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# Understanding failure mechanisms to design and manage risks for surface and underground excavations

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## Abstract

Ground failures are a common aspect of rock slope and underground excavation design. Some ground failures are accepted and even economically attractive provided they are managed so that no personnel are at unacceptable risk. Too many failures impact on productivity and project costs. Larger scale ground failures are less tolerable due to their economic impacts, schedule delays and negative publicity, irrespective of how well they were managed. Poorly managed ground failures have been the cause of numerous fatalities in mining, tunnelling and construction industries.

Failure mechanisms in surface and underground excavations are becoming more and more understood. With improvements in numerical modelling since the 2000s, it has become possible to replicate almost all complex failure mechanisms in numerical simulations. However, more often than not, this level of detail occurs only after a ground failure event has happened. Much of the predictive stability modelling that is undertaken during the design of a rock slope or underground excavation contains significant geological uncertainty, includes simplification when simulating potential geotechnical problems and does not consider all plausible failure mechanisms. For many projects, this design is static, and is not revisited after excavations commence and more engineering geological information becomes available.

This paper focuses on the need for understanding failure mechanisms to effectively design surface and underground excavations and manage residual risks through a combination of a continuous improvement framework and critical control measures.

## Keywords

Failure mechanisms, Rock slopes, Underground excavations, Continuum, Discontinuum.

# 1 Introduction

Collapses or failures of rock slopes or underground excavations present various forms of risk to people, equipment, infrastructure and the environment.

Rock slope failures on man-made and natural slopes include slope instability, rock falls and landslides, as well as debris flows and shallow slips in weathered rock. Similarly, for underground excavations, collapses may involve excavation-scale instability, rock (or block) falls, squeezing, rockburst and inrushes in some conditions.

## 1.1 Socioeconomic impacts of ground failure events

The socioeconomic consequences of rock slope and underground excavation failures have direct costs such as removing the failed rock debris and stabilizing the excavation. There is also a wide range of indirect costs which include, but are not limited to:

- Potential losses of life;
- Damage to vehicles and injury to passengers on highways, railways and tunnels or damage to equipment and injury to personnel in mining;
- Damage to buildings and infrastructure in urban areas in the event of near surface tunnel collapses;
- Traffic and goods transport delays;
- Business disruptions, loss of revenue or reduced productivity;
- Decreased land values and associated loss of tax revenue;
- Flooding and/or disruption of water supplies where rivers are blocked by landslides;
- Impairment or sterilization of mineral resources;
- Legal costs, social costs, reputational damage to designers, construction and mining companies with potential for company closure.

In the twelve-year period between 2004 and 2016, almost 56,000 fatalities were recorded globally (i.e. average of 4,666 per annum) from non-seismic slope failures including landslides (Froude and Petley, 2018). The majority of these slope failures involved human activity such as mining, construction and hill cutting and many were initiated by rainfall. As illustrated in Figure 1, more recent research indicates the frequency of fatal landslide events has increased considerably in 2024 and 2025 and included 4,933 and 5,091 fatalities, respectively (Petley, 2025; 2026).

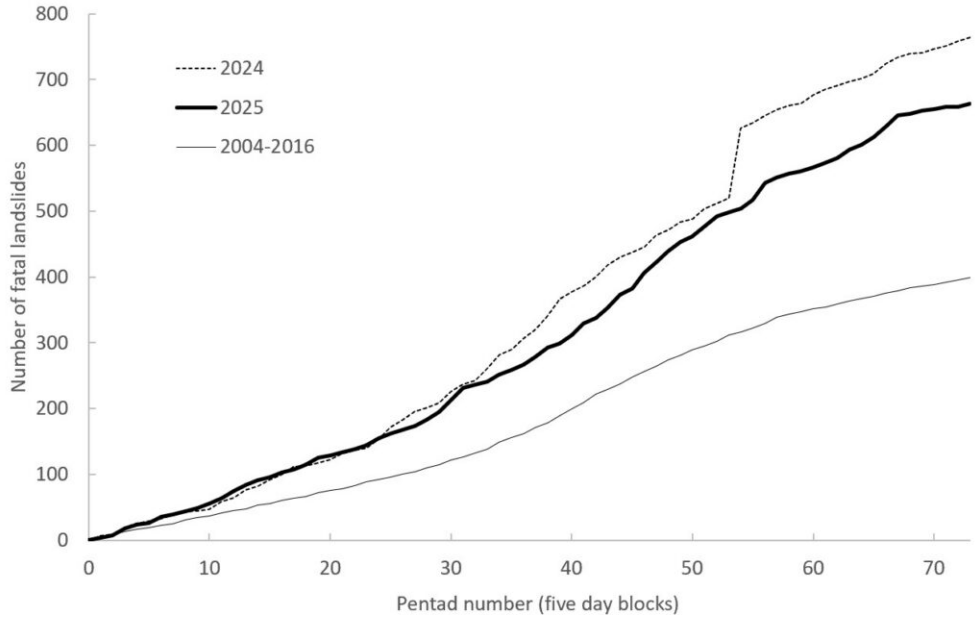



Fig. 1 Global fatal landslides in 2025 plotted using pentads (five-day blocks) with the 2004-2016 long term average and the exceptional year of 2024 are included for reference (Petley, 2026).

Documented examples of socioeconomic consequences of rock slope failures include:

- The Manefay Slide at the Bingham Canyon copper mine in 2013 was approximately 50 million cubic metres in volume (Ross, 2017). It was estimated to cost over $1 billion to clean

up. In Salt Lake County, Utah, sales-tax revenues reduced by 3.4% by the end of 2013 and were attributed to the slope failure (Ward, 2015);

- Several fatal landslide events have occurred in recent years in the mining industry and resulted in 200 fatalities in 2026 at Rubaya Coltan mine, Democratic Republic of the Congo (RFI, 2026) and 53 fatalities in 2023 at Xinjing coal mine, China (Li et al. 2025);
- In Medellin, Colombia, where bedrock is highly weathered and intense rainfall events occur, 74% of deaths resulting from natural hazards are due to slope failures. A single slope failure in 1974 resulted in over 770 fatalities (Carvajal et al. 2012).

The consequences of rock failure in underground excavations were the source of significant research and technological improvement throughout the 20th century; including improved rock mass classification and empirical design, the use of improved ground support such as shotcrete and systematic bolting to replace steel arches as well as the use of tunnel boring machines (TBM).In the 21st century, further advancements included the use of numerical simulations of stability, monitoring, and an enhanced 'safety first' culture. By way of comparison, the construction of the Gotthard tunnels in Switzerland had significantly less rock failure related fatalities in the 20thand 21stcenturies as excavation methods and ground support technology improved (Idris et al. 2004):

- 1872-1882: 177 fatalities during railway tunnel construction;
- 1969-1980: 12 fatalities during road tunnel construction;
- 1999-2016: 2 fatalities during Gotthard Base Tunnel construction (Wikipedia, 2026).

For similar reasons to those in tunnelling, underground mining has also observed dramatic reductions in fatalities in many countries. By way of example in Figure 2, fatalities from roof failures in underground coal mines in the United States (US) have reduced significantly. Mark (2024) indicated that more than 50,000 fatalities from roof falls were recorded in the 20th century. This has dramatically reduced in the first part of the 21st century with zero fatalities from roof falls being recorded in 2016, 2018, 2019 and 2020 (Mark, 2024). However, further improvement remains possible with 46 fatalities and 33 permanent disabilities recorded from roof falls in coal and non-metal mines in the US between 2010 and 2019 (Rashed et al. 2022).

Higher levels of improvement were observed in the Western Australian mining industry where rock falls were ranked the number one cause of fatalities in the 1980s and 1990s (DMP, 2014). Between 2000 to 2012, only three fatalities were recorded in underground mines related to working below unsupported ground and two from pit slope failures.

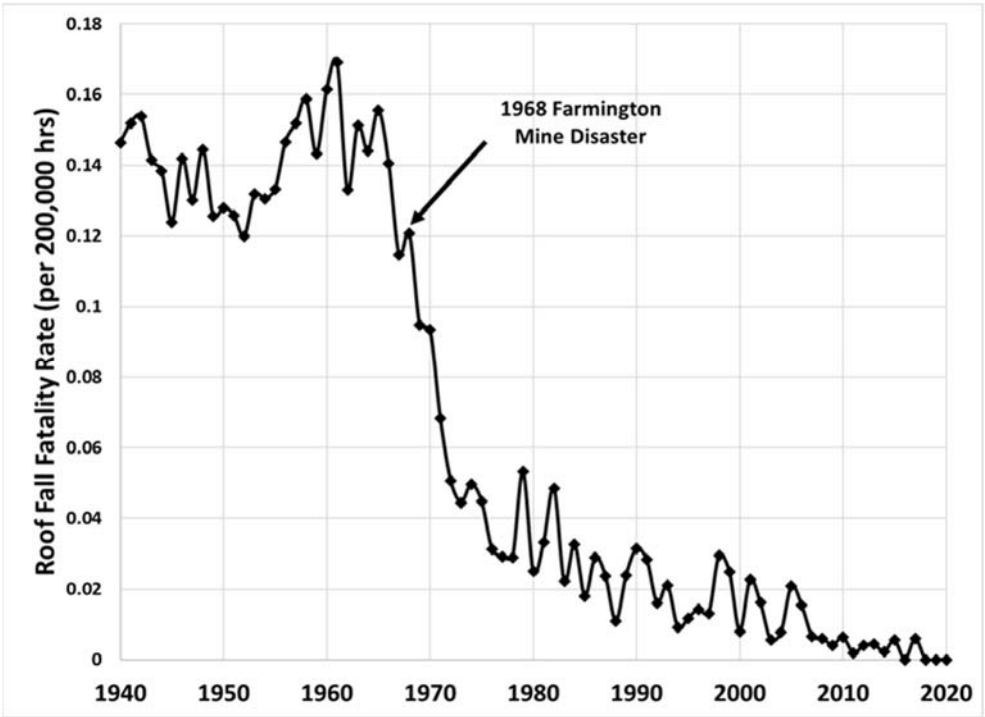



Fig. 2 Roof fall fatality in US underground coal mining, 1940-2020 (Mark, 2024).

Rock failures can also initiate secondary hazards. A rock slope failure into a body of water can trigger a seiche (tsunami in a closed body of water). Examples can be found in nature and civil and mining engineering projects, including:

- Rock slides from Ramnefjell mountain into Lake Lovatnet caused enormous waves that washed up to 74 m above the lake's water level in Norway during the first part of the 20th century and caused more than 60 fatalities (Grimstad, 2005; Waldmann et al. 2021);
- The Vajont slide in Italy in 1963 inundated a reservoir, sending a wave over the crest of the dam that destroyed the town of Longarone and resulted in over 2000 fatalities (Kiersch, 1965; Ghirotti et al. 2013);
- A pit wall failure into a flooded pit at Katanga copper mine in the Democratic Republic of the Congo resulted in seven fatalities (Jamasmie, 2016). Days later, a similar event at Ok Tedi copper-gold mine in Papua New Guinea caused another fatality (Tlozek, 2016).

A voluminous failure underground can cause an airblast or a sudden and often dangerous displacement of a massive volume of air; for example, a pillar collapse during block caving caused an air blast that killed four miners at North Parkes mine in Australia in 1999 (Bailey, 2003).

Smaller sized rock falls or failures in operating railway tunnels can be the cause of derailment as had occurred in the Vierzy railway tunnel in 1972, which resulted in 108 deaths (Arnould, 2010).

## 1.2 Geological (inherent) uncertainty

In both civil and mining engineering projects, it is practically impossible to assess the stability of rock slope cuttings and benches, tunnel advance or underground mine development in real-time, using analytical approaches such as kinematics, limit equilibrium or numerical modelling. The rate of excavation advance is usually too fast for this (Bar, 2024). Empirical methods using rock mass classification systems such as RMR (rock mass rating) and Q for underground excavations and SMR (slope mass rating) and Q-slope offer near real-time approaches for assessing stability and support requirements. However, rock mass classification systems often may not be able to address all the issues at stake. The trigger for the instability of a design may depend on minor details such as the occurrence, the precise orientation or the persistence of a geological structure (joint, fault, etc) in a particular position in the rock mass in relation to the excavation, the larger or smaller extent of an altered zone or the length and quality of a reinforcement element (Alejano, 2025).

Since rock slopes, tunnels and mines are excavated in natural geological formations (which usually have limitations with respect to site investigations), significant uncertainty and variability inevitably exists in the estimation or calculation of resting forces. Uncertainty in slope and underground excavation design primarily stems from the inherent natural variability of ground conditions; i.e. geological and engineering geological uncertainty and anomalies (Hoek and Diederichs, 2006).Figure 3 describes rock slope design uncertainty associated with the geotechnical components and is based on a similar concept for tunnelling scenarios by Paraskevopoulou and Boutsis (2020).

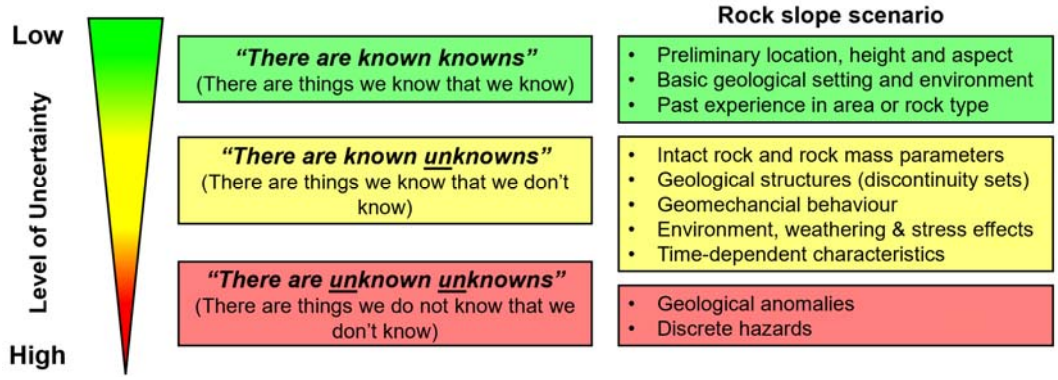



Fig. 3 Design uncertainty in association with the geotechnical engineering components for a rock slope scenario (Bar, 2024).

## 1.3 Design frameworks and processes

Rock slope design has been considered an iterative process in civil and mining engineering projects for 50 years since Hoek and Londe (1974) developed the closed-loop framework shown in Figure 4. This flow chart has a robust initial design process, which is followed by monitoring and observing

slope performance during excavation, and feeding back observations, lessons learned and new data into improving the design.

Similar frameworks for the design of rock slopes have been adopted in civil and mining engineering with guidance provided by Wyllie and Mah (2004) and for ground support design of underground excavations (Hoek et al. 2000).

In the late 2000's, divergence occurred between rock slope design in civil and mining engineering applications with the development of guidelines for open pit slope design (Read and Stacey, 2009) as part of the mining industry funded Large Open Pit (LOP)project. This was followed by an update to rock slope engineering guidance for civil engineering applications by Wyllie (2018). The resultant slope design process in Figure 4 for open pits by Read and Stacey (2009) includes an iterative process for considering mine planning and risk evaluation prior to excavation. However, unlike its predecessors, it does not have a clear feedback loop for improving and optimizing the design based on monitoring, new data, slope performance, etc.

It is understood that the next update of the LOP project (circa 2027) will include a closed-loop framework following the general principles of Hoek and Londe (1974) with consideration for modern slope and underground monitoring systems and their capability in identifying emerging hazards or threats (Nicoll et al. 2017; Lynch et al. 2018; Bar and Dixon, 2021).

Hoek et al (2000) and ÖGG (2010) provide similar schematic procedures for the geotechnical design of underground excavations. Many consulting firms and mining companies prescribe internal procedures for rock slope and underground excavation design for the purpose of standardization (Haile et al. 2020). In many instances, such processes may deviate significantly from the general principles by Hoek and Londe (1974) and Hoek et al (2000) and focus on achieving standardized outputs without investigating failure mechanisms in sufficient detail.

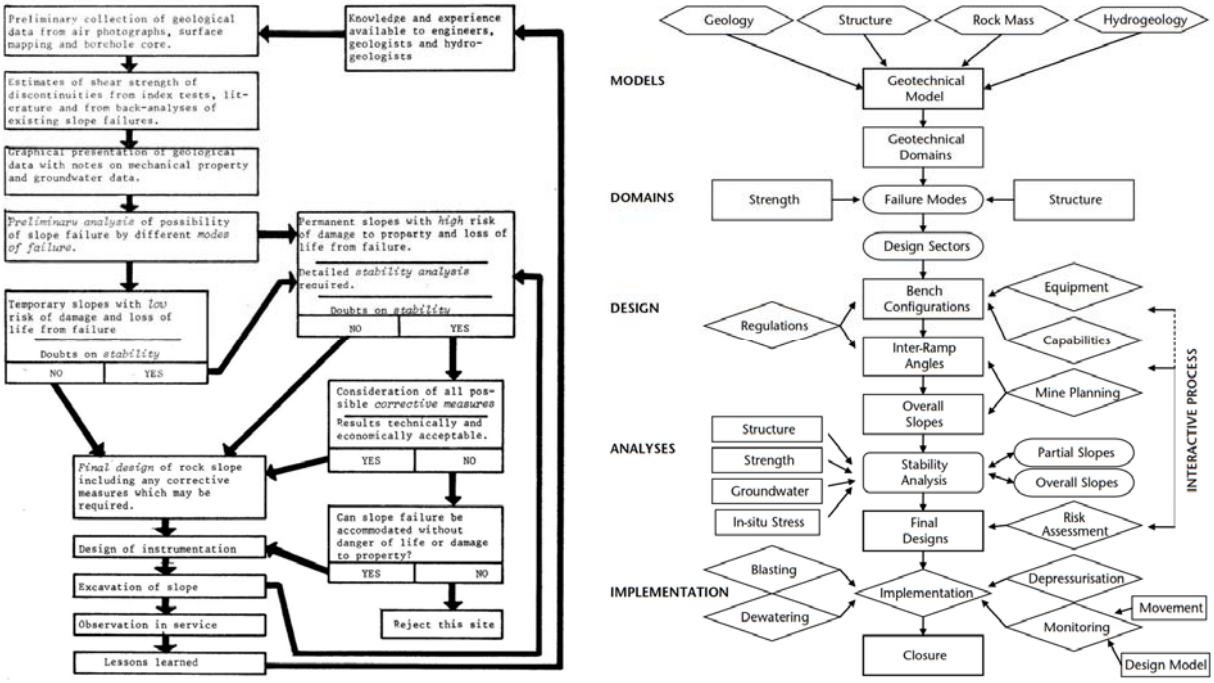



Fig. 4 Left: Closed-loop rock slope design flow chart (Hoek and Londe, 1974). Right: Linear open pit slope design process (Read and Stacey, 2009).

Rock excavation design and management in civil and mining engineering applications are quite different in terms of:

- The design life of a civil engineering slope, tunnel or cavern could be in excess of 100 years, whilst a slope, development or stope in a mine could range from a few weeks or months (individual benches, stopes and production developments), to a few years (inter-ramp slopes and capital development),or several years (overall slopes, shafts, etc).
- Risk exposure in civil engineering excavations involves the public (e.g. several thousands of people or more per day on roads, highways and railways) with very little or no access control

and without routine monitoring instrumentation. On the contrary, in mining, access is strictly controlled (e.g. typically less than 200 mine workers per day) with several options for additional control measures; including monitoring instrumentation.

- Prior to design, site investigations for civil engineering excavations may be limited to outcrop mapping and limited drilling for understanding subsurface conditions. On the contrary, mineral deposits are extensively drilled to understand the economic feasibility of deposit extraction. Arguably there should be a lower level of uncertainty in a mining engineering project than in civil engineering; however, geological and geotechnical conditions around a mining project are often significantly more complex.

Despite the abovementioned differences, uncertainty related risks described in Figure 3 affect the design and management of rock slopes and underground excavations in civil and mining engineering projects in a similar manner.

By adopting the framework for slope and underground excavation design and management in Figure 5, uncertainty and risk can be reduced and proactively managed through a continuous improvement process. Such a process would assist in managing safety and has capacity to add economic value by enabling slope optimization as well as underground excavation size and support optimization.




Fig. 5 Continuous Improvement Framework for Safe and Economic Rock Excavation Design and Management.

The continuous improvement framework in Figure 5 can be incorporated into a Ground Control Management Plan (GCMP) or Slope Management Plan (SMP) with detailed processes for each component. By way of example, various elements of the 'Geotechnical Model and Understanding Failure Modes' could be described in further detail, as could the process for their development and their limitations:

- Geological model (lithology, weathering and alteration);
- Structural model (major, medium and minor);
- Rock mass model;
- Stress model;
- Groundwater model;
- Potential failure modes.

Understanding the limitations of a geotechnical model and potential failure modes can be used to define the risks from which appropriate control measures can be developed including the selection of suitable modelling and monitoring methods.

# 2 Understanding failure mechanisms

Understanding failure mechanisms that can occur for a specific rock slope or underground excavation is a key step for selecting appropriate stability analyses to develop a suitable design. Such a design may consider slope angles and heights, underground excavation spans and shapes, ground support requirements, depressurization (e.g. pore pressure reduction) requirements, etc.

## 2.1 Rock slopes

Real world rock slope failures are rarely circular, spherical or ellipsoidal. In a recent review of over 500 rock slope failures in open pit mines, rock mass and circular failure modes accounted for only 10% and 8% of the database, respectively (McQuillan and Bar, 2024). The vast majority and remaining 82% were structurally-driven failures such as planar, wedge, toppling or step-path mechanisms. Similarly, in a database of over 600 rock slopes assessed as part of the development of the Q-slope method for rock slope engineering, Bar and Barton (2024) identified that circular failure accounted for only 3% of cases.

Since most failure modes in rock slopes are structurally-driven and may involve one or more geological faults or other persistent singularities, which are almost certainly oblique (i.e. not perfectly parallel) to the slope, stability should be assessed in three dimensions (3D). Rock slope failure modes should be assessed using 3D analysis method in a geotechnical design review process in the case of any of the following conditions (McQuillan and Bar, 2023):

- Non-linear slope geometry;
- Spatially or laterally varying geological and hydrogeological conditions;
- Spatially varying material strengths, including anisotropic material behaviour in the same unit;
- Singularities and persistent geological structures, striking and intersecting up to 50° from the slope orientation;
- Highly variable 2D results within close spatial proximity to each other.

Many rock mechanics and geotechnical engineering textbooks describe the four simple failure mechanisms for rock slopes shown in Figure 6 (Wyllie and Mah, 2004):

a. Plane failure, i.e. planar sliding along one persistent discontinuity which is dipping out of the face. Planar failures generally occur when the discontinuity is sub-parallel (±20°) to the strike of the slope;
b. Wedge failure, i.e. sliding along the intersection of two discontinuities. The plunge of the line of intersection between the two joint planes must be flatter than the dip of the slope face;
c. Toppling failure occurs in strong rock containing discontinuities dipping steeply into the face. Two types of toppling exist: block (*e*) and flexural (*f*) per Figure 7;
d. Circular failure in rock fill, very weak rock or closely fractured rock with randomly oriented discontinuities (Wyllie and Mah, 2004). However, the textbook definition of circular failures is seldom applicable to rock slopes. Although in many instances, failure mechanisms visually appear circular or ellipsoidal in jointed rock masses, they are usually step-path mechanisms involving some dominant discontinuity sets. As such, the term ‘circular failure’ is often interchanged with the terms ‘rock mass failure’ or ‘step-path failure’ when dealing with rock slopes. In rare cases, circular failures occur in extremely weak, homogeneous rock masses that display soil-like properties (e.g. saprolites).

Goodman and Shi (1985) and Poisel and Preh (2004) discuss failure mechanisms for rock slopes in greater detail as shown in Figure 7:

e. Block toppling involving the forward rotation of column or slab-shaped blocks, similar to dominos. These mainly occur in high strength rocks with low discontinuity strengths (Goodman and Bray, 1976);
f. Flexural toppling caused by bending of column or slab-shaped rock blocks like a cantilevered beam (Hittinger and Goodman, 1978);
g. Rock slumping involving the backward rotation of rock blocks (Kieffer, 1998);
h. Slab translation, rotation or tower descent can occur where high strength rocks overlay an incompetent base – “hard on soft” (Poisel and Eppensteiner, 1988);
i. Column buckling of slab-shaped blocks can occur where the slab thickness is significantly less than the slope height (Cavers, 1981);
j. Kink band slumping involves an ‘S’ shaped deformation in laminated rock due to slope creep (Kieffer, 1998);

k. Active-passive blocks (or wedges) involve the sliding of multiple rock blocks on a polygonal sliding plane, which is usually 'activated' by the upper block(s) and resisted to a degree by the lower block(s) (Gußmann, 1981);
l. Translational creep involves continuous downslope deformation and is mainly associated with low strength rocks such as shales and phyllites (Zienkiewicz et al. 1975);
m. Block falls, also termed rock falls or falling of loose blocks (Goodman and Shi, 1985);
n. Single block rotation on a discontinuity (Wittke, 1990);
o. Complex failure mechanisms formed by a combination of components of failure mechanisms described in *a* to *n*. Geological conditions are often complex such that failure mechanisms often involve a combination of the simple modes of failure (i.e. a combination of plane, wedge, toppling and circular) to form a 'complex' (or composite) failure mechanism. Often, this is also referred to as 'step-path failure' and this category of failure involves various combinations of shearing on discontinuities and fracturing of intact rock bridges (McMahon, 1971; Baczynski, 2000; Baczynski and Bar, 2019).

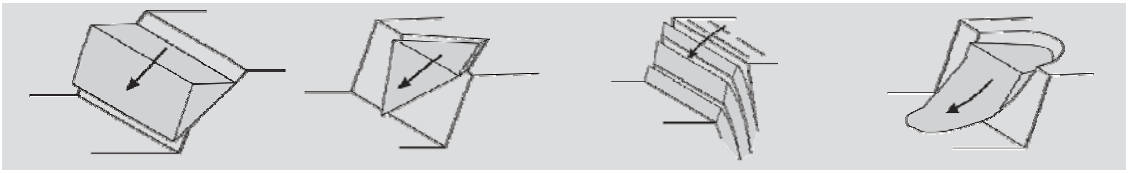



Fig. 6 Illustrations of simple rock slope failure mechanisms (Wyllie and Mah, 2004).

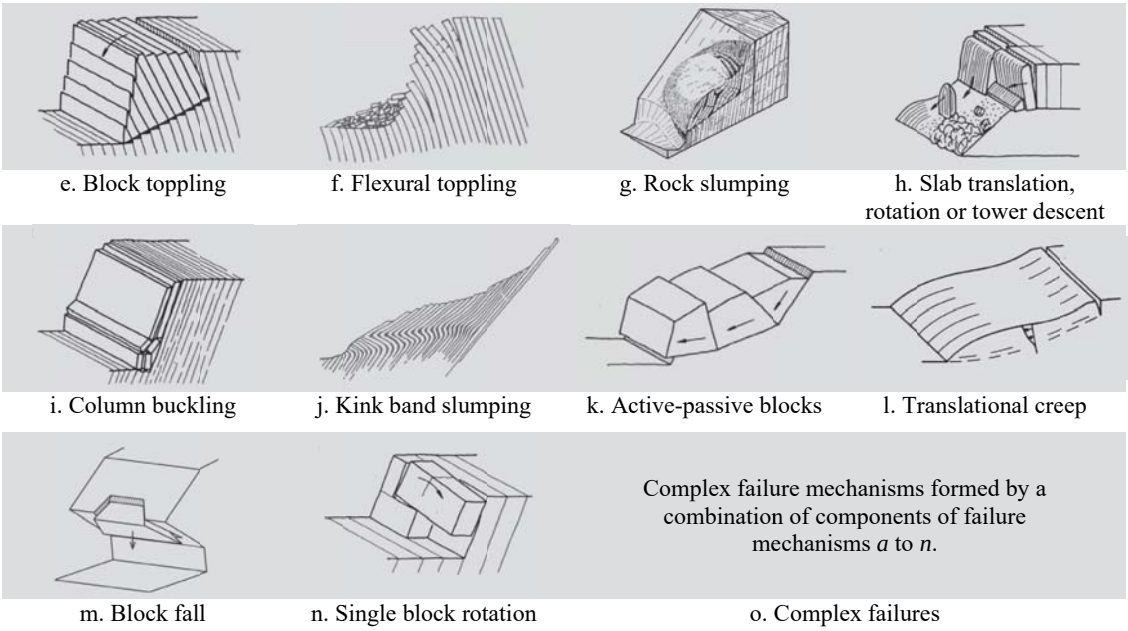



Fig. 7 Illustrations of additional rock slope failure mechanisms (Poisel & Preh, 2004).

Yuan et al. (2026) describe several combined failure mechanisms in rock slopes (e.g. active-passive blocks comprising a planar slide in the active block causing toppling in the passive block below).

Slope stability in weak and complex rock masses is a practical problem where anisotropy, weathering, and heterogeneity must be incorporated adequately in geotechnical models for stability analyses (Jovanovski et al. 2024).

For the design of rock slopes, it is critical to ensure that reasonable failure modes are investigated based on the geotechnical model. Quite simply, assessing for rotational (circular or near-circular) failure modes is considered inadequate in the context of real world, physical outcomes (Bar, 2024).

McQuillan et al. (2020) undertook a survey of the Australian coal mining industry enquiring what types of slope stability analyses were routinely applied. The results indicated over 90% of engineers applied 2D Limit Equilibrium Method (LEM) and over 80% undertook kinematic analysis using stereographic projections. Kinematic analysis is able to assess the potential for simple failure modes such as planar and wedge sliding and toppling. 2D LEM is able to calculate the Factor of Safety of

slopes with potential for planar sliding and circular or near-circular rotational failure mechanisms. Neither of the abovementioned commonly used methods can assess the Factor of Safety of slopes with potential wedges, toppling, active-passive blocks, complex or combined failure modes, or slopes with complex geometry or the inclusion of one or more faults or anisotropy that are not perfectly perpendicular to the 2D cross-section. Remarkably, less than 15% of survey responses indicated the use of 3D numerical methods; which are able to assess complex failure mechanisms which may be structurally-driven.

For the purpose of calculating a Factor of Safety, 2D and 3D LEM can only be used for failure mechanisms *a*, *b*, and *d* from Figures 6 and 7, and only when discontinuities are not steeply dipping (i.e. generally with a dip ≤70°). The remaining failure mechanisms require numerical simulations (i.e. finite element, finite difference, discrete element or hybrid methods).

## 2.2 Underground excavations

As part of a general underground excavation design procedure (Figure 9), three key steps are usually followed (Schubert et al. 2003):

- Determining rock mass types (i.e. assessing engineering geological conditions and geotechnically relevant key parameters for each ground type);
- Determining actual rock mass behaviour types (i.e. observing excavations for signs of excessive stress, deformation patterns and failure mechanisms);
- Determining excavation and ground support criteria and verifying system behaviour.

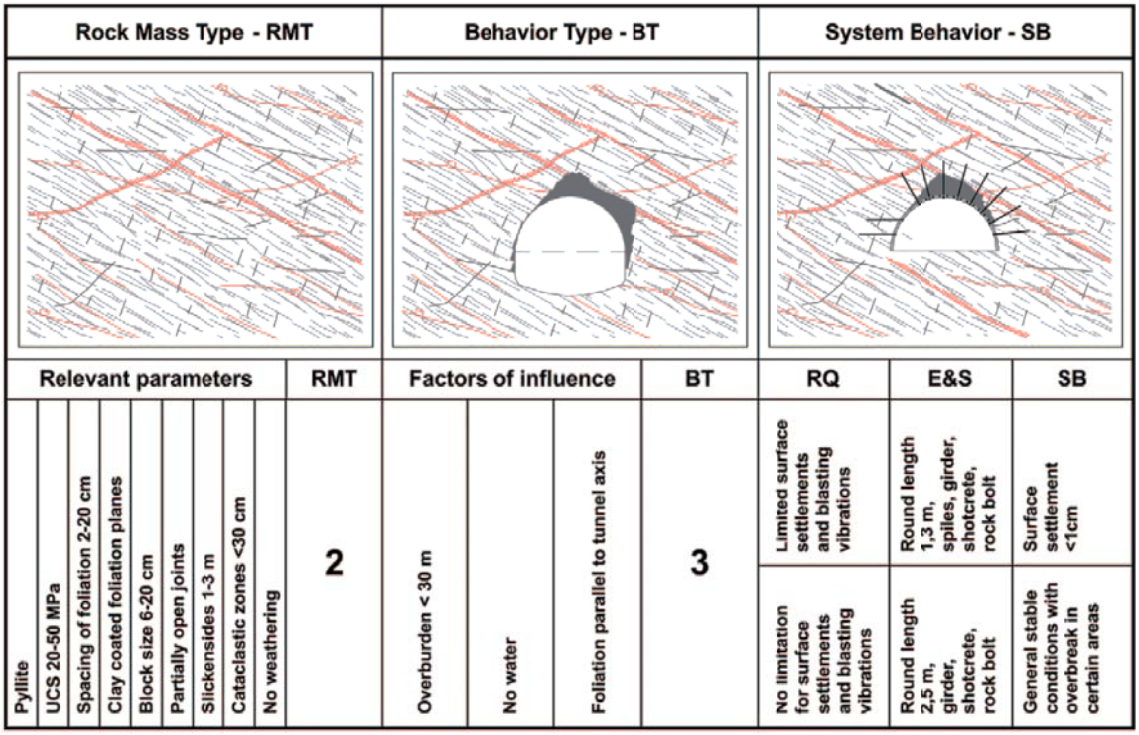



Fig. 8 Example of the underground excavation design procedure (Schubert et al. 2003).

When determining behaviour types for each rock mass, consideration must be given to local influencing factors including the relative orientation of discontinuities to the excavation, groundwater conditions, stress situation, etc. The behaviour of underground excavations should be evaluated for the full excavation without considering any excavation method or sequence and support or other auxiliary stabilization measures. Figure 9 summarizes eleven general categories that most-commonly occur:

1. Stable ground with the potential of small, local, gravity-induced falling or sliding of blocks;
2. Voluminous discontinuity-controlled, gravity-induced fall and sliding of blocks (or wedge failures) with the occasional local shear failure on discontinuities. Stability is controlled by the geometrical and mechanical characteristics of the discontinuities;
3. Shallow stress-induced failure in combination with discontinuity- and gravity-controlled failure;
4. Stress-induced failure involving large ground volumes and large deformation (i.e. squeezing ground);

5. Rockburst: a sudden and violent failure of the rock mass, caused by highly stressed, brittle rocks, and the sudden release of accumulated strain energy. Brittle failure and rock bursting typically occurs at depth;
6. Buckling and anisotropic strain of rocks with a narrowly-spaced discontinuity set, frequently associated with shear failure;
7. Voluminous overbreak in the crown with progressive shear failure (also termed chimney type failure);
8. Ravelling of dry or moist, intensely fractured, poorly interlocked rocks or soil with low cohesion;
9. Flow of intensely fractured, poorly interlocked rocks or soil with high water content;
10. Swelling ground involves a time dependent volume increase of the ground caused by physical-chemical reaction of the ground and water in combination with stress relief;
11. Complex failures may involve a combination of several behaviours with strong local variations of stresses and deformations over long sections due to heterogeneous ground (e.g. in heterogeneous fault zones; block-in-matrix rock, tectonic melanges, etc).

The aforementioned failure mechanisms can be conceptualized using sketches as shown in Figure 8, and later evaluated using numerical methods, which can then form the basis for determining excavation requirements and ground support systems. 3D LEM is only applicable to behaviour type 2 (discontinuity-controlled block fall).

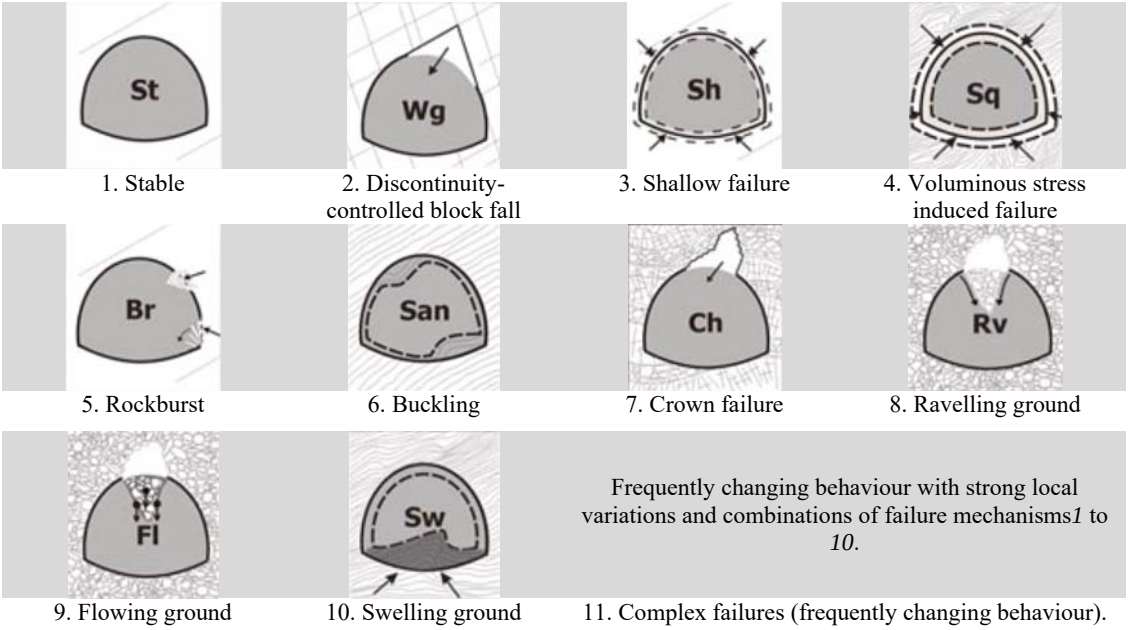



Fig. 9 Underground Excavation Behaviours and Potential Failure Mechanisms (after Schubert et al. 2003; Oke et al. 2013)

Hoek & Brown (1980) presented concepts, which have been developed over 45+ years to relate intact rock strength (UCS or $\sigma_c$) and the maximum principal stress ($\sigma_1$) using the stress versus strength ratio and GSI (formerly $RMR_{76}$) to indicate potential failure modes of underground excavations (Figure 10).

Barton et al. (1974; 2024) provide similar stress versus strength ratios (Stress Reduction Factor B, $SRF_b$) to provide insights into potential stress-related rock mass damage.

The continued use of rock mass classifications including RMR or the Q-system for assessing excavation stability and ground support requirements remains invaluable. Conceptual and empirical approaches are particularly useful for validating the appropriateness of numerical modelling outputs prior to the commencement of excavations and during excavation as part of a reconciliation or validation process. However, on their own, empirical approaches are generally unable identify the plausible failure mechanisms without significant engineering geological judgement.

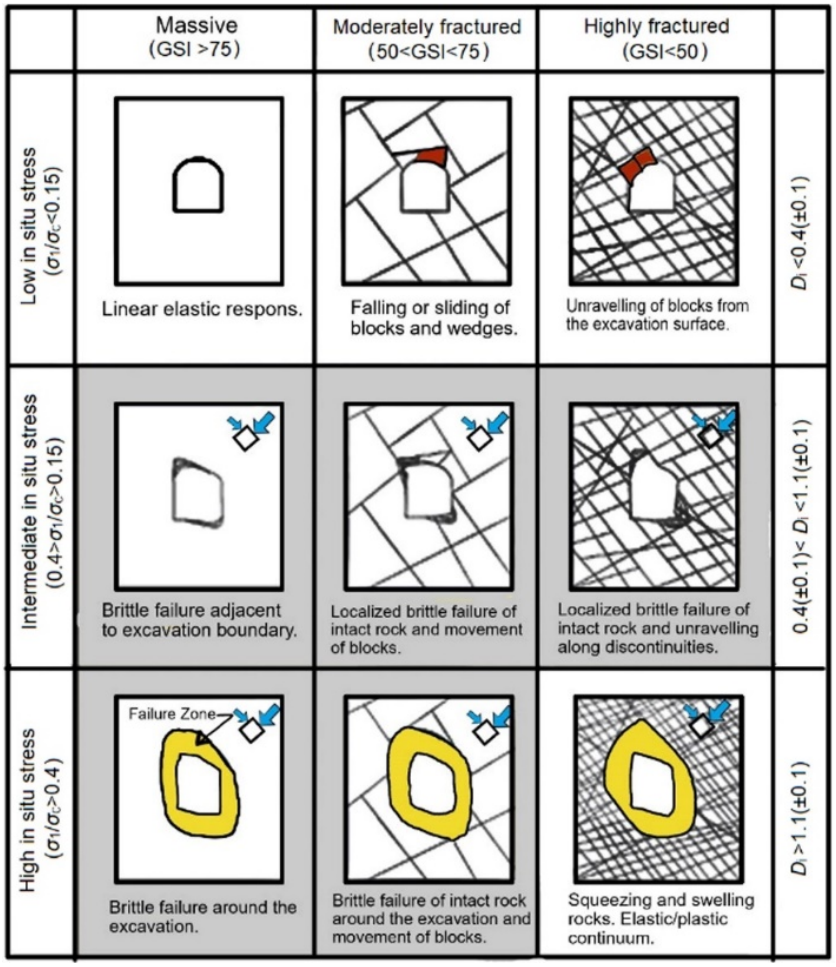



Fig. 10 Potential Failure Modes based on the Ratio of In-Situ Stress and Intact Rock Strength and GSI (after Askaripour et al. 2002; Castro et al. 2012; Hoek & Brown, 1980; Martin et al. 1998).

# 3 Simplification risks when simulating geomechanical problems

Fifty years ago and beyond, rules-of-thumb and empirical methods formed the basis of underground excavation and ground support design. Empirically-derived rock mass classification systems such as Rock Mass Rating, RMR (Bieniawski, 1973; 1989) and the Q-system (Barton et al. 1974; 2024), were developed to assess rock mass quality from drill core and face mapping for predicting or updating support requirements for underground excavations.

Erharter et al (2025) undertook a global study on the use of 37 (empirical) rock mass classification systems and identified that the most used systems for tunnels, caverns and underground mines were the Q-system, RMR and Geological Strength Index, GSI (Hoek & Brown, 1997; Marinos, 2019). Similarly for rock slopes, the most used systems were GSI, Q-slope and RMR.

By commonly applying rules-of-thumb and empirical approaches, significant simplifications were made from understanding ground behaviour to developing a solution for excavation stability based on available case records.

## 3.1 Levels of simplification in numerical models

Numerical simulations or models can be used to overcome limitations of kinematic and limit equilibrium analyses. Numerical simulations can be completed in 2D or 3D and have the capability to calculate stresses, deformation (strain) and pore pressure using a variety of constitutive relationships, which include (in order of increasing complexity) linear-elastic, multi-linear elastic, elastic (hyperbolic), elasto-plastic and elasto-visco-plastic.

Numerical models divide the rock mass into finite or discrete elements. Each element is assigned an idealized stress-strain relation and properties that describe how each material behaves. These elements are connected in a continuum model (e.g. boundary element, finite element or finite difference elements), separated by discontinuities in a discontinuum model (discrete elements or discrete fracture

networks), or hybrid models (e.g. combined finite-discrete elements). Jing (2003) and Stead & Coggan (2012) provide detailed descriptions of continuum and discontinuum methods.

Discontinuum models represent the rock mass as an assemblage of discrete blocks, and discontinuities are treated as boundary conditions between blocks (Gigli et al. 2012). The discrete blocks can be rigid or deformable. A termination criterion is required to stipulate whether a discontinuity terminates in the rock mass or against other discontinuities and is fundamental for providing the rock mass with strength derived from intact rock bridges. However, it is practically impossible to represent every discontinuity within a rock mass, so simplifications are required. Holistically, in a practical problem, site investigations and geological uncertainty are a key limitation in precisely representing the geometry and geomechanical characteristics of individual intact rock blocks and discontinuities.

Continuum models assume materials behave isotropically as a continuous body, which are often described using the Mohr-Coulomb or Hoek-Brown failure criteria (Hoek & Brown, 1980). Large discontinuities such as faults are explicitly modelled as interfaces; however, persistent joints, bedding or foliation and other smaller scale discontinuities are implicitly modelled by user defined ubiquitous joint models or directional shear strengths. In continuum models, no actual failure surface is formed; however, an estimation of slip surface location and shape can be predicted by the concentration of shear strain in the model (Sjöberg, 1999). Deformation (strain) in a continuum model is represented as a continuous displacement field, which may not be realistic for large slope deformations.

Numerical simulations require increased computational time and computing power compared with limit equilibrium; and 3D numerical simulations require significantly more time and power than 2D.Irrespective of the modelling method applied, several simplifications are necessary (Sjöberg, 1999) and include but are not limited to the mechanics of the predicted failure mechanism, slope geometry, loads acting on the slope, geology, complexity of geological structure and groundwater. Model calibration is required to ensure that numerical simulations, including many or all of the aforementioned simplifications, produce results that represent actual ground behaviour.

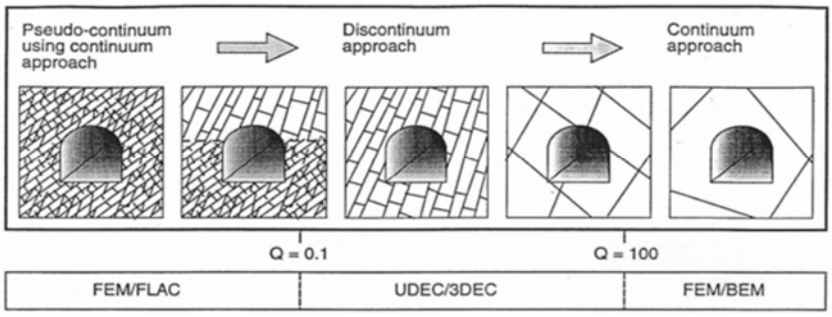



Fig. 11 Conceptual Rock Masses separated by broadly appropriate modelling techniques for underground excavations, approximately 90% of which are likely to require discontinuum treatment (Barton, 2025).

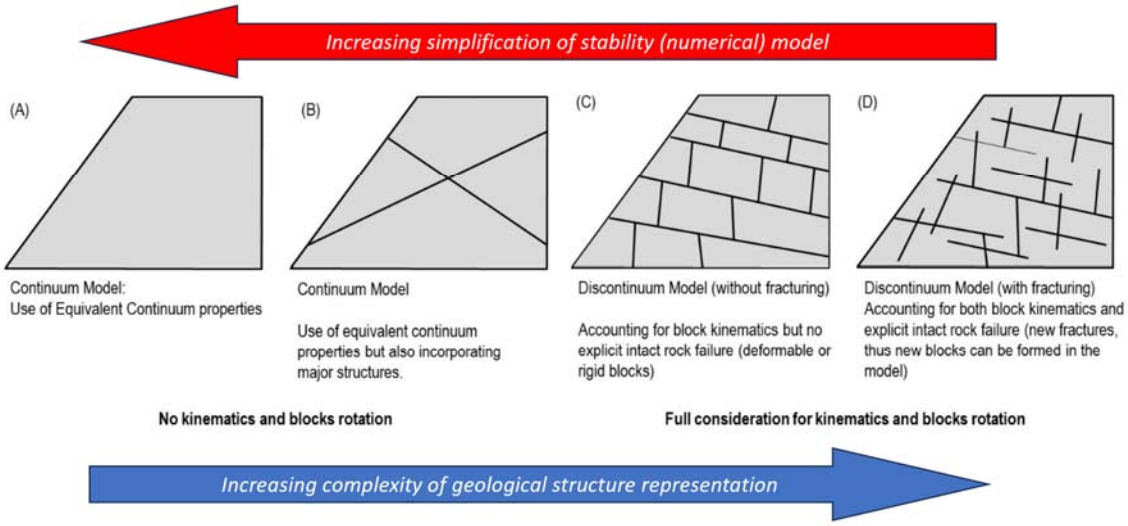



Fig. 12 Example of how continuum modelling trades structural complexity (C and D) for geometrical simplicity (A and B) in rock slopes (Shapka-Fels & Elmo, 2022). Level of simplification required generally increases with slope size (height).

Figures 11 and 12 present examples of typical underground and rock slope excavations modelling with various modelling techniques and levels of simplification. As excavation (or slope) sizes increase, and/or, when discontinuity spacing or fracture frequency increase, so does the need for simplification and use of continuum approaches.

By way of example, GSI, which is a key component of the Hoek-Brown Failure Criterion (Hoek & Brown, 1980; 2019), is commonly used for estimating rock mass strength and stiffness characterization in LEM and various (continuum) numerical models. The Hoek-Brown failure criterion for intact rock is based on a strong set of empirical data. GSI offers an intuitive classification of rock mass quality that uses nomographs (or can be derived from RMR and Q) to downgrade intact rock properties (strength and stiffness) to equivalent rock mass properties in a continuum model, which reduces the need for modelling all discontinuities explicitly (i.e. a simplification of the assemblage of intact rock pieces and discontinuities to a homogeneous and isotropic mass). Limitations of GSI and the Hoek-Brown Failure Criterion include, but are not limited to:

- The Hoek-Brown failure criterion requires that the rock mass behave isotropically, and that failure does not follow a preferred direction imposed by the orientation of a specific discontinuity or a combination of two or three discontinuities. In these cases, the use of GSI is meaningless as the failure is governed by the shear strength of these discontinuities and not of the rock mass (Marinos et al. 2005);
- The GSI system should not be applied to those rock masses in which there is a clearly defined dominant structural orientation (Marinos et al. 2005);
- It is inappropriate to assign GSI values to excavated faces in strong hard rock with a few discontinuities spaced at distances of similar magnitude to the dimensions of the tunnel (Marinos et al. 2005);
- The GSI system assumes that, because the rock mass is made up of a sufficiently large number of joint sets and randomly oriented discontinuities, it can be treated as a homogeneous and isotropic mass of interlocking blocks (Hoek & Brown, 2019).

## 3.2 Conceptual consequences of oversimplification

Failure mechanisms modelled for a rock slope can be significantly dependent upon the input parameters and modelling assumptions that are used. By way of example, the blocky limestone rock mass shown in Figure 13 has been used to conceptually model the failure mechanisms from Figure 6 (i.e. planar, toppling and circular), which are identified depending on the inputs and assumptions that are used.

The blocky limestone is moderately strong with continuous bedding and perpendicular, discontinuous, orthogonal cross-joints as shown in Figure 13. For simplicity, the rock mass has been shown using a 2D cross-section and one of the cross-joints is therefore not considered.

Material properties for the rock mass are shown in Table 1. These consider a continuum approach in which GSI is used to downgrade rock mass strength with the inherent assumption of homogeneous and isotropic conditions (GSI=55); and a discontinuum approach which considers intact blocks (GSI=90) are separated by discrete bedding planes and cross-jointing with different orientations.

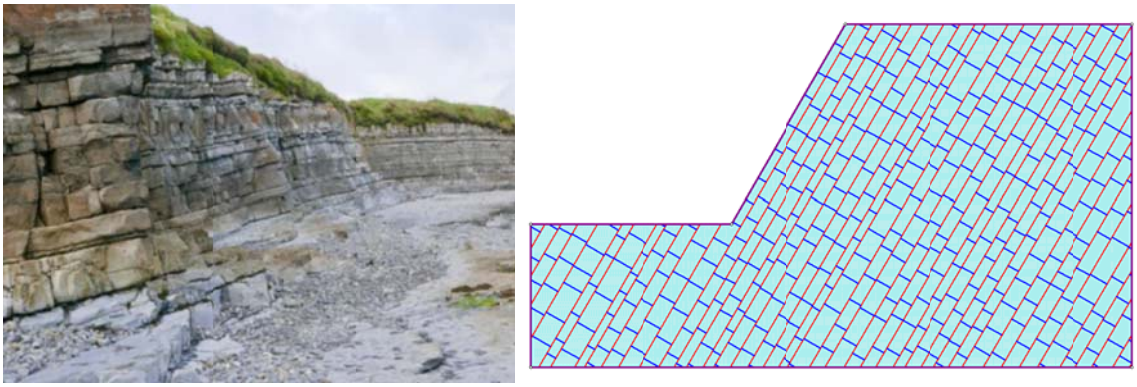


Fig. 13 Blocky limestone rock mass. Left: photograph of rock outcrop; Right: discontinuum model representation of rock mass (note: bedding dip angle different to photograph).

Table 1Material properties for blocky limestone rock mass

| **Parameter** | **Continuum** (Homogeneous, Isotropic) | **Discontinuum** (Heterogeneous, Anisotropic) |
|---|---|---|
| Limestone UCS [MPa] | 50 | 50 |
| Hoek-Brown material constant, $m_i$ | 9 | 9 |
| Limestone Geological Strength Index, GSI | 55 | 90 ^ |
| Damage Factor, D | 0 | 0 |
| Limestone Young's Modulus, $E_m$ [GPa]* | 14 | 80 |
| Limestone Poisson's Ratio | 0.3 | 0.3 |
| Bedding Orientation | - | Variable (refer Fig. 14) |
| Bedding Persistence | - | Continuous |
| Bedding Cohesion, c' [kPa] | - | 20 |
| Bedding Friction Angle, φ [°] | - | 35 |
| Cross-Joint Orientation | - | Perpendicular to bedding |
| Cross-Joint Persistence | - | Discontinuous |
| Cross-Joint Cohesion, c' [kPa] | - | 0 |
| Cross-Joint Friction Angle, φ [°] | - | 40 |
| Bedding/Cross-Joint Normal Stiffness, $k_n$ [MPa/m] | - | 100,000 |
| Bedding/Cross-Joint Shear Stiffness, $k_s$ [MPa/m] | - | 10,000 |
| Fault Zone Cohesion, c' [kPa] | 0 | 0 |
| Fault Zone Friction Angle, φ [°] | 20 | 20 |
| Fault Zone Young's Modulus, $E_m$ [GPa] | 1 | 1 |
| Fault Zone Poisson's Ratio | 0.3 | 0.3 |

^ *Assumes intact rock (massive) with possibility of random joints.*
** Based on simplified approach by Hoek & Diederichs (2006).*
*Note: daylighting fault zone only applied in Figure 15 (shown in pink).*

2D Finite Element Method (FEM) analysis was performed in RS2 software (Rocscience Inc) on the continuum model and three discontinuum models each with different bedding and cross-jointing orientations to demonstrate different failure mechanisms. As illustrated in Figure 14, the four models attained vastly different results:

- Failure mechanisms included circular failure when a continuum approach was used, to daylighting and non-daylighting planar sliding and toppling mechanisms depending on the bedding (anisotropy) orientation.
- Factors of Safety (Critical SRF) ranged from 1.3 for a daylighting to 8.3 for a non-daylighting planar failure mechanism. Circular failure attained a Factor of Safety of 3.5.
- Variation in terms of Factor of Safety from the continuum model was -31% to 137% depending on the failure mechanism that was modelled.

The ability to model vastly different failure mechanisms and attain variation in Factors of Safety for a relatively simple rock mass should immediately highlight the need for engineering geological judgement and critical thought when considering model inputs and interpreting the validity of modelling results.

The model results in Figure 14 also highlight that both planar failure (daylighting) and toppling mechanisms have lower Factors of Safety than those attained by continuum modelling for circular failure, i.e. continuum approaches may *not* be conservative depending on the engineering geological conditions encountered.

In Figure 15, an adversely oriented (daylighting) fault zones was included in the same four models initially shown in Figure 14. With the inclusion of a major structure (fault zone), the models identified:

- Planar failure with sliding through the fault zone.
- Factors of Safety (Critical SRF) ranged from 0.55 to 0.74, i.e. all unstable.
- Variation in terms of Factor of Safety from the continuum model was only -26% to 22%. Further refinement through improved meshing and larger lateral model extents would likely reduce the variation in this model further.

The models in Figures 14 and 15 demonstrate that in instances where adversely oriented, major geological structures greatly influence stability, the simplification of smaller discontinuities such as bedding planes and joints to a continuum approach has a limited effect on global stability. However, in

cases where major structures are not the key influencer of stability, the smaller discontinuities and their orientations have a significant impact upon results, and simplification to a continuum approach can have a significant impact on the model outputs.

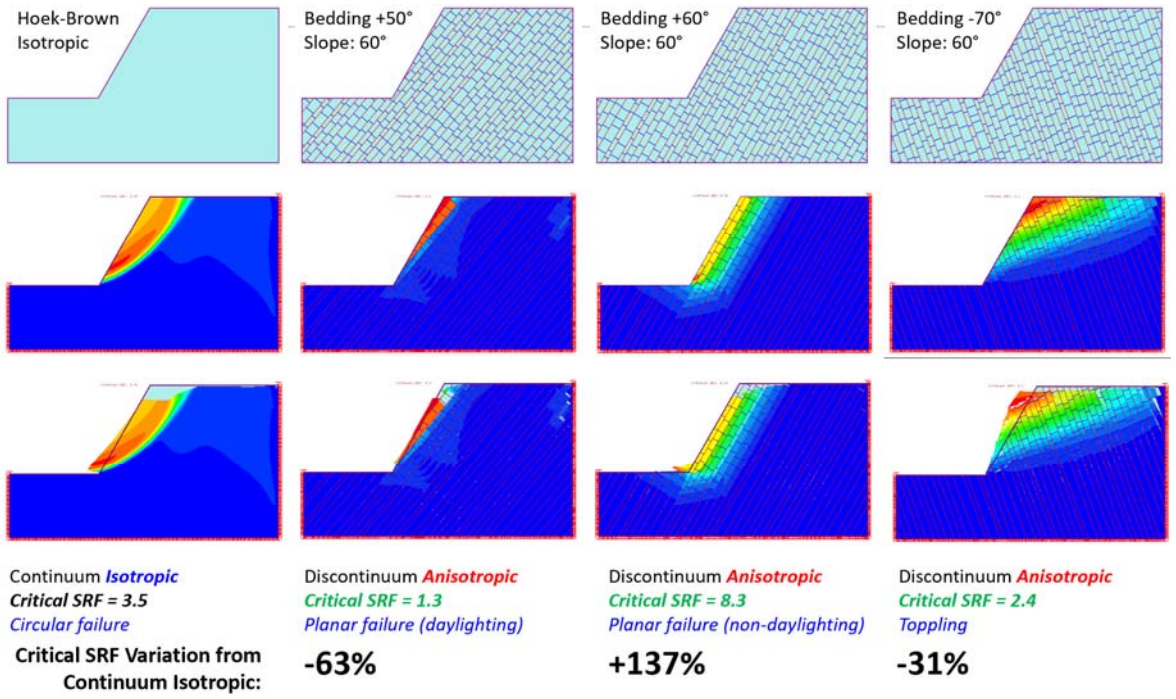



Fig. 14 Comparison of 2D FEM modelling results for a 60°,70 m high slope in a blocky limestone rock mass depending on the assumed engineering geological conditions, different levels of simplification, subsequent failure mechanisms and Factors of Safety (Critical SRF). From left to right: isotropic conditions modelling circular failure (similar to 'A' in Fig. 12); anisotropic conditions modelling daylighting planar failure; anisotropic conditions modelling non-daylighting planar failure; anisotropic conditions modelling toppling (similar to 'D' in Fig. 12 although simplified in terms of constitutive models).

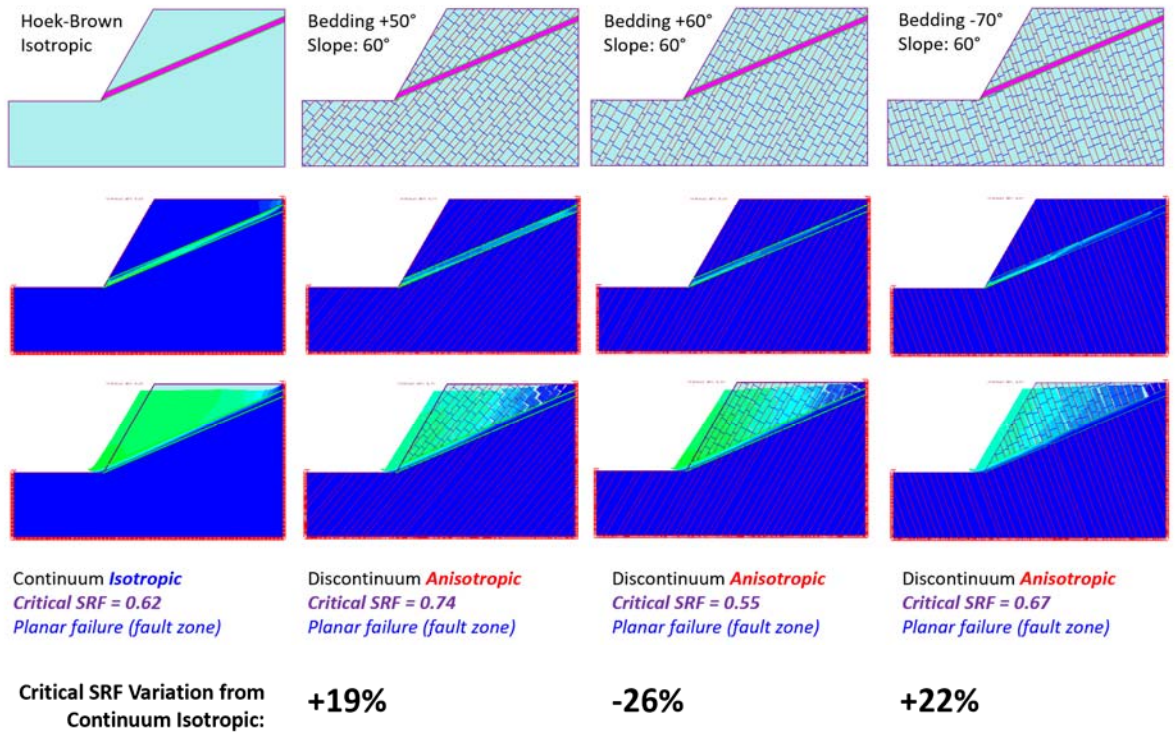



Fig. 15 Comparison of 2D FEM modelling results for a 60°, 70 m high slope in a blocky limestone rock mass undercut by a daylighting geological fault zone (pink) with assumed engineering geological conditions, different levels of simplification, and Factor of Safety (Critical SRF). From left to right: isotropic (similar to 'B' in Fig. 12) and anisotropic (similar to 'D' in Fig. 12) conditions all predicting planar failure along fault zone.

The same principles can be applied to modelling underground excavations to understand failure mechanisms, predict displacements and the appropriateness of ground support.

By way of example, Figure 16 shows modelled displacements for Gjøvik Cavern in Norwayusing continuum and discontinuum approaches. The results showed:

- Continuum model with a large zone of displacement and potential for a voluminous stress induced failure (failure mechanism #4 in Figure 9). Maximum displacement of 8.8 mm.
- Discontinuum model with localized zones of displacement in the roof and potential for discontinuity-controlled block falls (failure mechanism #2 in Figure 9). Maximum displacement of 4.2 mm.

Continuum predicted twice the maximum displacement of discontinuum. The discontinuum approach attained similar results to historicdiscrete element method modelling in UDEC software (Itasca) by Barton et al. (1994), which was validated by monitored displacements from borehole extensometers after construction.

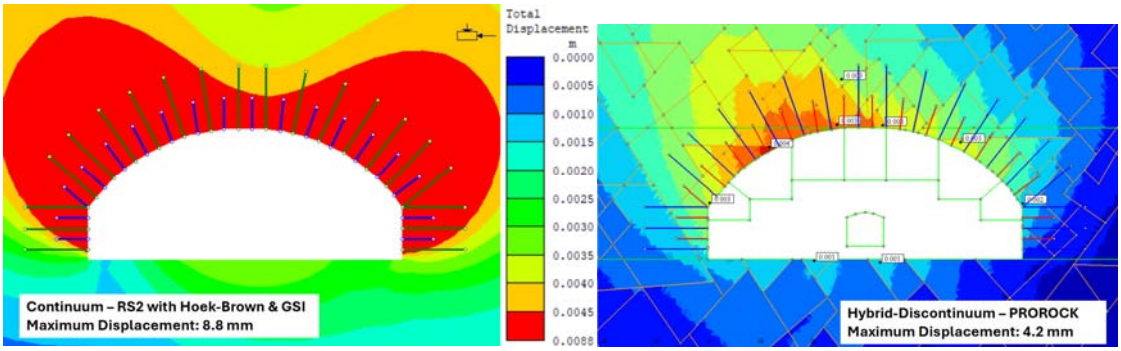



Fig. 16 Comparison of Gjøvik Cavern (62m Span) modelling results showing predicted displacements. Left: 2D FEM continuum, homogeneous and isotropic in RS2 software). Right: 2D Finite-Discrete Element Method (FDEM: hybrid discontinuum approach in PROROCK software by Scientia) Continuum modelling predicts double the maximum displacements and a substantially larger zone of displacement than the discontinuum approach (Bar, 2025).

A key limitation of the examples shown in Figures 14 to 16 is that only 2D analysis was performed (i.e. simplification into two dimensions). Inclusion of 3D analysis could identify more complex failure mechanisms in three-dimensions, which could be more critical than those identified in 2D.

It should also be noted that the number of material properties required to undertake discontinuum analysis is considerably more than for continuum (as shown in Table 1). As such, more effort is required in site investigations and characterization.

## 3.3 Real world consequences of oversimplification

In the 2020s, modelling circular failure mechanisms for rock slopes appears to be increasingly popular, despite the rarity of circular failures in the real world (i.e. 3 to 10% of rock slope failures).

By way of example, Figure 17 shows recent academia and software developments that have focused on modelling circular failure for large rock slopes. All examples assume homogenous and isotropic ground conditions and simplify the rock mass to a continuum (A in Figure 12) and aim to attain similar FoS outputs precise to two decimal places. These include using LEM and FDM to develop and validate slope design charts (e.g. slope height versus angle by Styles & Vakili, 2020) and spreadsheet tools (for predicting FoS by Carranza-Torres, 2021), which only consider circular failure mechanisms. It also includes software developments to enable manipulation of the damage factor (D) in the Hoek-Brown failure criterion to model shallower circular slip surfaces in 2D LEM (Ma et al. 2022).

Notwithstanding the abovementioned tools' usefulness for assessing the potential of circular failure, their popularity is disproportional to their relevance in terms of real world rock slope behaviour.

The same applies for underground excavations where simplification of a rock mass to continuum approach for modelling is common practice.These simplifications, in many instances, may have significant consequences for real world applications as is demonstrated through the following cases studies.

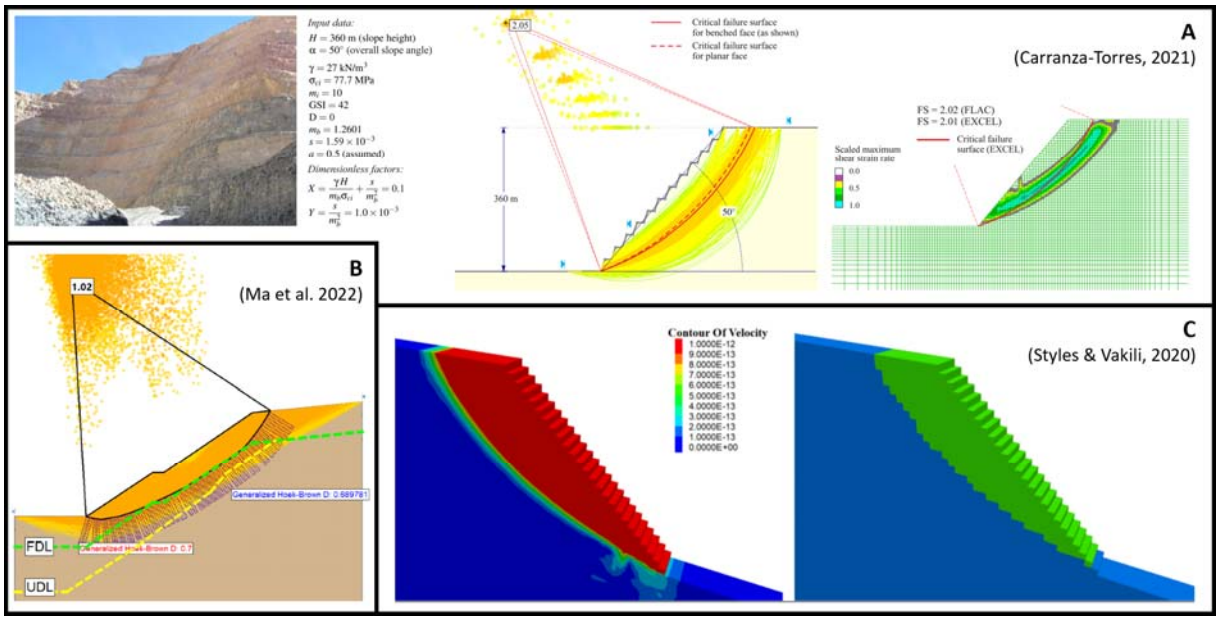



Fig. 17 Recent popularity of modelling circular failure mechanisms for rock slopes despite their rarity in the real world (only 3 to 10% of rock slope failures are 'circular' in previously mentioned databases). A: 200m high rock slope (UCS = 77 MPa) with high FoS modelled in 2D LEM (Slide2) and 2D FDM (FLAC) by Carranza-Torres (2021). B: Circular failure modelled in 2D LEM (Slide2) despite the use of gradational damage factor (D) manipulation with depth to attain FoS of approx. 1.00 (Ma et al. 2022). C: Circular failure modelled for 200m high rock slope using 3D FDM (FLAC3D with SlopeX plug-in) by Styles & Vakili (2020).

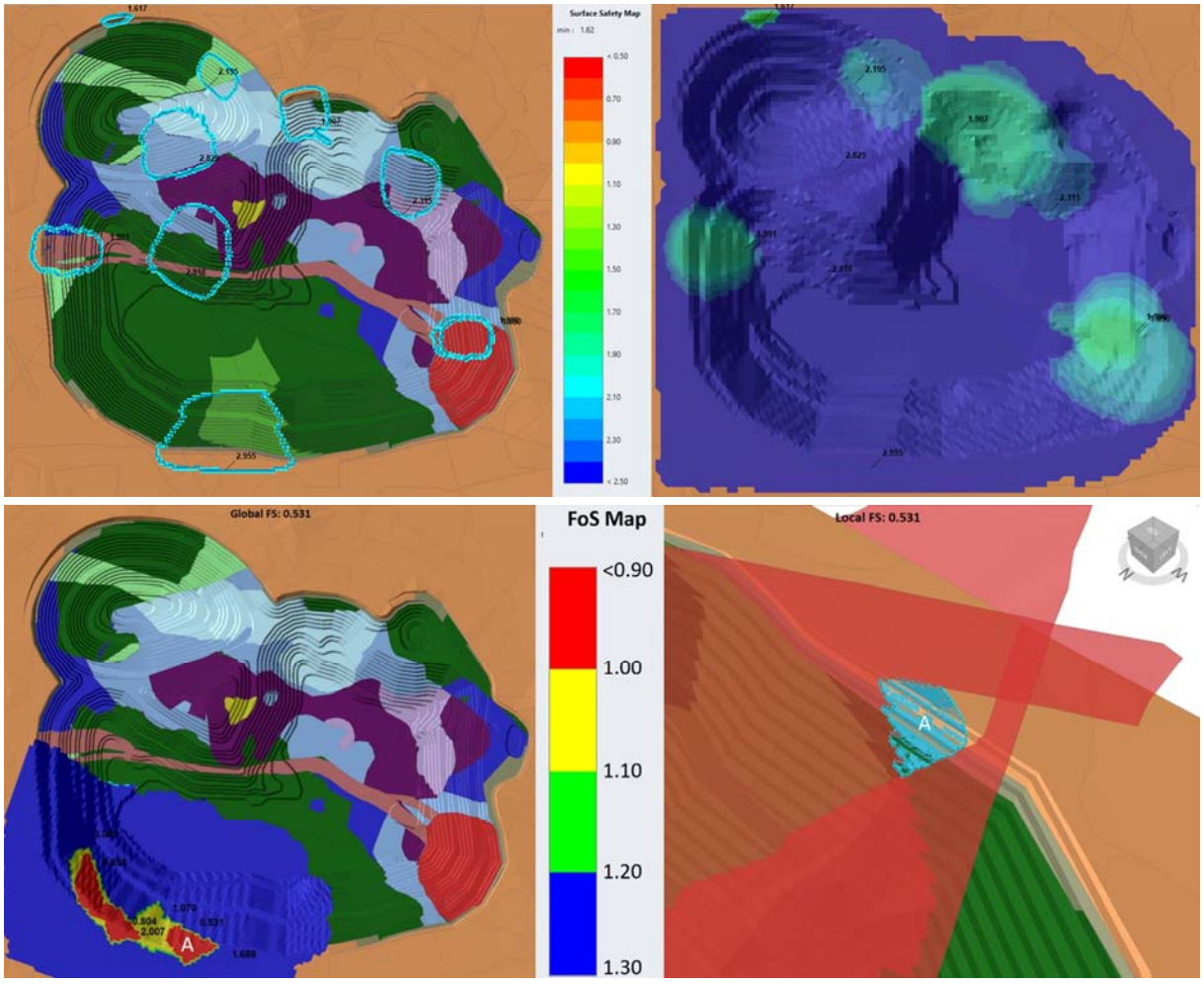



Fig. 18 3D LEM analysis of Aktogay Open Pit Copper Mine (Bar & Zlobin, 2024). Top: Faults omitted from analysis: Factor of Safety exceeds 1.50 for all slope sectors. Bottom: Faults included in south-west quadrant for detailed analysis: Factor of Safety is less than 1.00 for three individual wedges and approximately 1.07 for a fourth, larger wedge.

### 3.3.1 Pit slope stability modelling with and without faults

Failing to assess the potential for a large range of failure mechanisms, including complex ones can result in inappropriate stability analyses being undertaken and resulting in inappropriate designs. By way of example, Figure 18 presents the results of two sets of three-dimensional (3D) LEM analyses for a proposed design of Aktogay copper mine, with and without the inclusion of geological faults (Bar and Zlobin, 2024).

The geological faults are oblique to the slope in the south-west quadrant and form low Factor of Safety (FoS) wedges; which are expected to be unstable. Without the inclusion of faults, the analyses only check for circular and ellipsoidal rotational failure mechanisms through the (isotropic) rock mass and indicate a high FoS (overestimated by up to 50%) with apparent (but non-existent) potential for slope steepening.

If 2D LEM were to be undertaken, as appears to be most commonly practiced according to McQuillan et al. (2020), the modelled failure mechanism would have been planar sliding instead of wedges; which is intrinsically incorrect and would have different stability modelling results yet again.

### 3.3.2 Heap leach pad stability modelling using LEM and FEM

For rock fill structures, such as a waste rock dump or heap leach pad, assessment of failure mechanisms and the selection of appropriate stability analysis techniques is critical. In the author's industry experience, 2D LEM is most commonly used for modelling heap leach pad stability.

Figure 19 illustrates how 2D and 3D LEM analyses attained high FoS while modelling sliding along a constructed liner and shearing through the Heap Leach (HL) fill material (Bar et al. 2025). In comparison, 2D and 3D Finite Element Method (FEM) was able to identify active-passive block sliding along the liner. In this case, 2D LEM analysis results overestimated FoS by over 35% compared to the 3D FEM results.

As mentioned earlier, LEM is unable to model active-passive blocks. Such profound differences in FoS should be considered very consequential, particularly in light of the recent heap leach failures in Türkiye and Canada.

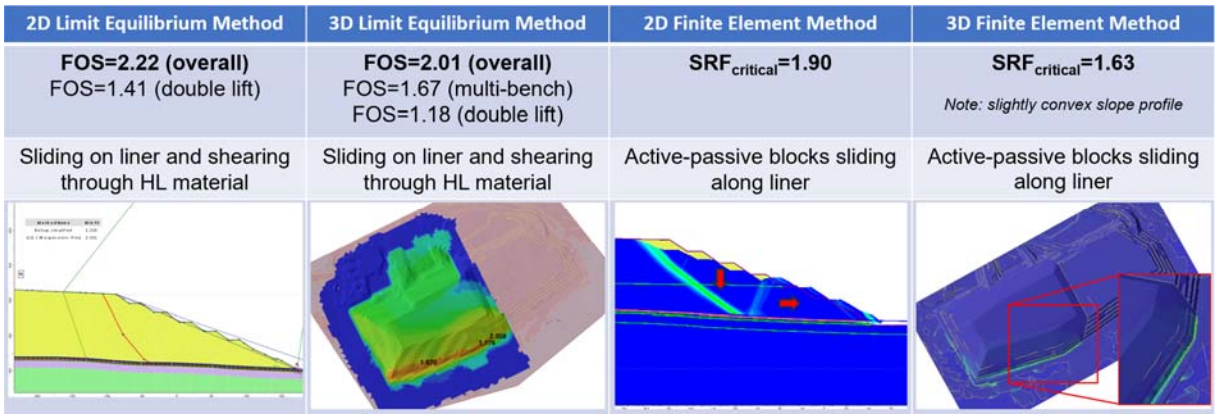



Fig. 19 2D and 3D LEM and FEM analysis of Aktogay Heap Leach Pad (Bar et al. 2025). From left to right: decreasing modelled Factor of Safety and differences in identified failure mechanism.

In the examples shown in Figures 18 and 19, not considering the potential failure mechanisms based and undertaking appropriate stability analyses would have resulted in overestimation of Factors of Safety and could have provided an unachievable and unsafe design.

### 3.3.3 Drainage tunnel modelling: continuum versus discontinuum

A drainage tunnel in Papua New Guinea was proposed in a blocky to very blocky siltstone rock mass (Bar & Baczynski, 2018). As shown in Figure 20, the siltstone has sub-horizontal bedding with two to three sets of sub-perpendicular joints. Bedding and joint conditions are generally fair with smooth and slightly altered joint walls.

Q values range from 1 to 4, and GSI typically ranges from 40 to 55.In-situ stresses are low with the tunnel depth not exceeding 300 m ($\sigma_1/\sigma_c<0.12$), and intact rock strength, $\sigma_c$, is in the order of 70 MPa.



Fig. 20 Very blocky siltstone. Left: fresh siltstone in existing exploration tunnel (3 m wide x 3.5 m high). Right: Slightly weathered siltstone outcrop (approx. 2m high in photograph).

Table 2 Material properties for very blocky siltstone rock mass

| **Parameter** | **Continuum** (Homogeneous, Isotropic) | **Discontinuum** (Heterogeneous, Anisotropic) |
|---|---|---|
| Siltstone UCS [MPa] | 70 | 70 |
| Hoek-Brown material constant, $m_i$ | 10 | 10 |
| Siltstone Geological Strength Index, GSI | 45 | 100 ^ |
| Damage Factor, D | 0 | 0 |
| Siltstone Young's Modulus, $E_m$ [GPa] * | 9 | 40 |
| Siltstone Poisson's Ratio | 0.26 | 0.26 |
| Bedding/Joint Normal Stiffness, $k_n$ [MPa/m] | - | 100,000 |
| Bedding/Joint Shear Stiffness, $k_s$ [MPa/m] | - | 10,000 |

^ *Assumes intact rock (massive).*
* *Based on simplified approach by Hoek & Diederichs (2006).*

Numerical methods were used to assess drainage tunnel behaviour using two approaches with 2D FEM (RS2 software). Table 2 summarizes the material properties used for elastic analysis to understand potential failure mechanisms for an unsupported tunnel. Figure 21 shows modelled displacements for the drainage tunnel using continuum and discontinuum approaches, which indicate:

- Continuum model with a relatively large zone of displacement and potential for a shallow failure (failure mechanism #3 in Figure 9). Displacements exceed 20 mm up to a depth of approximately 1 m behind the side walls and below the floor.
- Discontinuum model (intact blocks separated by discontinuities) with localized zones of displacement in the roof and potential for discontinuity-controlled block falls (failure mechanism #2 in Figure 9). Maximum displacement of 3 mm at a depth of approximately 1 m behind side walls and roof.

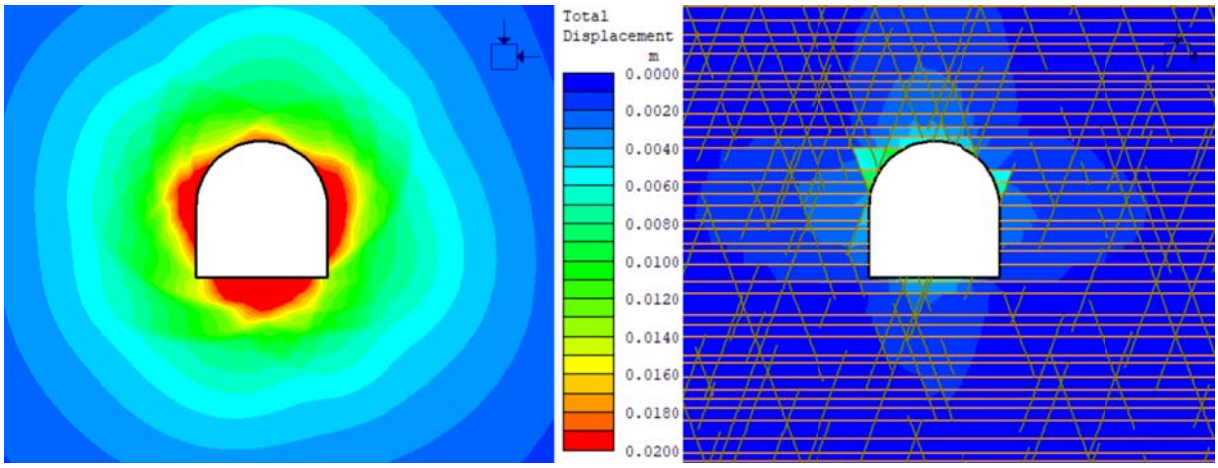



Fig. 21 Unsupported Drainage Tunnel (5 m wide x 5.5 m high) 2D FEM Elastic Model Results in RS2 software showing Total Displacement (Bar, 2025). Left: Continuum, homogeneous & isotropic model outputs showing potential for shallow failure with over 20 mm displacements up to a depth of approx. 1 m behind side walls and below floor. Right: hybrid discontinuum model outputs showing potential for discontinuity-controlled block falls and less than 3 mm displacements at a depth of approx. 1 m behind side walls and roof.

Ground support requirements based on the continuum model (combination of shotcrete, mesh and bolting) would be significantly higher than predicted by the hybrid discontinuum model, which indicates only a need for spot bolting and mesh for small block fall protection. Based on the observed excavation performance in the existing exploration tunnel (Fig. 20), the hybrid-discontinuum model outputs are significantly more realistic than the continuum model.

# 4 Residual risk management

As a result of the inherent geological uncertainty in rock masses and the simplification that occurs when modelling geomechanical problems, residual risks remain present when excavations commence with the potential for socioeconomic impacts of ground failure events.

Residual risks need to be managed through the use of a continuous improvement framework (e.g. Figure 5) and critical control measures that are developed and implemented during study phases and during and after excavation.

Critical control measures are the equipment, systems, procedures, and policies that an organization uses to prevent injuries and death (Ross, 2017). Critical control measures can also be used for minimizing other socioeconomic impacts of rock slope and underground excavation failures such as damage to equipment, infrastructure, business disruptions, and the impairment or sterilization of mineral resources.Figure 22 presents a framework of five critical control measures are often used for managing ground failure risks (Bar et al. 2022).

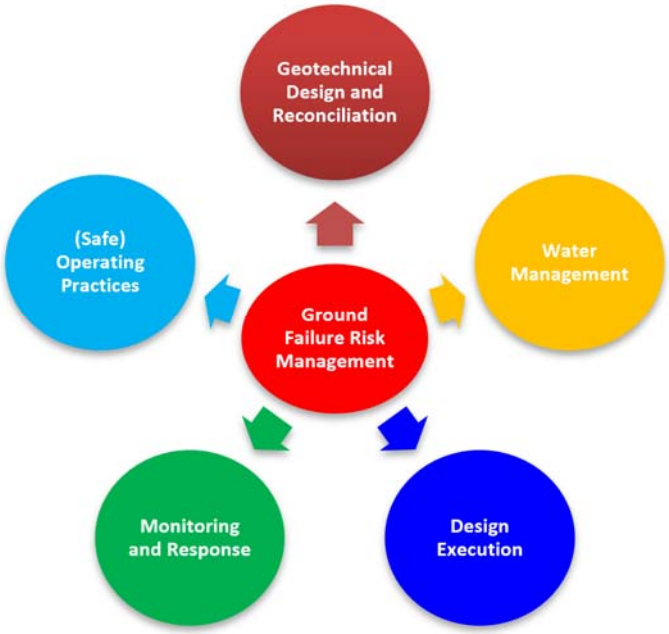



Fig. 22 Ground Failure Risk Management Critical Control Measures.

Geotechnical design involves the use of geological, rock mass, structural and hydrogeological data to develop a geotechnical model. This geotechnical model forms the basis of stability analyses, which are used to design a pit slope with acceptable factors of safety or probabilities of failure. Geotechnical models are often developed from limited data and inherently contain various degrees of uncertainty and risk.The reconciliation aspect of this control measure requires ongoing validation of the geotechnical model as the excavations advance to check whether geotechnical model assumptions remain valid and whether uncertainties pose a risk or provide economic opportunity. Elements of geotechnical reconciliation include the validation of excavation geometries (e.g. bench width capacity for rock fall risk management or overbreak and underground excavation spans), local geology, intact rock strengths, rock mass classifications, discontinuity orientations and conditions, presence and condition of major geological structures, pore pressures and displacement monitoring results.Geotechnical design and reconciliation is an engineering control measure that is particularly effective for managing larger failure mechanisms. It also assists in mitigating risks associated with smaller rock falls.

Water management involves a combination of groundwater extraction through pumping and surface water management to minimize water ingress to facilitate safe and productive operations (drilling, blasting and excavation). For slopes, dewatering also assists in depressurization (i.e. reducing pore pressure in the slopes), which is further achieved through the installation of horizontal drains on each pushback. Water management is an engineering control measure that is particularly effective for managing slope failures. It has a limited effect on mitigating risks associated with smaller rock falls.

Design execution focuses on achieving the correct geometric design, installing water drainage and minimizing damage to the rock mass from blasting. Operating practices focus on ensuring all personnel working on the project are aware of geotechnical hazards, how to work safely during normal operating conditions (e.g. never working under unsupported ground in underground excavations; a minimum safe distance of 15 meters away from slopes without a job hazard analysis or JHA). Design execution and operating practices are a combination of engineering and administrative control measures, respectively. They are primarily effective for managing risks associated with rock falls and smaller scale failure mechanisms. This control measure alone is not considered effective for large scale failure mechanisms (e.g. overall slope failure, crown pillar failure in an underground mine).

Monitoring and response may use an array of instrumentation to identify changes in ground displacements, pore pressure and excavation induced seismicity. This instrumentation and visual observations may be used to identify potential hazards. Trigger-action-response-plans (TARPs) can be developed to respond to potential hazards and may include temporary evacuations of the at-risk area. Although highly sophisticated instrumentation is used to identify emerging threats, this control measure remains administrative as it relies on personnel to identify the threat, respond and communicate it to the personnel, who are then required to act accordingly. Monitoring and response has been used for primarily for managing safety risks associated with slope failure and rockburst.

# 5 Discussion and conclusions

Many excavations in rock (slopes and underground) are not designed in detail; some are not designed at all and are merely excavated under the guidance of a project supervisor or equipment operator. Many excavations that appear to have a detailed design, may have significant geological uncertainty or include excessive simplification in stability predictions and may not assess failure mechanisms appropriately.

Current industry practitioners, academia and software developer trends relating to stability modelling appear to focus on attaining a precise Factor of Safety value. In the case of rock slopes, more often than not, stability models assume circular or quasi-circular failure modes in the analysis, rather than attempting to understand and model actual or more realistic failure mechanisms where FoS may be substantially different since circular failures account for less than 10% of real world rock slope failures. In many such instances, the model outputs and subsequent designs would largely be wrong since the plausible failure mechanisms were not investigated.

Alejano (2025) reviewed the application of rock mass classification systems, including their misuse. A similar review of case studies and published articles on the misuse rock slope and underground excavation stability modelling, including over-simplification and incorrect failure mechanism appraisal is warranted and strongly encouraged by the author. Such an undertaking should include academics and practitioners from various institutions around the world.

With inherent geological uncertainty, it is possible, even probable that geological anomalies such as faults were not identified prior to excavation. By adopting a continuous improvement framework and updating models throughout the design and excavation process, it is more likely that the anomalies would be identified and associated ground failure risks could be managed than if the models were completely static. It may perhaps be beneficial to start from the viewpoint that *'all models are wrong, but some are useful'*; planning for contingency and then focusing efforts to validate the models to determine the most appropriate or applicable results.

Rock engineers should be encouraged to think critically and assess potential failure modes more deeply rather than merely following a design workflow from point A to point Z. '*It is better to be approximately right than precisely wrong*'. The use of probabilistic analysis is often beneficial, yet the ability to include a statistically-derived location and orientation of a geological anomaly (e.g.

unfavourably oriented fault) remains out of reach. As such, the use of plausible sensitivity analyses is strongly encouraged.

The number of experienced rock engineers appears to be decreasing over the last decade with a significant drop in university graduates from the discipline. The demand for rock engineering projects requiring numerical methods is increasing, which means less experienced engineers with more projects around the world. At the same time, access to easy-to-use numerical modelling software has become ever more abundant, i.e. more engineers are applying numerical methods, and often they have less experience than their predecessors. Many engineers using numerical modelling methods have little or no field experience observing real world ground behaviour. Less experienced engineers, higher demand for stability modelling, and increased project throughput using prescribed design workflows will likely lead to incorrect assessments of failure mechanisms.

In order to minimize the risk and consequence of ground failure events relating to rock slopes and underground excavations, realistic failure mechanisms need to be well understood. By focusing to understand them in different engineering geological settings and simplifying problems only as much as is reasonable, future designs of rock slope and underground excavations may become safer and more economically attractive.

Lastly, for real world projects there are always time and budget constraints. Significant effort should be made by rock engineers during all project phases from initial designs to excavation and asset operation, to explain in economic terms (as well as safety), to project managers, directors and investors; the risks and consequences of inadequate design of rock slopes and underground excavations and the geological uncertainty that remains present if a continuous improvement framework and critical control measures for managing ground failure risks are not implemented.

## Acknowledgements

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