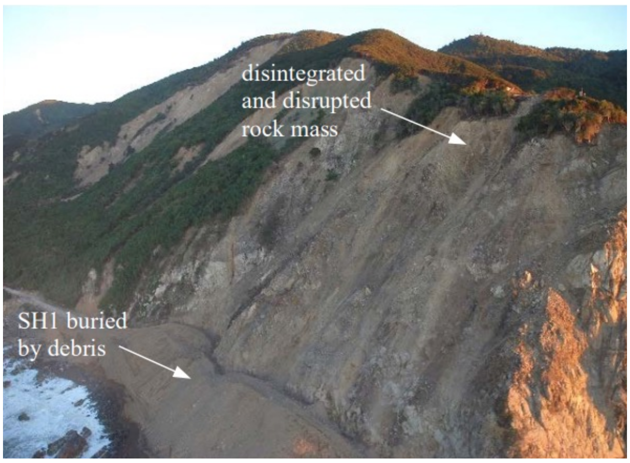


FIGURE 53: Disaggregated avalanche-type rock mass failure in Greywacke at Ohau Point (Mason et al., 2023).

FIGURE 54: Landslides in the Seaward Kaikōura Range (Jibson et al., 2018).

FIGURE 55: Leader 220 landslide dam (Jibson et al., 2018).

FIGURE 56: Main scarp and upper part of the Sea Front landslide (Jibson et al., 2018).

FIGURE 57: Translational failure of fill embankment and gabion retaining wall (RW) north of Kaikōura (Mason et al., 2023).

FIGURE 58: Cracks along outboard edge of mountain road above Mount Lyford (Jibson et al., 2018).

Mason et al. (2023) provide a useful summary of cut and fill slope performance along the transport corridor during this event and is recommended reading.

Experience in New Zealand, particularly in the Kaikōura Earthquake, broadly reflects experience with coseismic landsliding globally. Notably that:

- Most coseismic landslides in natural slopes and cut slopes are relatively shallow disrupted slides.
- Failures are often initiated in the upper part of slopes possibly due to topographical amplification effects.
- Deep seated failures are less numerous but can be very destructive and can significantly contribute to post earthquake hazards. These are more common in weaker, fine-grained Tertiary rocks.
- Constructed slopes tend to undergo limited displacement.
- Landslides are more likely to occur after an earthquake than before because the ground has been weakened. The trigger of post-earthquake landslides could be either rainfall or earthquake aftershocks.

Experience from landsliding in past earthquakes guides our assessment of expected performance of slopes in future seismic events and aids in validating results of quantitative analysis.

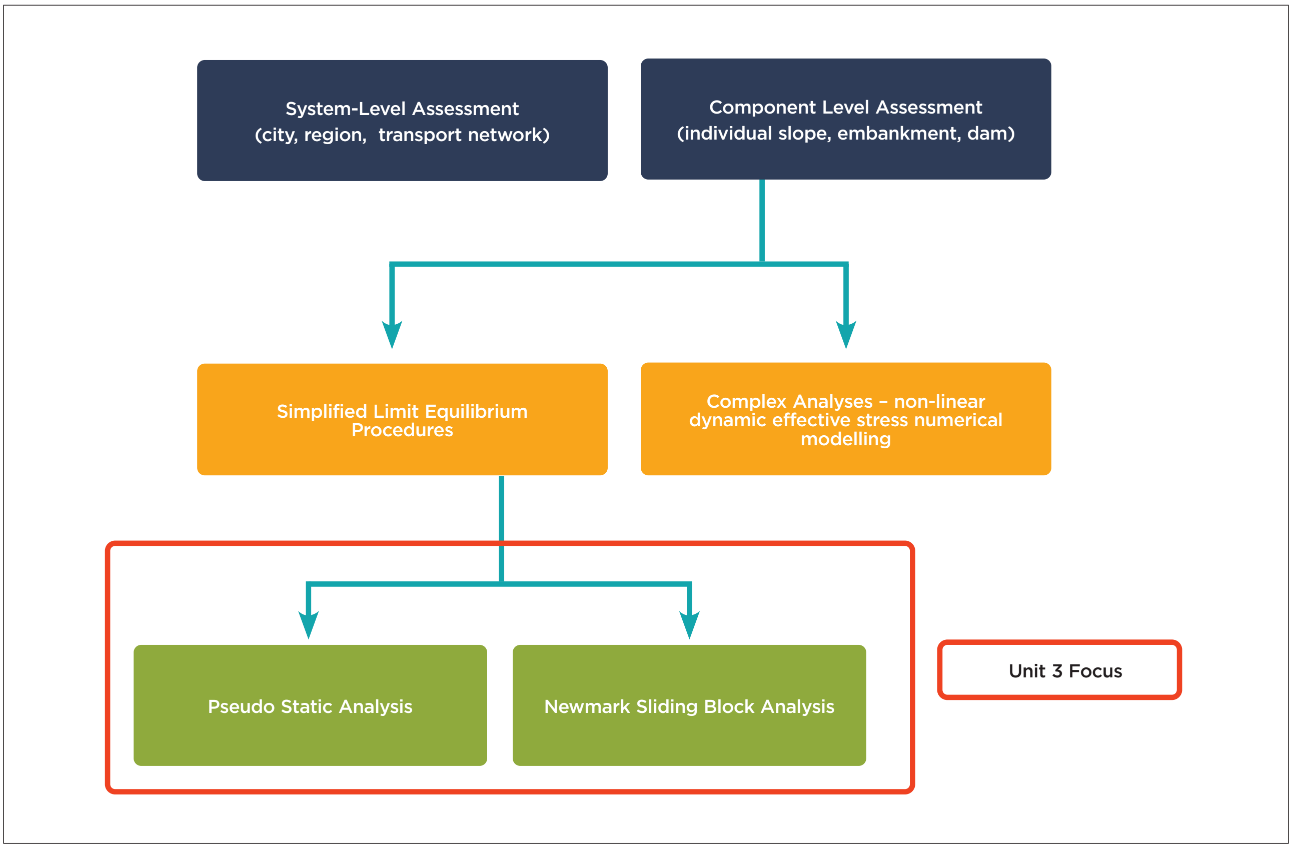


FIGURE 59: Seismic Slope Stability Assessment Methods – Unit 3 Focus

17.1.2 Assessment of Seismic Stability

Seismic stability assessment methods vary based on the geographic scale of the assessment and the complexity of the project and geological conditions and can be broadly divided into system-level and component-level assessments, as illustrated in Figure 59.

- System-level assessments, which often cover large areas, evaluate the performance and resilience of systems such as cities, regions, and transport networks. These assessments combine predictive regional seismic landslide modelling with consequence assessment to understand a system's risk and resilience.
- Component-level assessments analyse the seismic stability of specific components like bridge embankments, cut slopes, or dams. These methods can range from relatively simple (like pseudostatic and Newmark Sliding Block procedures) to complex non-linear dynamic numerical modelling. Simplified limit equilibrium-based approaches for assessing seismic slope stability are commonly used by geoprofessionals in New Zealand and globally. Design standards and guidelines for geotechnical practice often reference these simplified approaches (e.g. FHWA, 2011; MBIE Module 6; NZTA Waka Kotahi, 2022)

This guidance focuses on limit-equilibrium-based, simplified procedures for evaluating individual slope performance during earthquakes. Current practice is to use either:

- Pseudo-static – Limit equilibrium using a pseudo-static representation of seismic force (Section 17.5), or
- Displacement-based analysis using the Newmark sliding block concept (Section 17.6).

Pseudo-static methods tend to be used as screening analyses with displacement analyses carried out where pseudo-static checks indicate either failure, or exceedance of a threshold displacement.

More sophisticated analysis procedures, which involve dynamic effective stress numerical modelling and incorporate non-linear stress-strain properties of soils, are briefly discussed but are largely beyond the scope of Unit 3.

It's important to remember that earthquake engineering draws extensively on empirical data. Each significant earthquake provides valuable insights, leading to updates in our methods. Therefore, it's essential to keep abreast of new methodologies, as they are likely to replace the ones mentioned in this guidance.

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

1. Are there soils within or beneath the slope that could liquefy? This is an important consideration in some geological environments, typically natural or fill slopes in lower lying areas of New Zealand such as in alluvial, lacustrine and coastal environments. 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 extensively in Module 3.
2. If there are no soils that will undergo liquefaction, then what seismic deformations, if any, are expected?

A generalised process for utilising simplified seismic slope stability procedures to address these issues is shown in Figure 60 (reproduced from Part 7 Unit 1). There are several components to this process which are detailed in the following sections and include:

- Estimating Seismic Demand
- Estimating Dynamic Soil Strengths
- Establishing Threshold Deformation Limits or Interpreting Deformations
- Carrying out Seismic Analyses
  - Post-Seismic Check,
  - Pseudo-Static Analyses,
  - Newmark Deformation Analyses

INTERDEPENDENCY OF SIMPLIFIED ANALYSIS COMPONENTS

Simplified seismic slope stability procedures reduce a complex and dynamic process to a pseudo-static one, with the assumption of constant soil strength. It is important to recognise the interdependency of the seismic demand, soil strength, and deformation, to properly account for the simplifying assumptions. Figure 60 illustrates the relationship between these elements.

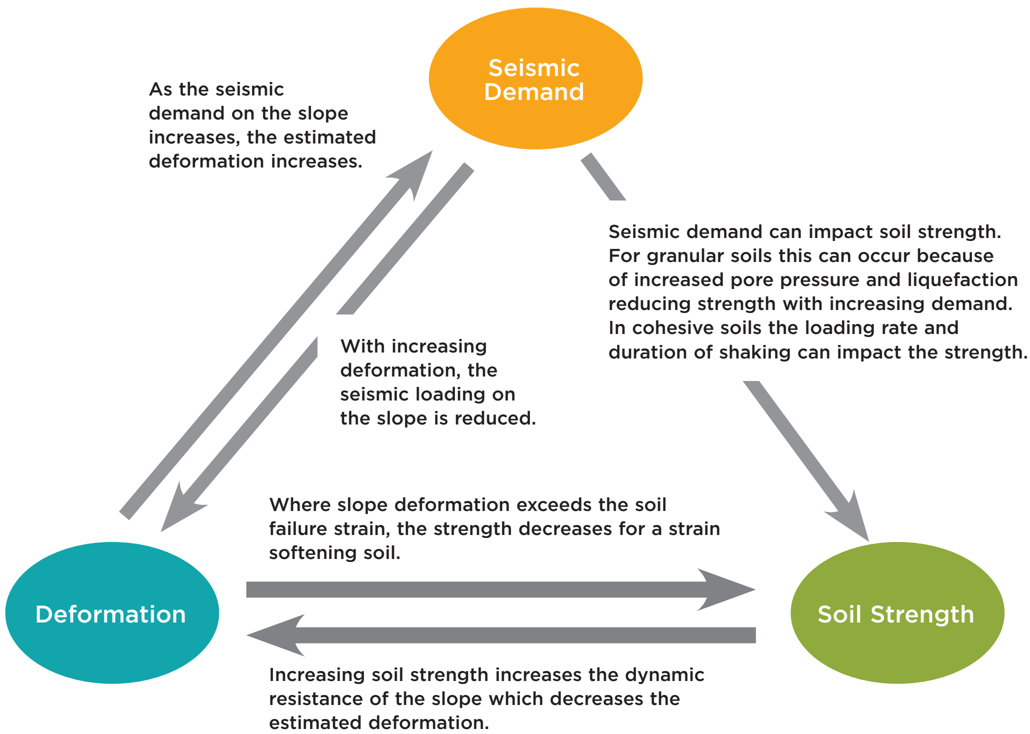


FIGURE 60: Interdependency of components of simplified seismic slope stability procedures

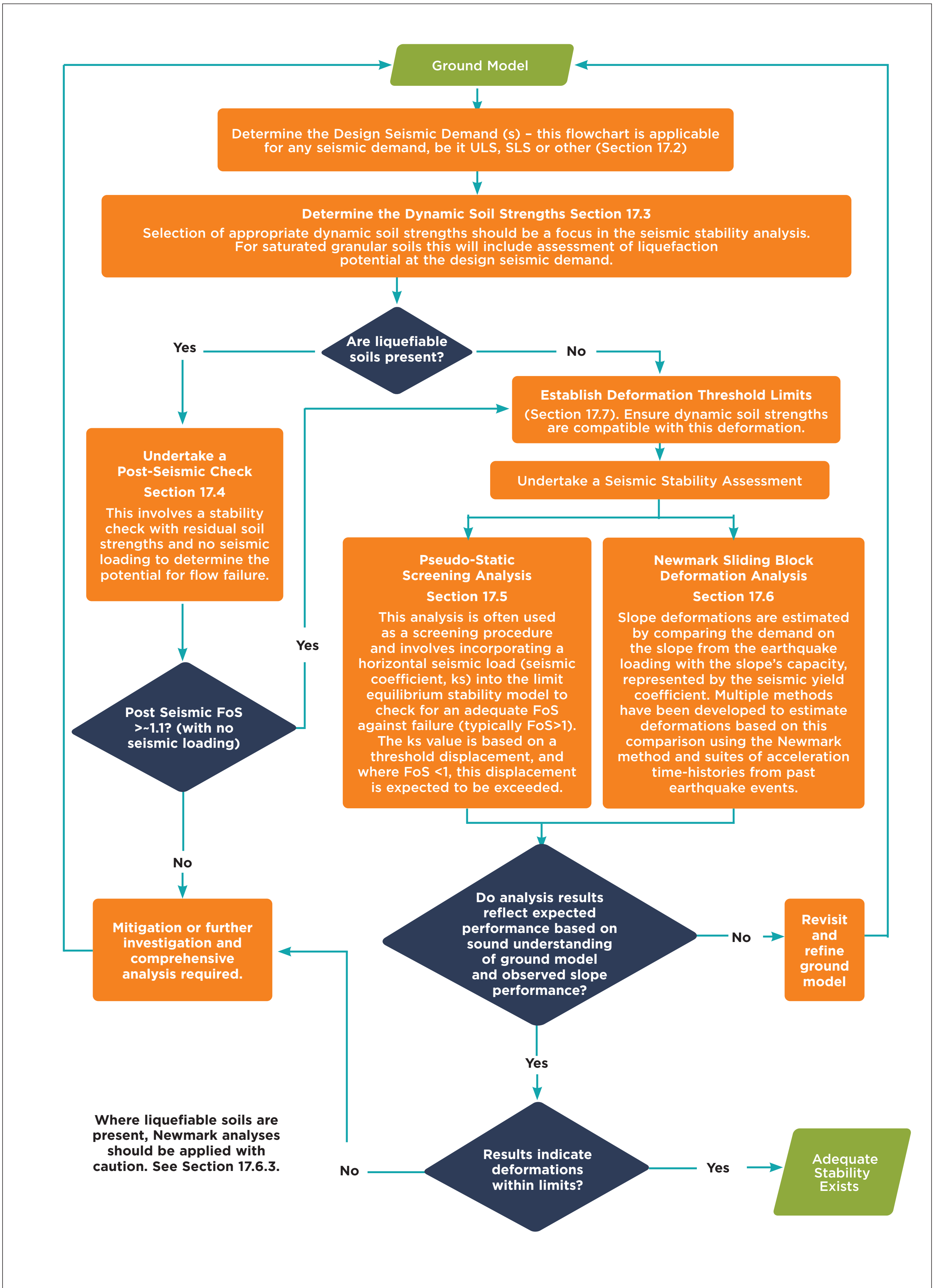


FIGURE 61: Simplified Seismic Slope Stability Assessment Process

17.2 SEISMIC DEMAND

To define the seismic demand, we first need to identify the return period of the earthquake of interest and then determine the ground motion parameters (e.g. M_w, PGA) for that event. The parameters required vary based on the analysis method. Some methods consider the slope's dynamic response, so the height or fundamental period of the slide mass are needed to calculate the seismic demand. Therefore, geoprofessionals should first familiarize themselves with the requirements of their chosen analysis methods (covered in Section 17.6).

17.2.1 Return Periods for Assessment

The choice of return period for analysis largely depends on the type of asset associated with the slope and its Importance Level. For slopes related to roads, NZTA/Waka Kotahi has defined return periods associated with performance load cases (e.g., Damage Control Limit State, DCLS) in the Bridge Manual (NZTA Waka Kotahi, 2022). For dams, guidance is provided in NZSOLD (2023).

For slopes associated with structures, NZS1170.0:2002 specifies two load cases - the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS), along with their corresponding return periods. These periods depend on the structure's Importance Level and design life.

While NZS1170.0:2002 specifies discrete earthquake load cases (ULS and SLS) for assessment, it is recommended that stability in intermediate return period earthquakes is assessed if slope movement is triggered between ULS and SLS. This helps identify a step change in performance.

Figure 62 illustrates how two sites (Case A and Case B) can exhibit similar performance at SLS and ULS return periods but pose different risks due to variations in the return period at which performance changes significantly. A performance-based approach, which assesses slope performance across a range of earthquake return periods, provides a clearer understanding of the seismic slope stability hazard.

17.2.2 Seismic Demand Parameters

Seismic demand parameters required for use with simplified Newmark or pseudo-static procedures vary depending on the specific method of analysis. These parameters may include Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), spectral acceleration at a period of interest (Sa(T)), moment magnitude (M_w) of the earthquake, and mean period of the earthquake ground motion (T_m). Sections 17.5 and 17.6 discuss the seismic demand parameters required for various pseudo-static and Newmark displacement procedures. Table 14 summarises methods for deriving these parameters.

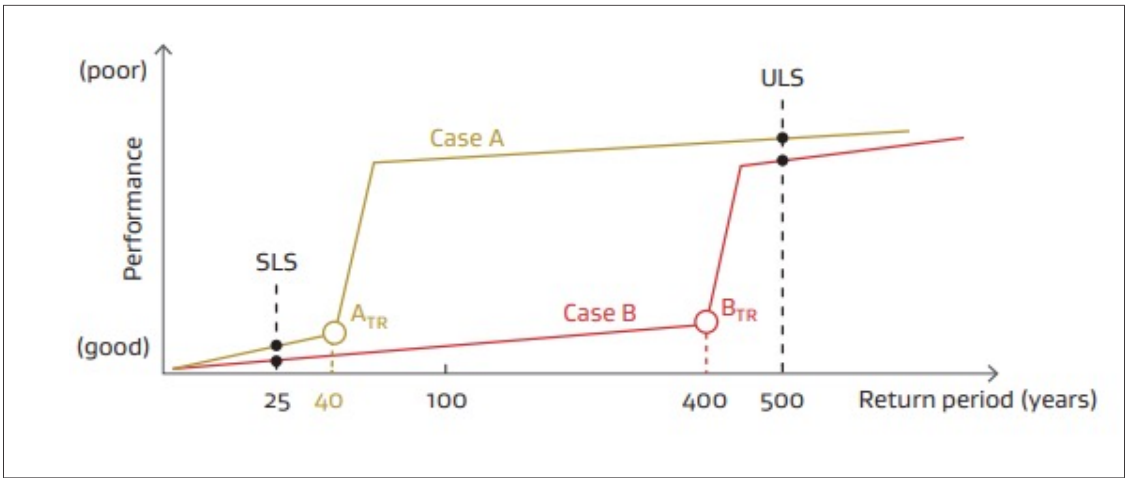


FIGURE 62: Step change in performance reproduced from Module 1

Table 14: Methods for Deriving Seismic Demand Parameters

Parameters Provided	Derivation Reference/ Method	Description
PGA, Sa(T), M_w	2022 National Seismic Hazard Model, (NSHM) Webtool	A New Zealand wide generic Probabilistic Seismic Hazard Analysis (PSHA) has been completed using the recently updated NSHM (Gerstenberger et al., 2022) and a webtool developed to provide results for a range of input parameters (location, return period, V_{s30}[1]). NZ NSHM (gns.cri.nz)
	TS 1170.5:2024	TS 1170.5, recently released for public comment, updates design seismic loading based on the 2022 update of the NSHM. NZS 1170.5:2004 remains the referenced standard for compliance with the New Zealand Building Code but is not recommended for use to derive seismic demand parameters for slope analysis. At the time of writing of this Guidance, TS 1170.5 is still in draft for public consultation, but once released as final it is expected that it will be a recommended source of seismic demand parameters.
	Local Generic PSHA	Some city and regional councils may commission generic PSHA specific to a region or urban centre.
PGA, PGV, Sa(T), M_w	Site-specific PSHA with or without site response analysis and ground motion studies	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 and the TS 1170.5 Commentary (DZ TS 1170.5 Supp 1:2024).
PGA, M_w	Module 1 Appendix A	Module 1 provides estimates of PGA and M_w based on generic PSHA run for multiple New Zealand locations (Cubrinovski et al., 2022). This method provides interim guidance for routine projects until updates to the NSHM (2022) are incorporated into NZS 1170.5. This method is included here for completeness as TS 1170.5 has not yet been finalised, but finalisation is anticipated shortly and once finalised we understand that earthquake demand parameters in Module 1 will be superseded.
T_m	Rathje et al. (2004)	An empirical relationship is presented to estimate the mean period of an earthquake ground motion (T_m) based on earthquake magnitude, distance to the fault rupture plane, and Site Class (using the Simplified Geotechnical Site classification system described by Rodriguez-Marek et al. (2001) which is slightly different than that described in NZS 1170.5: 2004).
PGV	Anderson et al., (2008); NGA-West 2 GMPEs	Anderson et al. (2008) provide a relationship to determine PGV from the spectral acceleration at one second (Sa(1s)) and earthquake magnitude based on work by Dr Norm Abrahamson. Sa(1s) can be obtained from the NSHM or site-specific PSHA. Estimates of PGV can also be obtained from the NGA-West-2 Ground Motion Prediction Equations (GMPE). Spreadsheet implementation of these models is available through the Pacific Earthquake Engineering Research Center (PEER) website.

[1]V_{s30} is the time-averaged shear wave velocity in the upper 30m of the subsurface profile.

WHAT METHOD SHOULD I USE TO DERIVE SEISMIC DEMAND PARAMETERS?

The decision of which method in Table 14 to use to define the magnitude, PGA, and spectral acceleration will depend on the availability of site-specific studies. Where a site-specific PSHA, or local generic PSHA has been carried out, results of these studies should be used. These studies should be at least as rigorous and up to date as the PSHA performed in the 2022 NSHM. They "... must be comprehensive and based on best-practice scientific interpretations, which include rigorous considerations of uncertainties in the assessment, and their implications on the computed results and interpretations. An independent peer review of site-specific studies is strongly recommended" (DZ TS 1170.5 Supp1:2024). Where site-specific studies are available but out of date, the 2022 NSHM or TS 1170.5 (when finalised) should be used.

For most routine projects, site-specific studies will not be available. In these cases, either the NSHM (2022) webtool or TS1170.5 (when finalised) should be used. TS1170.5 is based on the mean hazard curves from the 2022 NSHM so the seismic demand parameters obtained from these methods should be broadly similar for a given V_{s30} as illustrated in Figure 63. TS1170.5 provides detailed guidance for estimating or measuring V_{s30} and incorporating uncertainty in V_{s30} estimates. The guidance in TS1170.5 should be used for calculating V_{s30} for use in either the NSHM (2022) webtool (where V_{s30} is a required input) or in TS1170.5 (where V_{s30} is used to define Site Class for construction of hazard spectra).

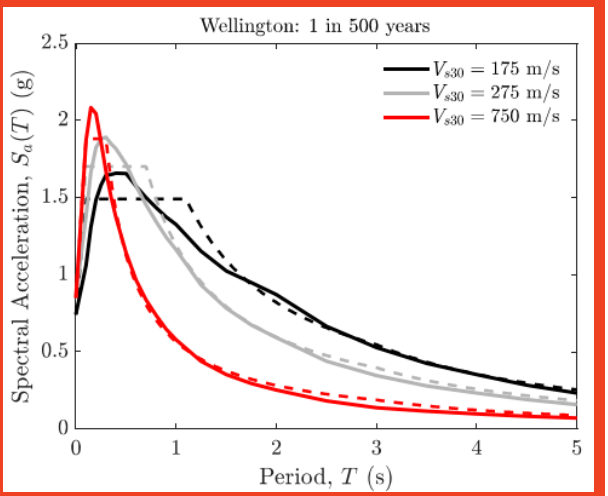


FIGURE 63: Comparison of elastic Uniform Hazard Spectra (UHS) acceleration demands from the NSHM 2022. Solid lines represent NSHM 2022 values and dashed lines represent the UHS (DZ TS 1170.5 Supp 1:2024).

17.2.3 Topographic Amplification

Ground shaking can be significantly amplified by topographic features such as long ridges and cliff tops. Seismic waves reflect down off the ground surface that form sides of hills. The reflected waves have a time lag and phase difference from incoming waves, leading to amplification or attenuation. Closer to the crest of a hill more wave interaction occurs leading to more amplification. For further details, Brabhaharan et al (2018) provides a comprehensive literature review of research on this subject.

The main points in the current understanding of topographic amplification include:

- Topographic amplification is highest at the crest of slopes and this is evidenced by observations of failures in past earthquakes (Brabhaharan et al., 2018).
- Amplification is greatest at the surface but decreases with depth into the slope. As such, shallow sliding is most affected by topographic amplification.
- In natural slopes that often comprise surficial soil or colluvium over more competent rock, this looser surficial material increases the amplification effects (CEN, 2004); (Brabhaharan et al., 2018). It is not clear if this is a topographic effect or amplification due to the lower stiffness of the shallower soils, or both.

A straightforward way to address the complex process of topographic amplification is to multiply the horizontal seismic loading by a topographical amplification factor (TAF). The following references provide advice for estimating the TAF:

- Brabhaharan et al (2018) (integrated into NZTA/Waka Kotahi Bridge Manual, 2022) provides factors for analysis and design of high cut slopes for transportation projects.
- Module 6, which is adapted slightly from recommendations in EC8 (European Committee for Standardization 2004), provides factors for analysis of retaining walls.

The advice in Brabhaharan et al (2018) can be used to estimate topographic amplification for assessment of unsupported cut slopes in natural materials, and this advice is summarised in Table 15. The advice in Module 6 can be used to assess topographic amplification for shallow sliding elsewhere.

17.3 DYNAMIC MATERIAL SHEAR STRENGTHS

Material shear strength during an earthquake can be affected by the displacement (Figure 64) and magnitude of seismic loading. As such, the dynamic shear strength selected for analysis should account for these factors as discussed below.

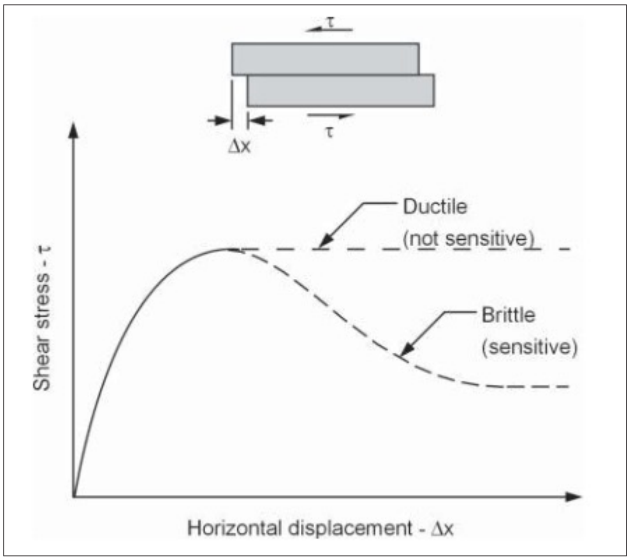


FIGURE 64: Strength can reduce with increasing displacement (Duncan, 2014)

17.3.1 Uncemented Coarse-Grained Soils (Sands and Gravels)

The dynamic shear strength of sands and gravels depends primarily on the potential for pore water pressure build-up or full liquefaction as outlined below and illustrated in Figure 65. See Module 3 for guidance on liquefaction assessment.

- No Liquefaction (factor of safety against liquefaction (FoSliq) > 1.4) - The dynamic strength of granular soils can be represented by effective stress drained strength parameters.
- Partial Pressure Build-Up (FoSliq = 1.1 to 1.4) – shear strength is reduced due to a decrease in effective stress from pore water pressure build-up. Figure 65 can be used to estimate excess pore water pressure and reduced effective stress. Excess pore pressure can be accounted for by reducing the friction angle as follows:

  $$\phi_{reduced}=\tan^{-1}((1-R_u)*\tan(\phi'))$$ **Equation 27**

  where:

  $\phi_{reduced}$ = reduced friction angle to account for excess pore pressure build-up

  R_u = $\Delta u/\sigma'$ = residual excess pore pressure ratio (Figure 65)

  ϕ' = effective stress friction angle with no excess pore pressure.

  Research on pore pressure build-up during seismic shaking is ongoing and additional methods for estimating excess pore pressures are likely to be developed.
- Liquefaction (FoS_{liq} < 1.1) – Use liquefied residual undrained shear strengths. Residual strengths of liquefied soils and slightly older methods for their estimation (Idriss & Boulanger, 2008; Olson & Stark, 2002; R. B. Seed & Harder, 1990) are discussed in Module 3. Robertson (2021), Kramer & Wang (2015), and (Weber, 2015) provide updated procedures that use the concept of a nonlinear, stress-dependent relationship between penetration resistance and residual strength. Owing to the substantial uncertainties in these correlations, the use of multiple correlations is warranted (National Academies of Sciences, 2021). We recommend that at least two of the more recent methods are used and that the method that produces the lower estimates be selected for design. Alternatively, evaluate the sensitivity of the results to the likely range of residual strength and account for the outcome of such a sensitivity study in the interpretation of results and decision-making process.

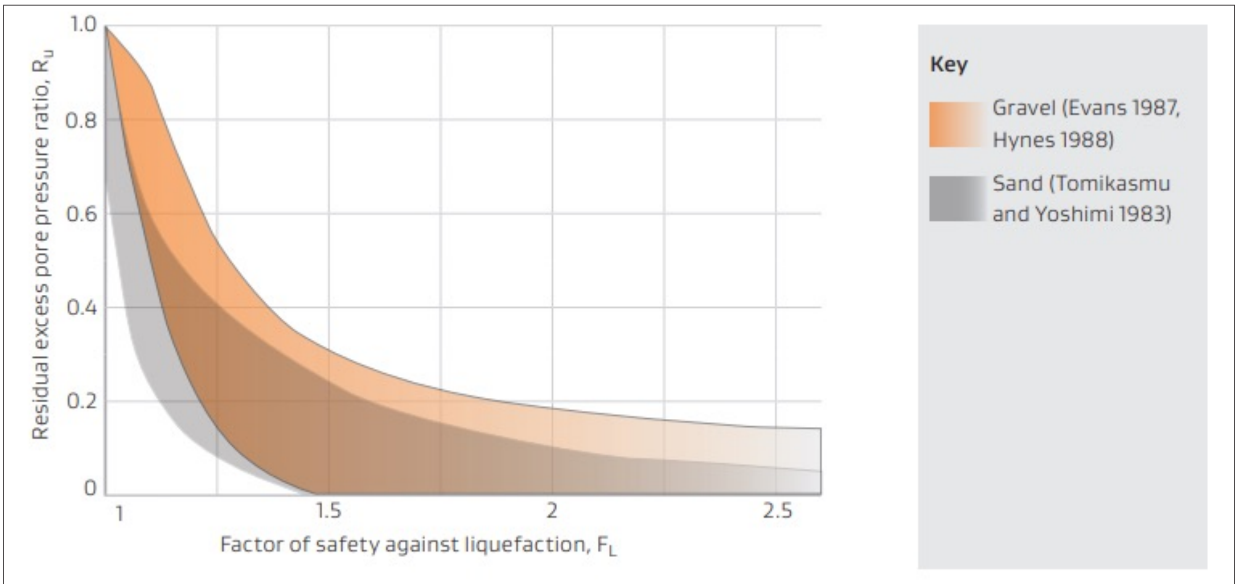


FIGURE 65: Excess pore pressure generation vs Liquefaction FoS (Marcuson et al 1990).

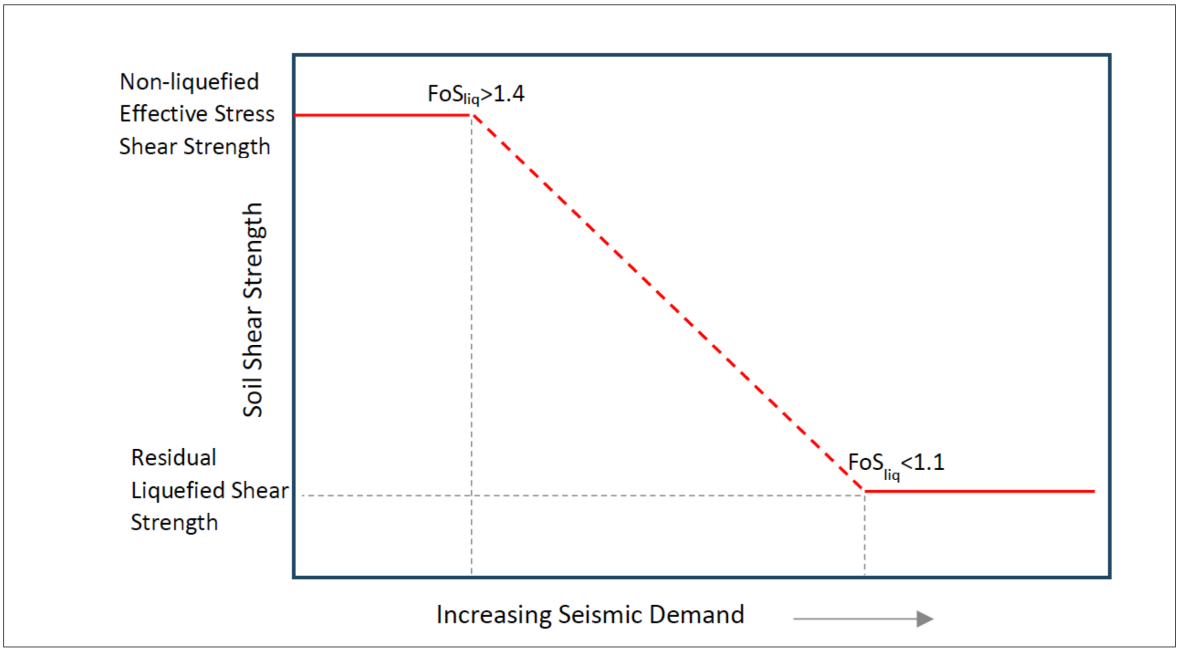


FIGURE 66: Effect of seismic shaking on soil strength in saturated granular soils

At low confining pressures non-liquefied dense sands may be dilatant and exhibit strain-softening behaviour. If large displacements are anticipated, strengths associated with that deformation should be used.

17.3.2 Cohesive Soils

Due to the rapid loading during earthquake shaking undrained shear strength parameters should be used for seismic stability assessment in cohesive soils. The shear strength selected for analysis for cohesive soils depends on the following:

- The dynamic effects on the strength:

  Chen et al (2006) provide 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 (M_w >~7.5) resulting in a decreased dynamic strength.

- The strain effects on strength:

  Most clays exhibit some strain-softening behaviour, and the shear strength selected for analysis should be consistent with the anticipated deformation. Where large deformations are expected residual dynamic strengths may be appropriate.

  Dynamic residual shear strengths can be estimated as the static residual shear strength.

17.3.3 Brittle or Sensitive Materials

Brittle or sensitive materials such as highly fractured/jointed rock (see also "Intact Rock" in Section 3), heavily overconsolidated clays (OCR>4), very soft sensitive clays, or cemented granular soils can experience significant strength loss at small deformations. For seismic stability analysis in these materials, the geoprofessional can either:

- Use peak material strengths and avoid deformation or limit it to small values (see Section 17.7 for discussion of deformation), or
- Use residual strength values in the analysis.

17.4 POST-EARTHQUAKE STABILITY ANALYSIS

The geoprofessional should evaluate the liquefaction potential of soils within or below the slope (see Module 3 for guidance). It's rare to find liquefaction occurring within or beneath natural slopes, except in cases of alluvial terraces/banks or where the buttressing soils at the base of the slope may undergo liquefaction. However, liquefaction of soils beneath constructed slopes in alluvial depositional environments is more common. If liquefiable soils are present, 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 67.

17.5 PSEUDO-STATIC ANALYSIS

Where significant 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, and is assumed to act through the centre of gravity of the soil mass (or slice) (Figure 68). The seismic coefficient is integrated into commercial slope stability software for use in conventional equilibrium analyses.

The seismic coefficient, ks should be selected based on the tolerable displacement for the given earthquake event and may also depend on the anticipated failure surface.

Older methods provided ks values as a ratio of PGA for a specified minimum FoS and displacement (Hynes-Griffin & Franklin, 1984; Kavazanjian et al., 1997; H. B. Seed, 1979). The PGA ratios for these older methods range from around 0.13 to 0.5 for tolerable displacements of around one metre. For many engineering applications this level of deformation is not acceptable. More recent methods are based on experience and results from deformation analyses and correspond to lower displacement thresholds (FHWA, 2011), or allow the geoprofessional to select the threshold displacement (Bray & Macedo, 2019).

Table 15 outlines methods that can be used to estimate ks. General steps for completing the assessment are shown in Figure 71.

Estimate dynamic soil strengths as discussed in Section 17.3. Use residual liquefied undrained shear strength associated with those layers susceptible to liquefaction.

↓

Perform stability analyses (typically limit equilibrium methods) 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 (see Module 3)

↓

Where the FoS is greater than the target, it may be desirable to estimate slope displacements as discussed in Section 17.6. However, simplified seismic slope displacement procedures should be applied with caution for cases involving soil that liquefies (See Newmark Methods and Liquefaction in Section 17.6.3).

FIGURE 67: Post-seismic analysis procedure

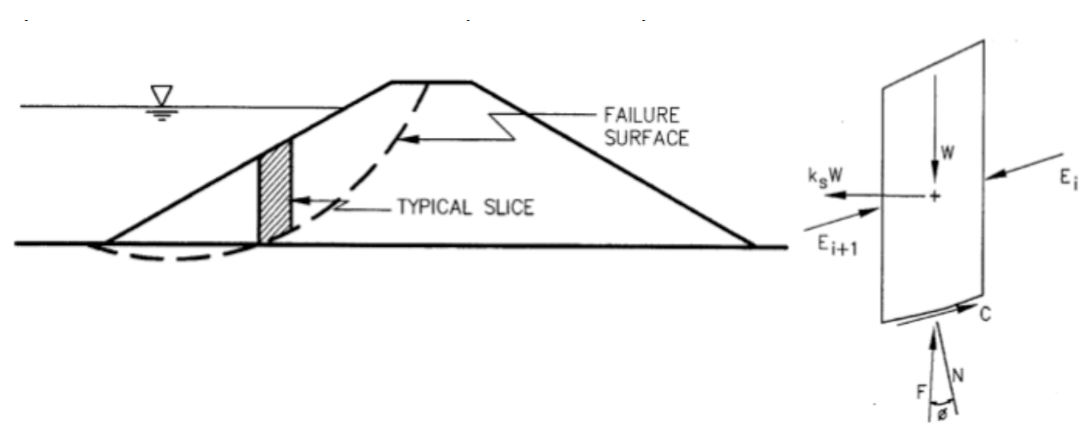


FIGURE 68: Earthquake loading representation in pseudo-static analysis (FHWA, 2011).

Table 15: Methods for estimating ks for Pseudo-static analysis

Method/Source	Comments
Bray and Macedo (2019) for Crustal Zone Earthquakes, Macedo et al (2023) for Subduction Zone Earthquakes **(Updates and supersedes Bray & Travasarou, 2009)**	These methods provide ks values that are consistent with the deformation-based methods introduced in the same papers. A method for crustal zone earthquakes is provided in Bray and Macedo (2019), and methods specific to subduction zone earthquakes are outlined in Macedo et al (2023). Notes on these procedures are outlined in Table 16. These methods account for the deformability of the potential slide mass and allows for the geoprofessional to select the threshold displacement value. Inputs also include the fundamental period of the slide mass (T_s) and the spectral acceleration at the degraded period of the slide mass ($Sa(1.3T_s)$). The input ground motion (i.e. spectral acceleration) is taken at the base of the slide mass assuming no material above. A FoS of 1.0 should be targeted with this method. Spreadsheet implementation of these methods can be found on Bray's webpage (Jonathan D. Bray | Civil and Environmental Engineering (berkeley.edu)).
Module 6	Module 6 introduces a wall displacement factor W_d, that is used to reduce the PGA (adjusted for topographic effects) to provide a ks. The level of reduction depends on the sensitivity of the situation to movement of a retaining structure with factors provided for six different scenarios. While the W_d factor is not directly correlated to a specific deformation, it is expected that deformations do not exceed threshold movements outlined in Table 4.1 of Module 6. Deformations associated with W_d factors are summarised below. W_d — Displacement 1 (i.e. ks=PGA) — negligible 0.7 — <50 mm — ks = PGA (adjusted for topographic effects)*W_d 0.5 — <100 mm 0.3 — <150 mm This approach to determining ks is appropriate for global stability checks related to retaining wall design for walls that fit into the defined cases, and where the level of deformation does not need to be determined explicitly. A target FoS of 1.2 is recommended in Module 6.
FHWA (2011) / Anderson et al. (2008)	The pseudo-static method outlined in FHWA (2011) is based on the procedure introduced in Anderson et al (2008) and as such is subject to the same limitations which are outlined in Table 16 (i.e. limited range of slope heights and only applicable to soil slopes). This method accounts for incoherence of the ground motion within the slope and provides ks values for negligible displacement and 50 mm of displacement. The input PGA is the maximum acceleration at the original ground surface beneath a fill slope or at the base of the natural slope. ks = α * PGA *Displacement Factor, where α = slope height reduction factor (1 for slopes less than 6m high) Displacement Factor = 0.5 where 50 mm of displacement is permitted, and 1 where negligible displacement is required. A factor of 1.2 should be applied to ks for rock sites (Site Class A or B). A FoS of 1.1 should be targeted with this method. Mostly, methods newer than this are preferred.
Brabhaharan et al. (2018)	This report recommends ks values for assessment of cut slopes for transportation projects. The values of ks vary depending on the scale and location of failure mechanisms of the cut slope. The recommended ks values assume negligible displacement is acceptable, and while not explicitly stated by the methodology, it is inferred that performance is acceptable where the FoS is greater than one. Failure in the upper quarter of the slope: ks = PGA* TAF Failure in the upper half of the slope: ks = PGA Failure of the full slope: ks = 0.65 * PGA Where TAF = Topographic Amplification Factor as recommended in Brabhaharan et al. (2018). This method is recommended for cut slopes in natural materials for transport projects. As there is not an allowance for deformation other methods are preferred where some deformation can be accommodated.

ASPECTS OF THE SEISMIC COEFFICIENT, k_s

- ks is not the PGA. It is typically less than the PGA to account for incoherence of the motion in the sliding mass and the allowance of some deformation. It is noted that in this context the PGA is assumed to have been adjusted for topographic effects where appropriate.
- The value of 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 69.
- In most cases, some deformation following an earthquake is tolerable and selection of ks should account for this.
- The average Maximum 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 (Figure 70). 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.
- Vertical acceleration is typically ignored.

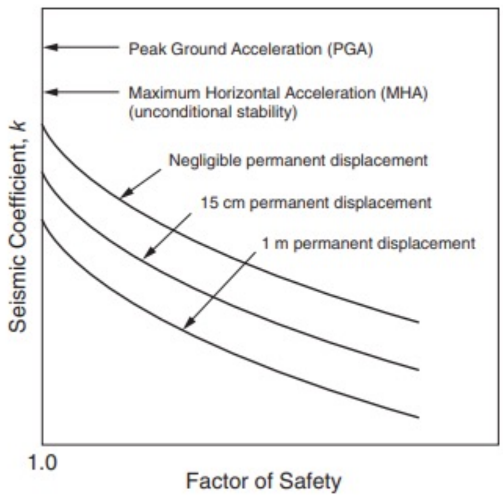


FIGURE 69: Relationship between seismic coefficient, deformation, and FoS (Duncan et al, 2014 after Kavazanjian, 2013)

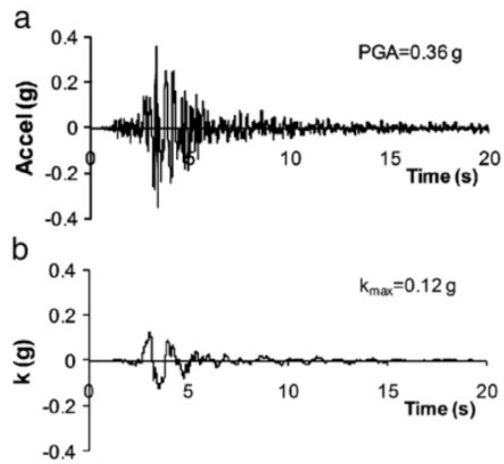


FIGURE 70: (a) Acceleration time-history and (b) ks-time history for a flexible sliding mass (Rathje & Antonakos, 2011)

Determine the seismic demand parameters for the return period of interest as outlined in Table 14

↓

If soils can lose significant strength at the design level of shaking, first carry out a post-seismic check as detailed in Section 17.4. If an adequate FoS exists in this case, continue to next step.

↓

Determine dynamic soil strengths compatible with the anticipated deformations (Section 17.3).

↓

Determine the allowable threshold displacement – based on input from stakeholders/design team. Displacements are discussed in Section 17.7.

↓

Calculate the seismic coefficient, ks, for the specified displacement in accordance with the selected method(s) (Section 17.5).

↓

Perform the stability analysis (limit equilibrium methods) with the calculated ks. If FoS > 1 (or the minimum value stipulated by the method), then displacements are assessed to be less than the selected threshold. If FoS < 1, displacements will be greater and deformation analysis (Section 17.6) should be undertaken.

FIGURE 71: Pseudo-static seismic slope stability screening procedure

17.6 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. A more comprehensive discussion on interpretation of displacement estimates is provided in Section 17.7.
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, when applied as a horizontal acceleration, results in a FoS = 1. The method calculates total displacement by double integrating portions of the earthquake record where acceleration exceeds the critical value. The Newmark sliding block method is illustrated in Figure 72.

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 spectrum or from synthetic records. 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 empirical procedures for estimating

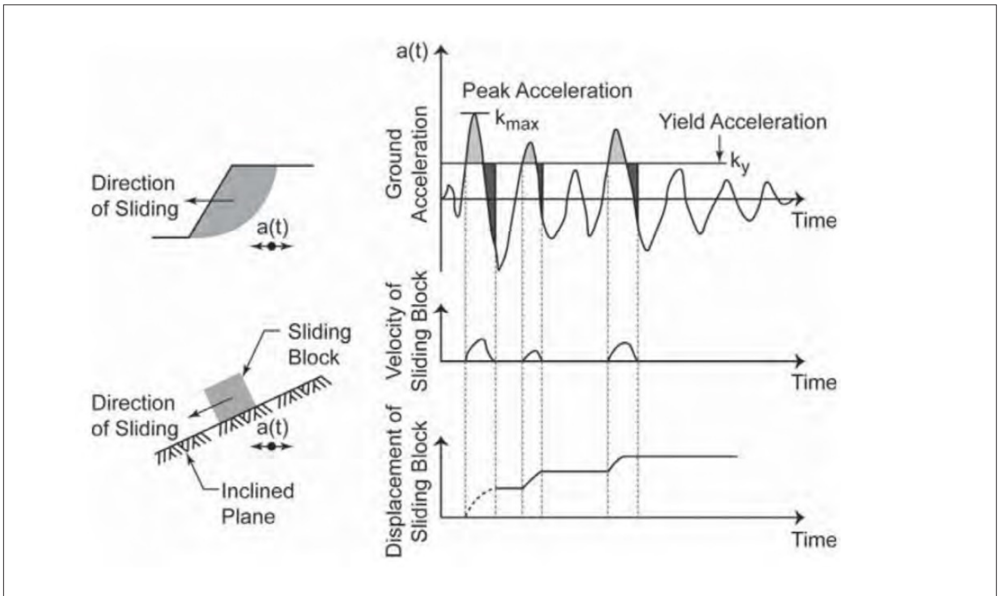


FIGURE 72: Illustration of the Newmark sliding block method (FHWA, 2011).

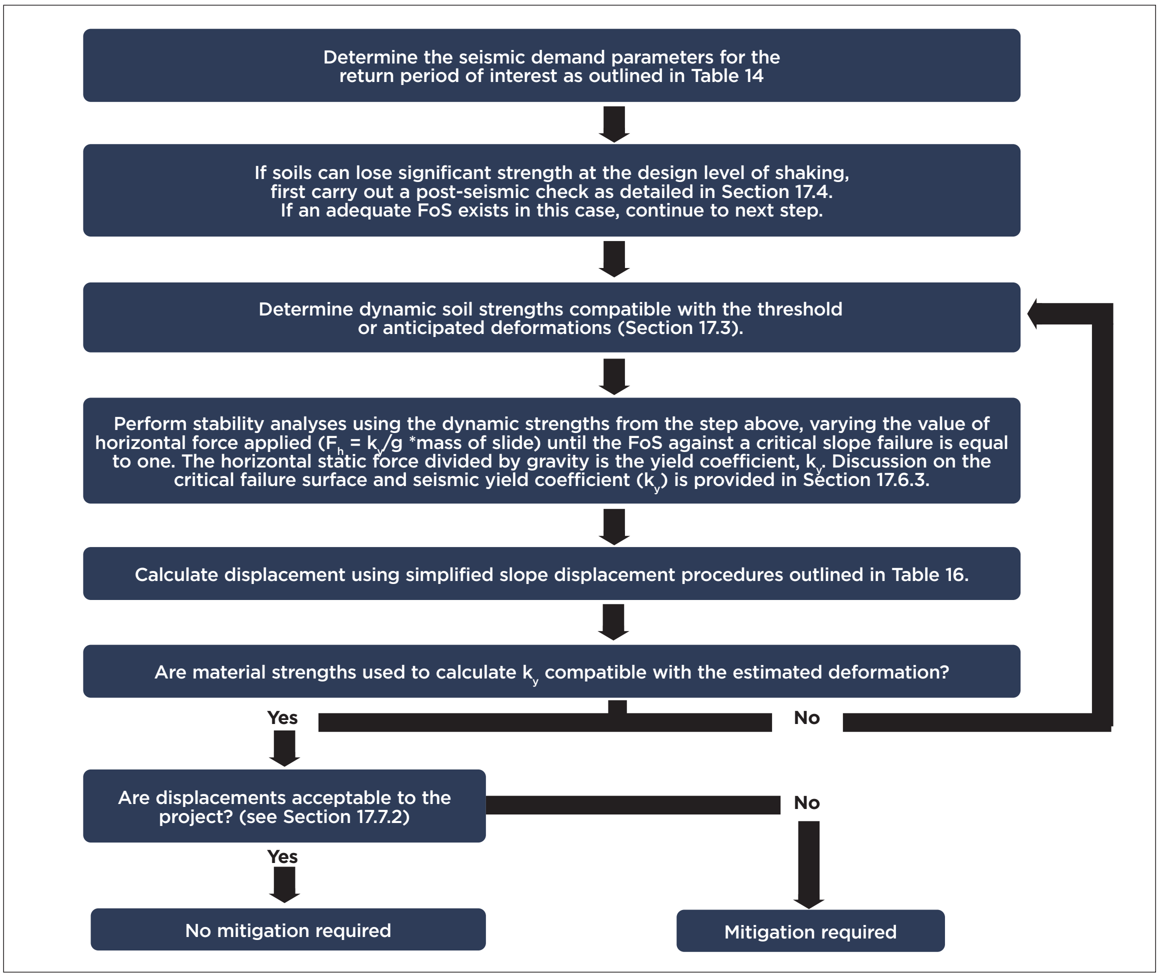


Figure 73: Procedure for Estimating Seismic Slope Displacements

seismic displacement. These procedures are discussed further below and the general process for estimating slope displacements using simplified methods is shown in Figure 73.

Newmark sliding block procedures calculate shear-induced seismic displacement, but do not capture volumetric compression of the slope due earthquake shaking and this aspect should be addressed separately. Seismic compression is discussed in Section 17.8.

17.6.1 Simplified Seismic Slope Displacement Procedures

Many simplified empirical seismic slope displacement procedures based on the original Newmark sliding block method have been developed over the past 50 years. Significant points of difference between these methods include:

- **The number of earthquake records from which empirical models are derived.**
  The early empirical models were based on analysis of very few records. Earlier models (e.g. Ambraseys & Menu, 1988; Ambraseys & Srbulov, 1995; Makdisi & Seed, 1978) have largely been superseded by more recent models that employ a much larger set of earthquake records and therefore can provide more robust predictions of displacement and uncertainty. These earlier methods are not recommended for use.
- **Whether the model assumes the sliding mass is rigid (rigid models) or accounts for the deformability of the sliding mass (flexible models).**
  Many empirical models employ the rigid assumption of the original Newmark method and assume that the sliding mass does not deform internally. This assumption is reasonable for thin slide masses of stiff materials. In these cases, the fundamental period of the slide mass is near zero and the dynamic response of the slide mass can be ignored (Rathje & Antonakos, 2011). Deeper slide masses of softer soils deform internally and modify the seismic loading on the slide mass. In these cases, the rigid assumption can be unconservative and rigid methods are not recommended.
- **Whether the models are based on coupled or decoupled analyses.**
  Incorporation of the deformability of the sliding mass into sliding block analyses for estimating displacement can be done using decoupled or coupled approaches. Figure 74 illustrates the various model approaches.

Decoupled Approach: The decoupled approach involves (1) computing the dynamic response of the sliding mass to develop an average[19] acceleration time history of the slide mass without consideration of the sliding displacement, then (2) using the resultant average acceleration time history as input in the rigid sliding block analysis to estimate displacement. Decoupling the slide mass response from the deformation analysis does not account for the effect of displacement on the dynamic response of the slide mass. The decoupled approximation is often judged to be reasonable given the large sources of uncertainty generally present in analysis (Rathje & Bray, 2000; Lin & Whitman, 1983), but can be conservative near resonance (Bray & Macedo, 2023; Jibson, 2011).
Coupled Approach: This is the most sophisticated sliding block analysis (Jibson, 2011). The coupled approach simultaneously computes the dynamic and sliding responses where the seismic coefficient is limited by the yield coefficient and the dynamic equations of equilibrium change during sliding to satisfy this constraint (Rathje & Antonakos, 2011). Simplified empirical models have been developed based on fully coupled sliding block analysis by Bray and Travasarou (2007), and most recently by Bray & Macedo (2019) and Macedo et al. (2023).

[19] Averaged in space across the slide mass. This is to reflect the observation that acceleration time-history will not be the same for every soil particle in the slide.

- **Tectonic Setting.**
  Until recently empirical sliding block procedures were based predominantly on shallow crustal earthquake records. Bray et al. (2018) introduced a procedure for subduction zone interface earthquakes which was updated by Macedo et al. (2023) to incorporate a larger ground motion database and to include intraslab earthquakes.

17.6.2 Selecting Appropriate Simplified Seismic Slope Displacement Procedures

A selection of more recently derived and commonly employed simplified displacement models are summarised in Table 16. We anticipate that new methods will be developed which will, in time, supersede those presented here.

Two to three procedures should ideally be used to estimate deformations to get a sense of the uncertainty in the estimates. Section 17.7.3 discusses the selection of displacement estimates.

The choice of which methods to employ of those outlined in Table 16 depends predominantly on the validity of the rigid assumption and the tectonic setting as discussed below and summarised in Figure 75. In general, the more recent methods outlined in Table 16 are preferred over the older methods as they are based on more earthquake records.

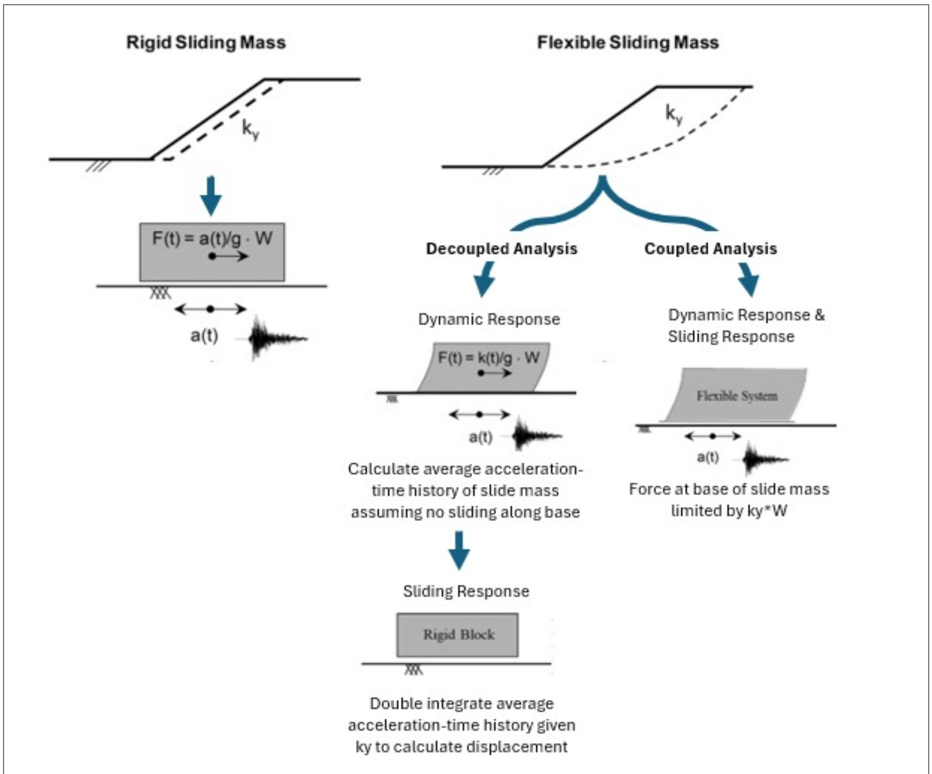


FIGURE 74: Flexible/Rigid and Coupled/Decoupled Sliding Block Displacement Approaches (adapted from Rathje and Antonakos, 2011; Bray, 2007)

Rigid vs Flexible Slide Mass

Rigid sliding block procedures should only be used where the slide mass can be considered rigid. This may be the case for:

- Shallow/thin sliding of stiff materials. This is often the case in natural slopes Jibson (2011).
- Very short slopes/retaining walls (on the order of a few metres).

For deep sliding in softer materials, methods that consider the flexibility of the slide mass should be used.

Jibson (2011) provides some advice on evaluating the validity of the rigid assumption through the ratio of Ts/Tm where Ts is the fundamental period of the slide mass (typically taken as 4H/Vs, where H is the slide mass height and Vs is the time averaged shear wave velocity of the soil in the slide mass), and Tm is the mean period of the earthquake motion which can be estimated using relationships presented in Rathje et al. (2004). Jibson (2011) indicates that where Ts/Tm is less than or equal to 0.1, the rigid assumption is valid.

The spreadsheet of the Bray & Macedo (2019) method suggests that if Ts < 0.05 s, then the slide mass should be considered to be rigid. Depending on the stiffness of the soil in the slide mass, Ts of 0.05 s corresponds to a slide mass thickness of around 2 m or 3 m.

Tectonic Setting

With the recent introduction of Bray et al. (2018) and Macedo et al. (2023)), simplified seismic displacement procedures developed using ground motions from subduction zone earthquakes are available. These models are expected to better reflect performance of slopes in subduction zone events (Bray et al 2018). For some regions in New Zealand, the seismic hazard is dominated by subduction zone events. For example, in Wellington, subduction zone events contribute approximately 80% to the overall hazard for 500-year earthquake. For these locations, the Macedo et al. (2023) methods are likely more suitable than the shallow crustal models. The NSHM webtool provides information on the tectonic sources that contribute to the seismic hazard for a specified location and return period.

The seismic hazard curves and uniform hazard spectra obtained from the NSHM webtool and TS1170.5 represent the aggregated (combined) hazard from all the contributing tectonic sources, both shallow crustal and subduction. This poses some difficulty as the inputs for the subduction zone models should represent the disaggregated seismic demand from the relevant tectonic sources. However, given the large uncertainty involved in estimating seismic displacement, there remains value in estimates of displacement from tectonic source-specific models using the aggregated seismic demand parameters. Where, for example, subduction interface earthquakes contribute most to the hazard, displacement estimates from this model can be given more weight than displacements from the intraslab and shallow crustal models. Where crustal earthquakes make up most of the hazard, as is the case for Auckland and Christchurch, only the shallow crustal models need be used. In any case, a comparison of results in Bray et al. (2018) indicated the Bray & Travasarou (2007) shallow crustal model provided reasonable estimates for large magnitude subduction zone interface earthquakes, and conservative estimates for lower magnitudes. As such, there is likely still value in shallow crustal models in regions where subduction zone events dominate the hazard.

17.6.3 Estimating the Seismic Yield Coefficient

The seismic yield coefficient, k_y, represents the resistance to sliding in simplified seismic slope displacement procedures, and is therefore a key input. The yield coefficient depends on the specific failure surface being assessed. There are three surfaces that may be considered:

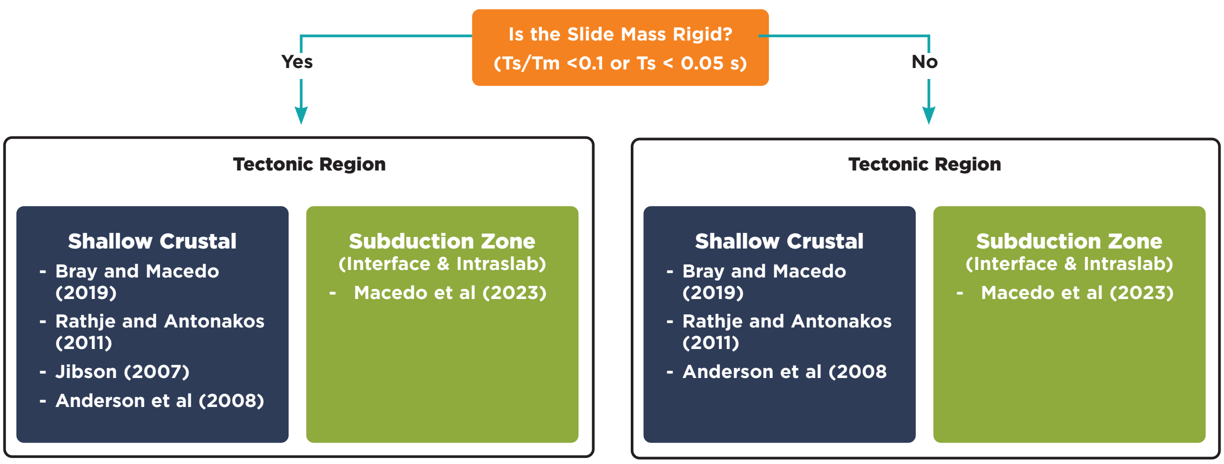


FIGURE 75: Summary of simplified seismic displacement methods (most preferred methods at top of lists)

1. The surface with the lowest static FoS – this is often not the surface that produces the most critical yield acceleration
2. The surface that produces the minimum k_y – this surface can become unrealistically deep and long. Some advice for managing this tendency is outlined below.
3. The surface that produces the lowest k_y/MHA – this is often the key failure surface as it represents the lowest ratio of the slope's sliding resistance to the seismic demand. In methods that consider flexible slide masses, the seismic demand on the slope depends on the particular surface being assessed. In the Rathje & Antonakos (2011) and Anderson et al. (2008) methods, the seismic demand reduces with increased depth of the sliding surface. In the Bray & Macedo (2019) and Macedo et al. (2023) methods, the seismic demands increase with slide mass height as the degraded period of the slide mass approaches resonance with the underlying ground motion.

Where the three surfaces above correspond to a shallow failure, the geoprofessional may be more interested in a deeper failure surface that, while having a higher k_y or k_y/MHA, is more critical as failure would result in higher consequence (i.e. where a deeper failure surface intersects an asset).

In these cases, the geoprofessional may be interested in multiple failure surfaces and the yield coefficient corresponding to each should be used to estimate displacement.

Ways of managing unrealistically deep/long yield coefficient failure surfaces include:

(1) Focussing on correctly modelling the changes in ground conditions laterally and with depth. It is common to simplify the modelled ground conditions by assuming soil stratigraphy extends laterally away from the slope. Soil layers are rarely horizontal and these changes in ground conditions laterally can limit the size of the sliding mass. Similarly, it is often assumed that uniform conditions extend below the depth of investigation however, in many cases, slope material increases in strength with depth.

(2) Assessing the geological admissibility of the yield failure surface from limit equilibrium analysis. Where results indicate a yield coefficient failure surface that does not match our expectations the ground model should be re-examined. For instance, a deep yield surface in natural ground with an average failure surface angle much less than the slope angle of similar nearby slopes is unlikely move substantially in a future earthquake, as it probably hasn't in a past earthquake.

[20] In this context, "index" means "an indication" or "an idea".

NEWMARK METHODS, LIQUEFACTION AND LATERAL SPREADING

There are many situations in which Newmark procedures can provide a useful indication of seismic slope performance where liquefiable soils are present. This tends to be in cases where liquefiable layers are deep, and/or thin and discontinuous and are not the main driver of instability (unlike in lateral spreading described below). Guidance in this section is applicable to these scenarios. However, it is important to appreciate that the underlying assumptions for Newmark procedures may not be compatible with the distributed deformation of liquefied soils (Module 3). The practitioner should acknowledge the large uncertainties that may exist in estimated deformations when using these procedures with liquefiable soils. Lateral spreading is a specific type of seismic slope instability resulting from soil strength loss due to liquefaction. It commonly occurs near riverbanks and shorelines where shallow liquefiable soils are present. Module 3 provides guidance for assessment of lateral spreading type movement. Where the post seismic FoS < 1 and in the situations where liquefaction-induced lateral spreading is expected to occur, qualitative and empirical lateral spreading methods outlined in Module 3 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 not appropriate in these cases.

17.7 INTERPRETING ESTIMATED DISPLACEMENTS

Displacement estimates from sliding block methods provide an index[20] of seismic slope performance, and when viewed in this way, estimates have correlated well with observations of performance (Jibson et al., 2000; Jibson, 2011). However, estimates do not necessarily correspond with measured displacements in the field and should not be expected to provide accurate measures of displacement. There are issues to address when interpreting estimated displacements:

1. will the estimated level of displacement cause further destabilisation and catastrophic ground failure?
2. if the answer to (1) is "no", then is the level of deformation tolerable/acceptable?
3. what is the uncertainty in the estimate?

17.7.1 Displacement's Effect on Stability

Of particular importance is understanding the levels of predicted displacement which can lead to significant slope destabilisation. Table 17 provides a summary of some published efforts at relating sliding block displacement estimates with slope performance.

Table 16: Simplified Seismic Slope Displacement Procedures

Model/ Method	Tectonic Setting	Number of Earthquake Records	Sliding Mass Deformability and Calculation Type	Seismic Demand Parameters	Slide Mass Parameters	Comments
Jibson, (2007)	Shallow Crustal	875	Rigid	PGA, M	Not considered	- A widely used empirical model that employs the rigid sliding block assumption - Easy to implement as seismic demand is represented by the easily derived parameters, PGA and M. - Jibson notes that his method is best suited to earthquake triggered landslides in natural slopes. These failures are mostly shallow, disrupted failures in brittle materials where the rigid sliding block assumption is reasonable. - Jibson indicates that *"...the intended use of these equations is for regional-scale assessment and mapping of seismic landslide hazards..."* or to quickly screen individual sites to evaluate what additional, more detailed, studies are required. In our experience, this method is widely applied outside of the intended use stated by Jibson. Provided this method is (1) applied to natural slopes for which the rigid assumption is valid, (2) other methods are utilised in tandem, and (3) displacements are interpreted as indices of performance as discussed in Section 17.6, this method can provide useful insights into seismic stability.
Anderson et al. (2008)	Shallow Crustal	1,800	Flexible and Rigid Decoupled	PGV, PGA, Sa(1 s)	Slope height	- This method was developed to provide a simplified approach for seismic evaluation of retaining walls and cut and fill slopes in the United States. - Wave scattering and incoherence effects are incorporated through use of a slope-height dependant reduction factor (α) applied to the PGA to represent the average peak acceleration within the slide mass. - The input PGA is taken at the previous ground surface beneath a fill slope or at the base of a natural slope. - Reduction factors were developed based on dynamic analyses for a slope height of ~10m and retaining wall heights from ~6m to 36m. - It is recommended that the wave scattering reduction factor only be applied for slopes 6m or higher. - PGV is estimated based on a provided relationship between PGV and the spectral acceleration at one second (Sa(1 s)). - The method is intended for use in soil slopes (natural and constructed), not rock slopes. - This method was developed for a specific use, and studies carried out to understand wave scattering effects in slopes relative to those for retaining walls are limited (only one slope height was assessed). As such, we recommend estimates from other methods be weighted more heavily.

Table 16: Simplified Seismic Slope Displacement Procedures (continued)

Rathje & Antonakos (2011)[1]	Shallow Crustal	2,383 (400)[2]	Flexible and Rigid Decoupled	PGA, PGV, Tm	Ts	- The method accounts for a range of dynamic response conditions from rigid to flexible sliding mass behaviour through use of the fundamental period of the slide mass (Ts). - Two models are presented; one based on PGA and M and one on PGA and PGV. Rathje and Antonakos (2011) recommend the PGA/PGV model. - This method requires estimation of the mean period of the earthquake ground motion (Tm) which can estimated using relationships presented in Rathje et al. (2004), and PGV which can be estimated using methods outlined in Anderson et al. (2008) or through the NGA West-2 models as discussed in Table 14.
Bray & Macedo (2019) (Updates & supersedes Bray & Travasarou, 2007)	Shallow Crustal	6711	Flexible and Rigid Coupled	Sa(1.3Ts), M, PGV sometimes needed	Ts	- This method is an update to the widely-used Bray and Travasarou (2007) procedure and accounts for the deformability of the sliding block. - The seismic demand is represented by the spectral acceleration at the degraded period of the slide mass (1.3Ts) where the input ground motion is at the base of the slide mass. Where Ts<0.05 s, the slide mass can be assumed to be rigid (i.e. Ts=0 and Sa(1.3Ts)=PGA). - As the input ground motion is for the base of the slide mass, the spectral acceleration values should be derived from V_{s30} values or seismic site class that is representative of the subsurface <u>below</u> (not including) the slide mass. In natural slopes, this will likely result in higher V_{s30} values as inputs in determining the spectral acceleration. - Spreadsheet implementation of this method and the Macedo et al. (2023) method can be found on Bray's webpage. - Where sliding is shallow and the rigid assumption is valid, topographic amplification factors should be applied to the input seismic demand (PGA)
Macedo et al. (2023) (Updates and supersedes Bray et al., 2018)	Subduction Zone Interface and Intraslab	6240 (interface), 8299 (intraslab)	Flexible and Rigid Coupled	Sa(1.3Ts), M, (PGV optional)	Ts	- Deformations are estimated using the same approach as Bray & Macedo (2019) but Macedo et al. (2023) models were developed utilising subduction zone (interface and intraslab) earthquake records. Bray's website provides a spreadsheet implementation of this procedure.

[1] Builds on work presented in Saygili & Rathje (2008) and Rathje & Saygili (2009)

[2] Number of records used in Rathje & Saygili (2009), with additional records from Rathje & Antonakos (2011) in parentheses.

Table 17: Sliding block displacement thresholds

<table>
<tr><th>Source</th><th colspan="2">Displacement Levels and Commentary</th></tr>
<tr><td>Wieczorek et al. (1985)</td><td colspan="2">50 mm
Used as critical displacement leading to catastrophic failure for a landslide hazard map of San Mateo County, California.</td></tr>
<tr><td>Keefer & Wilson (1989)</td><td colspan="2">100 mm
Critical displacement for coherent landslides in southern California</td></tr>
<tr><td>Jibson & Keefer (1993)</td><td colspan="2">50 – 100mm
This study focused on relatively deep-seated landslides in soils along bluffs bordering the Mississippi alluvial plan in the south-eastern U.S. that the study concluded likely occurred because of earthquakes. The following commentary is provided. "Laboratory shear- strength tests on samples from the Stewart and Campbell sites indicate that residual strength is reached after a total shear displacement of about 6 cm (Jibson, 1985); therefore, the 5-10 cm range is reasonable for these landslides. If this amount of displacement is exceeded, static factors of safety using residual shear strengths can be calculated to determine the stability of the landslide mass after the earthquake shaking (and consequent inertial land-slide displacement) ceases."</td></tr>
<tr><td rowspan="3">Bray & Rathje (1998)</td><td>< 25-50 mm: Small</td><td rowspan="3">These deformation categories are suggested for assessing the seismic performance of geosynthetic-lined solid-waste landfills.</td></tr>
<tr><td><150-300 mm: Moderate</td></tr>
<tr><td>>300-1000 mm: Large</td></tr>
<tr><td rowspan="3">Blake et al. (2002)</td><td><50 mm: Very Little</td><td rowspan="3">Thresholds pertain to a 10% in 50 yr PoE. The following commentary is provided:
- "For slip surfaces intersecting stiff improvements (such as buildings, pools, etc.), computed median displacements should be maintained at <5 cm
- For slip surfaces occurring in ductile (i.e., non strain softening) soil that do not intersect engineered improvements (e.g., landscaped areas and patios), computed median displacements should be maintained at < 15 cm.
- For slip surfaces occurring in soil with significant strain softening (i.e., sensitivity > 2), if ky was calculated from peak strengths, displacements as large as 15 cm could trigger strength reductions, which in turn could result in significant slope de-stabilization. For such cases, the design should either be performed using residual strengths (and maintaining displacements < 15 cm), or using peak strengths with displacements < 5 cm."</td></tr>
<tr><td>50-150 mm: Moderate</td></tr>
<tr><td>>150 mm: Large</td></tr>
<tr><td rowspan="2">Anderson et al. (2008)</td><td><100 mm: Stable</td><td rowspan="2">It is noted that these values are often considered as a general guide from a serviceability standpoint.</td></tr>
<tr><td>>300 mm: Unstable</td></tr>
</table>

Table 17: Sliding block displacement thresholds (continued)

California Geological Survey (2008)	<150 mm: Unlikely to correspond to serious landslide movement and damage	Jibson (2011) notes that these displacement thresholds pertain principally to deeper landslides; small, shallow landslides can be triggered at much lower displacement levels, around 20mm to 150mm (Jibson et al., 2000; Jibson, 2011).
	150 – 1000 mm: Could be serious enough to cause strength loss and continuing failure	
	>1000 mm: Very likely to correspond to damaging landslide movement	
Jibson & Michael (2009)	<10 mm: Low hazard category	These ranges were used to define hazard categories for shallow, disrupted coseismic landsliding for hazard mapping of Anchorage, Alaska.
	10-50 mm: Moderate hazard category	
	50-150 mm: High hazard category	
	>150 mm: Very high hazard category	
Massey et al. (2013)	100 mm – This displacement was used define areas of significant mass movement in the Port Hills following the 2010-2011 Canterbury Earthquake Sequence. GNS state "*This threshold was chosen because (1) it was an amount of displacement that could be measured with a reasonable level of accuracy in the field; and (2) it was an amount that had been used by others* [many cited in this table*] as a qualitative reflection of the impact that earthquakes would have on the stability of the slope.*"	
Engineers and Geoscientists British Columbia (2023)	<150 mm: Threshold of tolerable slope displacement when the sliding surface is between the building foundation perimeter and the face of the slope. The following commentary is provided "*The tolerable slope displacement of 15 cm is proposed as a guide, based on experience with residential wood-frame construction, and has been generally adopted in the industry. These guidelines are not intended to preclude Qualified Professionals from selecting another value that they deem appropriate.*"	

These past efforts highlight important considerations when developing performance categories based on displacement estimates, significantly (1) performance at a particular deformation depends on the slope materials' tendency for strength loss with strain (strain-softening behaviour) and the selected strength used in analysis (peak vs residual), and (2) significant shallow slope movements can be triggered at lower displacement levels than deep sliding.

Using the studies outlined in Table 17 we have developed the following categories of slope performance based on estimated sliding block displacements. The purpose of these categories is to broadly define their probable effect on a potential landslide. In other words, they address the question: "*Will the levels of deformation lead to strength loss and full mobilisation such that calculated sliding block deformations no longer reflect the slope performance?*" The seismic slope performance categories in Table 18 do not imply acceptable levels of displacement for a particular scenario. Threshold displacements are discussed in Section 17.7.2.

Table 18: Seismic Slope Performance Categories

Estimated displacement (mm)	Ductile material[1]		Brittle material[2]	
	Category	Comment	Category	Comment
0 - 10	Negligible Slope Movement	Adverse effects from seismic instability are unlikely.	Low Landslide Hazard	Instability is not anticipated
10 - 20			Low to Moderate Landslide Hazard	Low to moderate likelihood of landslide movement. Relates to a probability of failure of <5% based on Jibson et al. (2000)[3].
20 – 50	Minor Slope Movement	In this range minor signs of movement may occur. This level of movement is not expected to result in significant reductions in strength for sliding in most soils.	Moderate Landslide hazard	In this range, there is an ~5-15% probability of failure (Jibson et al., 2000)[3]. 50mm is a commonly adopted threshold for initiating slope failure, however smaller displacements could lead to strength drop to residual levels (Jibson, 2011)
50 – 150	Moderate Slope Movement	Deep seated slides are likely able to accommodate this level of movement without further mobilisation, but care should be taken in selecting soil strengths consistent with this level of deformation to calculate the yield coefficient. Geogrids may make the slope more tolerant of displacement.	High Landslide Hazard	It is plausible that evacuative failure could occur. There is a >33% probability of failure for displacements > 100 mm (Jibson et al., 2000).
150 – 300	Large Slope Movement		Very High Landslide Hazard	
300 – 1000	Major Slope Movement	In this range, slope movement is anticipated to cause damage to structures and infrastructure and may lead to strength losses that cause ongoing sliding. Deformations in this range will rarely be tolerable, and estimates should only be considered representative of potential performance where large strain shear strength in material along the failure surface is well defined and incorporated into the analysis.		
1000 +	Severe Slope Movement	Damaging landslide movement and significant slope destabilisation is likely.		

[1] This will typically apply for earth structures such as embankments and in soil slopes without brittle or sensitive soils. Where brittle soils (i.e. highly overconsolidated or cemented) or sensitive soils are present, the categories of slope movement are only applicable where strength consistent with anticipated deformation is selected.
[2] This will typically apply to shallow sliding in moderately steep (>35 degrees to the horizontal) to steep natural slopes or cut slopes in rock or colluvium.
[3] Developed for shallow disrupted slides that occurred in Southern California in the 1994 Northridge Earthquake.

17.7.2 Tolerable/Acceptable Seismic Slope Displacement/Performance

If estimated deformations are not expected to result in further slope destabilisation, the next question is whether that range of deformation is tolerable or acceptable for the affected assets. There is no one answer to this question as threshold displacements are a function of the ability of affected assets to tolerate displacement, and the performance of the asset required by stakeholders and the regulatory authority.

Different types of assets will have different minimum performance expectations at specified design limit state seismic return intervals (e.g. Serviceability Limit State, SLS for structures). An example of prescribed and inferred performance expectations and return periods for IL2 to IL4 structures is illustrated in Figure 76. Other assets, such as roads or dams will have different relationships between minimum performance expectation and earthquake return period depending on the relevant authority.

Project specific performance criteria may be more stringent than the minimum criteria outlined by the relevant authority and will depend on specific needs of the project. The geoprofessional should develop project specific criteria in conjunction with the project stakeholders. The seismic slope deformation limits adopted for the project, and the anticipated corresponding asset performance as a result of deformations should be clearly reported by the geoprofessional.

Guidance on required seismic slope performance by asset type includes:

- Roading
  - NZTA Waka Kotahi (2022)– This guidance provides threshold displacement values for bridges and embankments for highways and is often applied to local roads as well.
  - Brabhaharan et al (2018) – provides advice for seismic assessment of cut slopes for transport projects and assumes little to no displacement is allowed as cut slopes tend to be in brittle materials that are incapable of accommodating deformation without strength loss.
- Dams – NZSOLD (2023) provides high-level commentary on acceptable deformation. The seismic performance requirements of dams are outside the scope of this guidance and the discussion here is not intended to support dam design.
- Structures - There are no prescribed minimum displacement thresholds related to various performance scenarios (SLS, ILS, ULS) for structures due to seismic slope instability in New Zealand. This makes sense as different structures and building materials will have different levels of ductility and ability to tolerate permanent deformations. Some advice on structure response to deformation is given in the following references:
  - Module 6 provides advice on tolerable displacement at SLS and ULS for retaining walls related to IL1 to IL3 structures.

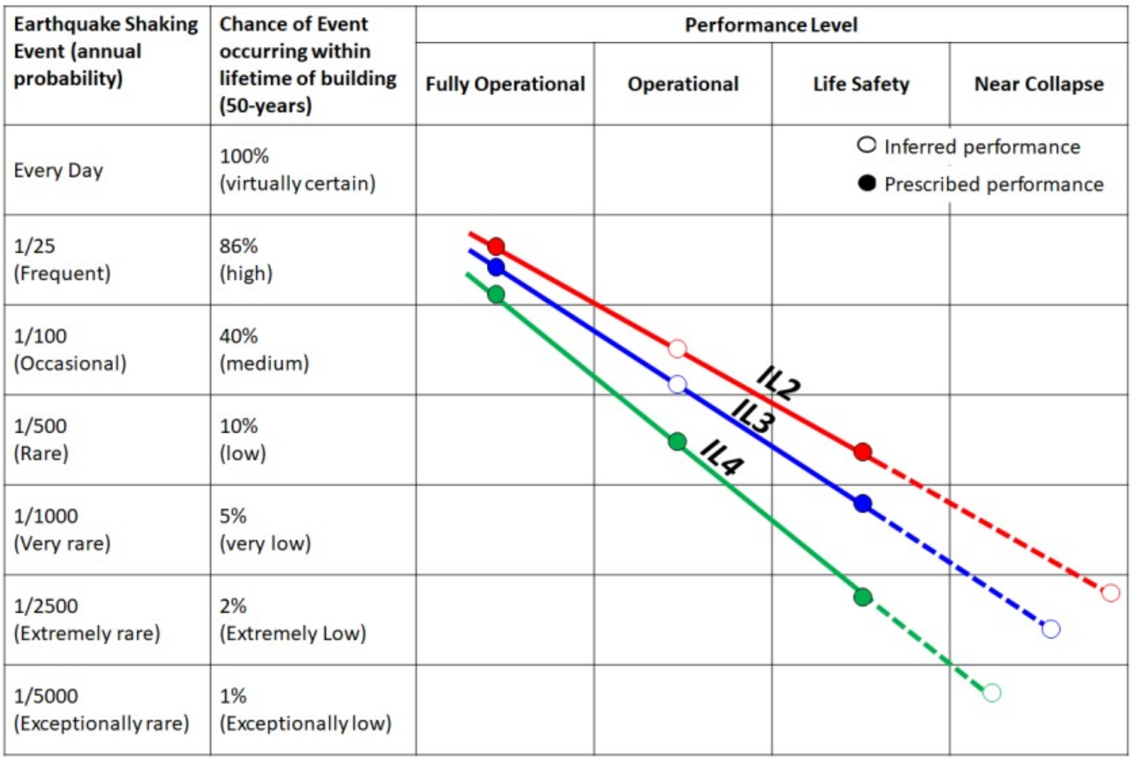


Figure 76: Seismic Performance Levels for IL 2-4 (NZS1170.0) from Taylor et al. (2023)

- Appendix B of B1/VM4 (MBIE, 2023) states that foundation design should limit the probable maximum differential settlement over a horizontal distance of 6 m to no more than 25 mm under SLS loads unless specifically designed to prevent damage under greater settlement.
- Loganathan (2011) provides a summary of damage categories for given levels of deformation (tensile strain, angular distortion, and settlement).

Building on this advice, Table 19 provides guidance for selecting deformation performance categories from Table 18 for timber-framed IL2 structures. The values are general guidance targeted towards timber-framed residential construction.

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Table 19: Acceptable deformation categories for timber-framed IL2 Structures

Performance Limit State	Ductile material	Brittle material
SLS	"Negligible Slope Movement" Category	No Movement
ULS	Each project will have specific requirements but for most IL2 structures at the head scarp of the anticipated slide plane or within the slide mass a threshold of "Minor to Moderate Slope Displacement" (i.e. 20 mm to 150 mm) may be allowable. Experience has shown that displacements related to slumping/compression at the toe of the slope commonly result in more damage to structures than in the tension zone. As such deformations limited to the "Minor Slope Movement" category (i.e. < 50 mm) may be more suitable for structures located at the toe of the slope.	Allowable deformations in these materials should generally be limited to small values. The "Low landslide hazard" category (i.e. <10mm) will typically be allowable. At higher deformations, it should be assumed that an evacuative failure is plausible.

Notes:
- Discussion on the slope categories (i.e. soil/constructed slopes and natural/cut slopes) is provided in the footnotes of Table 18.
- Project-specific criteria need to be developed in conjunction with the project stakeholders. The categories of slope movement outlined are general guidance and do not preclude selection of different values that reflect the project specific variables. In cases where structure foundations are sufficiently robust (e.g. rigid, mat slab foundations designed to cantilever) more deformation may be allowable for soil/constructed slopes. In other cases, more stringent criteria may be adopted. In all cases the reasons for the selection should be clearly communicated.
- Deformation performance categories referred to are described in Table 18.
- The deformation limit of 150 mm for ULS is intended to define a tolerable limit for typical timber-framed residential construction at which damage caused by the movement does not prevent safe egress or loss of structural integrity.

17.7.3 Considering Uncertainty and Selecting Design Displacement

Following seismic analysis, there may be many displacement estimates from which the geoprofessional can select to compare against the criteria outlined in Sections 17.7.1 and 17.7.2. Multiple estimates are a result of:

- use of multiple displacement methods (ideally three),
- accounting for the uncertainty within each method (i.e. 16th, 50th, or 84th percentile estimates), and
- assessing deformations for a range of k_y values which can indicate how sensitive the estimation of performance is to the ground model or reflect multiple failure surfaces of interest.

The choice of which estimate to select for interpretation and design can be difficult and should consider the uncertainty in the estimation of k_y, and the sensitivity of the slope to movement (i.e. consequence). For most routine projects the following approach is reasonable:

- Where two or more appropriate methods have been used, the highest of the median estimates can be selected, or
- Where one method has been used, the 84th percentile estimate (or one standard deviation above the mean) can be used.

It may also be appropriate to take an average of the median values from multiple methods for design, provided the implications of higher-than-expected displacements (say the upper bound estimates) are considered. This is called a scenario analysis. For example, where a new road embankment is constructed of ductile materials, and a median seismic displacement of 150 mm and upper bound value of 350mm are calculated, the median value may be suitable for design provided the slope and assets can tolerate 350 mm of movement without catastrophic failure. In situations where stability is sensitive to deformation (discussed in Section 17.7.1) it may be prudent to select the upper bound (or 84th percentile) value. These types of sensitivity checks are recommended as part of routine practice.

In all cases, the geoprofessional should clearly communicate methods used, and provide justification for the displacement estimates selected for design.

HIGH SEISMIC DEMAND AND APPLICABILITY OF SIMPLIFIED METHODS

The recent updates to the National Seismic Hazard Model (NSHM, 2022) have resulted in significant increases in the design seismic loading in parts of New Zealand. Increases in the seismic demand will increase the anticipated deformation and likelihood of a landslide. The industry is grappling with these changes and the impact they have on design. Considerations for the geoprofessional when confronted with high estimated deformations are:

1. **Ensure that results are consistent with observations of performance. If the slope of interest is an unaltered slope in an area of no observable signs of instability it is unlikely that an earthquake with a return period in the hundreds of years will produce deep instability with metres of deformation. Always assess the results of the slope stability analysis against expected performance and expected failure mechanism. If these are inconsistent, either indicating more or less stability than expected revisit and refine your ground model.**
2. **Reduce conservatism in the ground model through additional investigation. One way to refine the ground model used in analysis is through additional investigation. It is common to make simplifying assumptions about the material strength and distribution within a slope based on limited data. These assumptions are often conservative to account for the uncertainty. Additional investigation and testing to refine material shear strength and material distribution can reduce the level of conservatism in your model and produce more realistic analysis results.**
3. **Undertake more complex analysis which can better reflect the response of the site to seismic loading. Slope response to earthquake shaking is a highly complex process and this guidance has described the simplifying assumptions in common assessment methods. Simplified methods may miss critical dynamic response and behaviour, particularly at very high seismic demands, for which there are fewer case studies to validate these methods. Methods that account for the complexity in seismic demand, the dynamic behaviour of soil/rock, and the soil-structure interaction may better capture the performance of the site. It is important to keep in mind that results of more complex analysis are only of value where the inputs to the analysis reflect reality, and these assessments need to be accompanied by a robust investigation and laboratory testing programme.**
4. **Reconsider the required performance (i.e. threshold displacements). In cases where the slope is expected to accommodate deformation without catastrophic failure and strain-compatible material shear strengths are used in the analysis, it may be possible to allow additional deformation by either accepting lesser performance or designing assets affected by the slope to tolerate additional movement while still meeting performance criteria.**

Even considering these items, there will be situations where more robust solutions are required than would have been required under the previous design seismic loading. It is important to remember that the updated loadings reflect our best and current understanding of the seismic hazard in New Zealand. Ultimately this will allow us to better achieve a level of seismic resilience in our communities that is commensurate with the hazard. It is our responsibility as geoprofessionals to effectively communicate these changes and the reasons for them to our clients and other project stakeholders.

17.8 SEISMIC COMPRESSION

Newmark sliding block procedures calculate shear-induced seismic displacement, but do not capture volumetric compression of the slope. Deformation due to seismic compressions should be addressed separately.

Seismic compression is the settlement of unsaturated soil during earthquake shaking, particularly loose sands and poorly compacted fill (Figure 77). Slope deformation of hillside fills due to seismic compression has been documented in California (Stewart et al., 2001). It also occurred in the 2016 Kaikoura Earthquake, about which Mason et al. (2023) note:

"Fill embankments on level ground experienced widespread consolidation ('seismic compression' of Stewart et al., 2004) leading to settlement and cracking of the road pavements, and subsidence of the embankments and vertical deformation of the tracks along the railway corridor. Fill embankments on sloping ground suffered the most extensive and severe damage. The predominant modes of failure were consolidation/compression, slumping of the edges of the fill slopes, and displacement of the embankment and underlying natural soils."

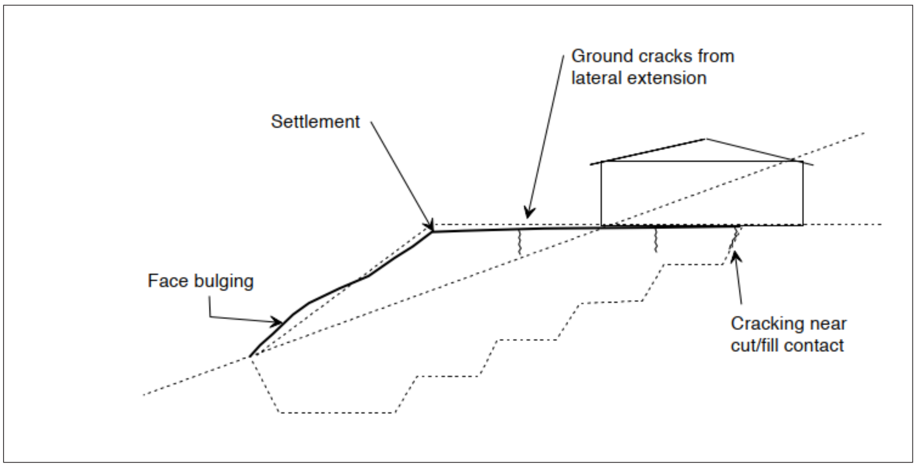

FIGURE 77: Schematic showing typical damage of seismic compression to fill slope (Stewart et al., 2001)

Seismic compression may be a significant mechanism of damage-inducing deformation in situations where the slope is comprised of loose sandy soils and/or fill. Structures or improvements that span the cut to fill boundary are most at risk of damage due to seismic compression. In these scenarios the potential for seismic compression should be assessed. Stewart et al. (2004) presents a simplified procedure for evaluating seismic compression susceptibility and magnitude.

17.9 NUMERICAL 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, PGA, and magnitude from 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.

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. Therefore, results of complex numerical analysis should always be checked against results from simpler analyses and observations of slope performance.

18 BACK-ANALYSIS

Back-analysis of a slope failure involves retroactively analysing the failure by conducting slope stability analysis using the estimated original ground surface and adjusting input parameters (typically strength parameters) until a FoS ≤ 1 is achieved on the surface that failed. 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 greater than 1. It is generally accepted that some incipient slope movement typically occurs between a FoS of 1.0 and 1.1. On this basis, if the slope has remained stable for many years without failure, then a minimum FoS of 1.1 over the range of conditions it has been subject to in that time (high groundwater, earthquakes, surcharge loading etc.) is suitable.

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 (see Wesley, 2010b, for examples). 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 important 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. If a factor of safety inconsistent with past performance has been calculated, then something in the ground model will need to be altered – for instance, the ground water table may be in the wrong place. Wesley (2010b) warns that because the seepage and pore water conditions at the time of failure are unlikely to be known, there will still be uncertainty in the strength parameters derived. Cornforth (2005) notes that even if there are errors in the measured pore pressure and back calculated strength, the same errors occur in the remedial calculations, so that any errors are cancelled out.
- Back-analyses are challenging if the failure has extended through multiple soil units. They will work best if it is assumed that soil strength along the failure surface is a constant (Cornforth, 2005).
- If the previous landslide has a ratio of length / height of less than 6 (Stark & Ruffing, 2017[21]), then two-dimensional (2D) analyses can result in back-calculated mobilized shear strengths that significantly higher than the three-dimensional (3D) analysis result. This does not matter if the dimensions of the failure surface in future (usually following a landslide mitigation) will be similar to those in the past. However, if the potential failure surface geometry changes significantly due to remedial measures, then either a 3D analysis should be carried out, or adjustments to the strengths derived from the 2D back analysis should be adjusted using the chart presented in Stark & Ruffing.

[21] Stark & Ruffing refer to a width/height ratio. By "width" it is inferred that they mean "landslide dimension perpendicular to the primary direction of movement", which the Unit 3 authors consider is better described as "length".

19 UNCERTAINTY AND PROBABILISTIC ANALYSES

Slope stability analysis often involves a high level of uncertainty related to the inherent variability in:

- Shear strength
- Pore water pressure
- Distribution of materials within the slope
- External environmental effects such as surface loads, rainfall events, earthquakes.

It is common practice for these uncertainties to not be explicitly defined in analyses but managed through FoS requirements and/or conservative soil strength assumptions. Recommendations in Section 13 are provided to aid the geoprofessional in selecting a minimum FoS that is consistent with the levels of uncertainty and consequence specific to the project. Alternatively, the geoprofessional can undertake a probabilistic stability analysis to quantitatively define the likelihood of slope failure.

Advancements in technology make the handling of uncertainty in analyses more accessible as modern commercial software integrates tools for sensitivity and probabilistic analyses. In New Zealand, the explicit consideration of uncertainty through routine sensitivity studies and/or probabilistic analysis 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 basis for decision-making.

Approaches for treatment of uncertainty include:

- Sensitivity Analysis/Parametric Studies - This is a simple yet powerful technique to assess the influence that each input parameter has on the performance or FoS of the slope. For most routine studies, sensitivity analysis will be sufficient to gain an understanding of uncertainties, and a full probabilistic analysis is not required.
  The approach is outlined in Section 10.3 of Module 3 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 Analysis – in this method the probability of slope failure is estimated 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). Gather ample data on these parameters. This data can come from field tests, laboratory experiments, and relevant literature.
  - Represent the uncertain parameters using probability distributions based on available data or expert judgment. This is discussed in Section 19.2.
  - Estimate the probability of failure. This is commonly done using Monte Carlo simulations as discussed in Section 19.1.
  - Once the probability of failure and the consequence of the failure are understood, slope stability can be assessed within a risk framework.

Probabilistic seismic stability assessments are described in Bray & Macedo, (2023), Macedo et al., (2018), Rathje et al. (2014), and Travasarou et al. (2004). These assessments involve combining the uncertainty in the seismic demand hazard curve with the anticipated displacements for a range of slope material parameters. A logic-tree approach can be adopted, where the weights consistent with the expected distribution of the parameter are assigned. Analysis is undertaken for each branch of the logic tree to produce an annual exceedance probability versus calculated slope displacement hazard curve for each branch. The mean hazard curve can then be calculated and anticipated performance is judged based on the calculated displacement.

19.1 PROBABILISTIC FORMULATIONS FOR SLOPE STABILITY

Probability of failure can be estimated using several methods including the Taylor series method, the point estimate method and Monte Carlo simulation method. Monte Carlo simulation is computationally intensive but is the method implemented in modern software packages making it accessible and widely adopted. In Monte Carlo simulation, large sets of variables are randomly sampled consistent with their prescribed probability distributions. For each set of samples limit equilibrium methods are utilised to calculate the FoS (i.e. trial runs) resulting in many FoS values. The proportion of those values with an FoS less than one indicate the probability of failure.

Some considerations when carrying out probabilistic analysis are outlined below.

- In Monte Carlo simulation, how many trial runs are "enough"? In most cases a few thousand trial will be adequate, and the solution will not be overly sensitive to number of trials beyond this. A useful way of assessing whether the number of trials is adequate is by running analyses and reviewing convergence plots that show the change in the results with the number of trials.

- To reduce computation time the probabilistic analysis is typically carried out for the critical failure surface from deterministic analysis. The deterministic critical failure surface will not always be the representative of the probabilistic critical failure surface. Where it is desirable to run probabilistic trials on only the deterministic surface, the sensitivity to consideration of the deterministic critical failure surface versus the probabilistic critical failure surface should be checked.
- Modelling software typically allows for accounting of uncertainty in the material parameters, groundwater table and external loads. However, there are other sources of uncertainty that are not as easily captured, such as the distribution of materials within a slope, the slope geometry and the potential for alterations of the slope, etc. These unaccounted-for uncertainties can result in an actual probability of failure different than that calculated.
- Probabilistic analyses are typically aimed at capturing the spatial variability and uncertainty in soil strength parameters but often don't reflect time-dependant variability such as that arising from high rainfall events or earthquake loading. Without time-dependent variables, the computed probability of failure does not have a timescale. The probability of failure is instead related to the time-scale assumptions made in selection of distributions and statistical parameters. Logic-tree approaches can be useful for undertaking probabilistic assessment for time-dependent events such as earthquake loading.

For further discussion, Abramson et al. (2002) and Duncan et al. (2014) provide a general overview of probabilistic slope stability.

19.2 QUANTIFYING UNCERTAINTY

Modern limit equilibrium slope stability software packages have integrated tools that make undertaking a probabilistic analysis relatively straightforward computationally; the difficulty lies in appropriately quantifying the uncertainty in the input parameters to achieve meaningful results.

The uncertainty in input parameters is represented by a statistical distribution and associated parameters, typically mean value and standard deviation to create a probability density function (PDF). These inputs are discussed below.

19.2.1 Statistical Distribution

In practice there is rarely adequate data and understanding to select the appropriate statistical distribution for input parameters so we must use experience with similar projects (Adams, 2015; Abramson et al., 2002). Commonly used distributions in geotechnical engineering are outlined in Table 20 along with general advice for their use in slope stability analyses.

Table 20: Statistical Distributions

Distribution	Notes	When to use
Normal	- Most common - Generally truncated at ±3 standard deviations from the mean (Abramson et al., 2002). - Best used where the variability is small (Look, 2017). - There is some differing of opinion on the use of this approach for soil strengths. Look (2017) notes that this distribution is generally not applicable to strength as negative values can result while Abramson et al. (2002) suggests that the normal distribution should be used for most parameters unless there is sufficient evidence to the contrary.	- Use as the default distribution for parameters where the variability is small, typically unit weight and friction angle. - Can be used for cohesion and undrained shear strength but needs to be truncated to prevent unrealistically low or negative values. - Truncate at $\pm 3\sigma$ from the mean - Can be used for defining seasonal groundwater variation.
Uniform/ Broad Triangular Distribution	- A uniform distribution describes an equal probability of the variable between specified limits and a triangular distribution is defined by a highest, lowest and most common with linear interpolation between. - A uniform or broad triangular distribution can be useful distributions where there is very little data on which to base selection of a mean and standard deviation. - While being statistically unlikely these distributions reflect high uncertainty (Adams, 2015).	- Use where there is insufficient data to confidently define a more sophisticated distribution
Lognormal	- This is one of the simplest of the distributions that avoid negative values. - Look (2017) recommends this distribution for soil and rock applications.	- Use for rock strength parameters (Look, 2017) - Can be used for soil strength parameters, particularly cohesion and undrained shear strength.
Exponential	- This distribution can be useful for defining parameters associated with extreme or infrequent events such as the horizontal loading from earthquake shaking and the level of the water table reflecting extreme rainfall events.	- Use to define rare events such as extreme rainfall events or earthquakes

19.2.2 Standard Deviation and Coefficient of Variation

Where a normal or lognormal distribution is used to define a parameter's variability, a mean and standard deviation are required. Where there are enough measurements, the standard deviation can be computed as follows:

$$\sigma = \sqrt{\frac{1}{N-1}\sum_{1}^{N}(x-\bar{x})^2} \qquad \text{Equation 28}$$

where:

σ = the standard deviation (note that sigma is defined elsewhere in this document as the overburden stress)

N = the number of measurements

x = the measured variable

$\bar{x}$ = the mean value of x

There is often not enough data to determine the standard deviation according to Equation 28. Rough estimates of parameter variability based on published values may be of some use, however these cover wide ranges so experience and judgement are required (Duncan et al., 2014). Published values of soil parameter variability are typically reported as Coefficients of Variation (COV). The COV is the standard deviation divided by the mean (Equation 29) and is a convenient measure as it is dimensionless. Published ranges of COV are outlined in Table 21.

$$COV = \frac{\sigma}{\bar{x}} \qquad \text{Equation 29}$$

Table 21: Published Coefficients of Variation for Geotechnical Parameters

Parameter and Testing Type		COV (%)	Reference
Unit Weight		<10	Phoon & Kulhawy (1999)
		3 to 7	Harr (1987), Kulhawy (1992)
Friction Angle		5 to 15	Phoon & Kulhawy (1999)
		2 to 13	Harr (1987), Kulhawy (1992), Duncan (2000)
Undrained Shear Strength, S_u	UC Test	20 to 55	Phoon & Kulhawy (1999)
	UU Test	10 to 30	
	CU Test	20 to 40	
	Vane Shear Test	10 to 40	
		10 to 20	Kulhawy (1992)
	S_u (overall)	13 to 40	Harr (1987), Kulhawy (1992), Lacasse & Nadim (1997)
S_u/σ'		5 to 15	Lacasse & Nadim (1997), Duncan (2000)

Another way of estimating parameter variation is using the Nσ rule (Foye et al., 2006), an adjustment to the 3σ rule described by Dai & Wang (1992). This rule of thumb uses the concept that 99.7% of all values of a normally distributed variable fall within three standard deviations of the mean, coupled with the fact that geoprofessionals tend to underestimate the range that ±3σ spans. This rule is expressed as

$$\sigma = \frac{HCV - LCV}{N_\sigma} \qquad \text{Equation 30}$$

Where HCV is the highest conceivable value, LCV is the lowest conceivable value and N_σ is a value less than 6. Duncan et al (2014) indicate that an N_σ value of 4 is appropriate for many conditions.

19.2.3 Parameter Correlation

Some parameters have been found to be correlated. For example, cohesion and friction angle have often been found to be negatively correlated, i.e. materials with low friction angles tend to have high cohesion and materials with low cohesion tend to have higher friction angles (Grivas, 1981; Lumb, 1970; Wolff, 1985). It is possible to account for these correlations in commonly used software packages (e.g. Slope/W, Slide2) through the use a correlation coefficient; however, these coefficients can be difficult to define with the quantity of data typically available (Adams, 2015).

19.3 INTERPRETING RESULTS OF PROBABILISTIC ANALYSIS

Results of probabilistic analyses are expressed a probability of failure or through the reliability index (β). The reliability index is directly related to the probability of failure and indicates the number of standard deviations between a FoS of one and the most likely FoS. The relationship between reliability index and probability of failure is shown in Figure 78.

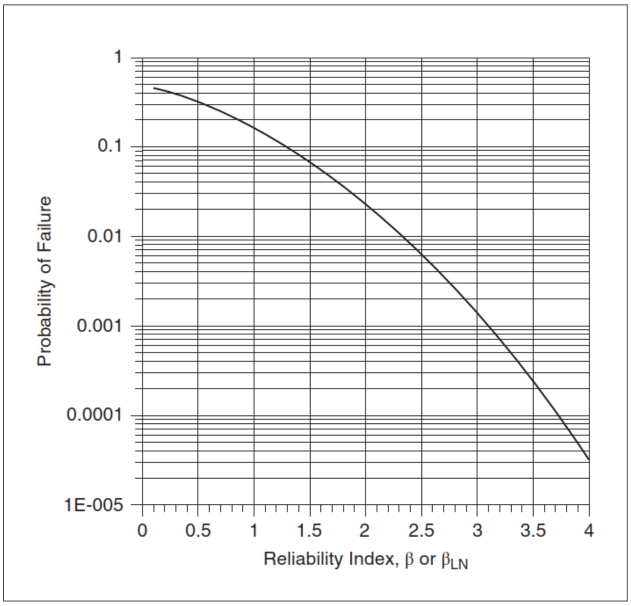

FIGURE 78: Relationship between reliability index and probability of failure (Duncan et al, 2014)

There is no universally accepted threshold value of probability of failure. The significance of the probability of failure depends on expected consequence of that failure. For example, a shallow failure may have a high probability of failure, but minor consequence so therefore is less critical than the deeper failure surface with a lower probability of failure that results in serious consequences.

Defining an acceptable or tolerable probability of failure for a given consequence (i.e. risk) is a complex, context-specific task that depends on local regulation and a clear understanding of the desired outcomes as agreed on by the project stakeholders. Section 13.9 provides ranges of acceptable annual probabilities of failure for a range of consequence levels reflecting acceptable risk from a variety of reviewed risk thresholds used in New Zealand. These ranges are summarised in Figure 79 and consequence categories are shown in Table 8. The applicability of these values should be specifically considered on a project-by-project basis.

The results of probabilistic analysis can be also applied within a quantitative risk framework where the consequence of the failure is defined quantitatively. Risk assessment is discussed in Unit 1 Part 6.

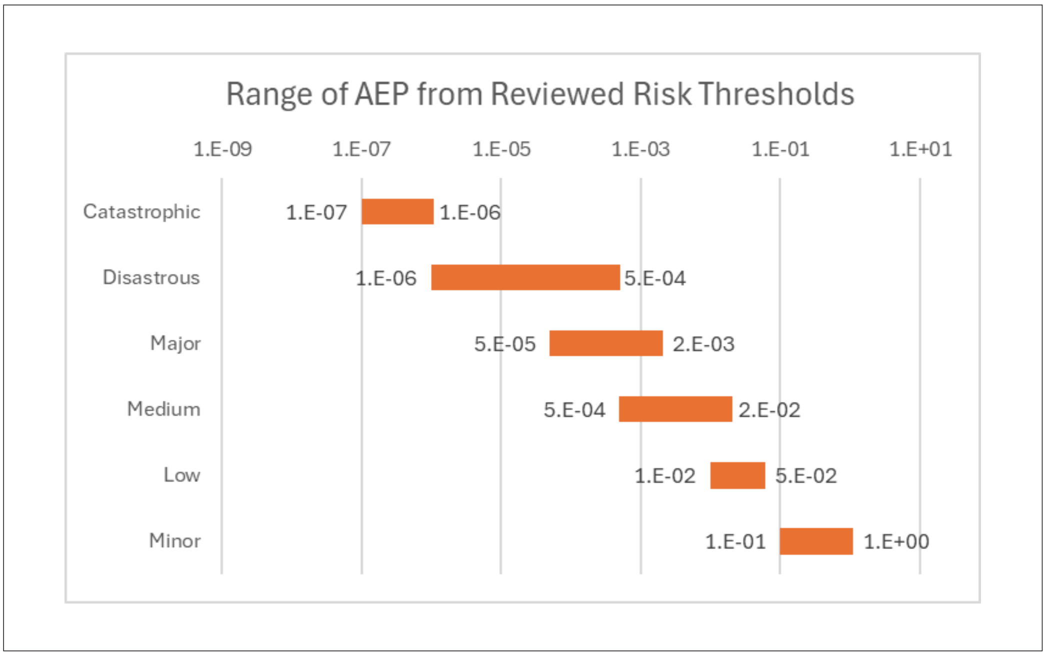

FIGURE 79: Range of Acceptable Annual Probabilities of Failure vs Consequence

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