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On the use of numerical methods to assess underground excavation stability: continuum versus discontinuum and hybrid approaches

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Abstract: Empirical methods for characterizing rock exposures, drill core, and tunnels under construction, such as RMR and Q, have developed from 50+ years of experience and case records. Both methods continue to be routinely used for the selection of ground support for tunnels, caverns and in underground mines around the world. Through progressive improvements in computing capability and availability of software over the last two decades, numerical modelling techniques such as finite element analysis have increased in popularity in recent years. Initially, numerical methods were applied by experts with significant experience in computing, geomechanics, and the design and construction of underground excavation; however, in recent years with generational change, a higher demand for numerical modelling and reducing number of geotechnical experts, numerical modelling in many cases is being more frequently undertaken by engineers with very limited field and operational experience. When used thoughtfully and appropriately in terms of representing engineering geological and geotechnical conditions, numerical models are powerful tools for predicting ground behaviour and the effectiveness of support measures. However, when applied thoughtlessly or misused, numerical model outputs often present incorrect and inappropriate results with 'perceived confidence' to project managers and directors. This paper uses case studies to demonstrate appropriate and inappropriate numerical methods for different rock masses using continuum, discontinuum and hybrid approaches to assist engineers to improve the reliability and usefulness of their models.

Keywords: underground excavations; numerical methods; continuum; discontinuum; hybrid

1. Introduction

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 (2024) undertook a global study on the use of 37 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 and Brown, 1997; Marinos, 2019).

GSI is a key component of the Hoek-Brown Failure Criterion (Hoek and Brown, 1980; 2019), which is used for estimating rock mass strength and stiffness characterization in various 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 and can also be derived from RMR and Q. 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

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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 and Brown, 2019).

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. In many cases, linear-elastic analysis is adequate; however, non-linear constitutive relationships should be applied if the focus of the analysis is deformation, not stress distribution (Duncan, 1996).

As conceptualized in Figure 1, numerical models can 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, BEM; finite element or finite difference elements, FDM/FLAC), separated by discontinuities in a discontinuum model (discrete elements or discrete fracture networks), or hybrid models (e.g. combined finite-discrete elements, DEM/UDEC). Jing (2003) and Stead and Coggan (2012) provide detailed descriptions of continuum and discontinuum methods and Mahabadi et al. (2012) discusses a type of hybrid finite-discrete element (FDEM) approach.

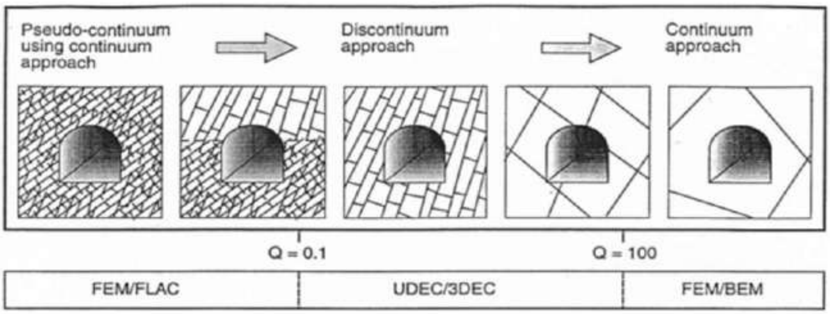


Fig. 1. Conceptual Rock Masses separated by broadly appropriate modelling techniques, approximately 90% of which are likely to require discontinuum treatment (Barton, 2025).

Continuum models assume materials behave isotropically as a continuous body. 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 inferred from the concentration of shear strains (or sometimes, stresses) in the model. Deformation (strain) in a continuum model is represented as a continuous displacement field, which may not be realistic for large slope deformations (Sjöberg, 1999).

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.

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 and complexity of geological structure. Model calibration is required to ensure that numerical simulations, including many or all of the aforementioned simplifications, produce results that represent actual ground behaviour.

Numerical models in 3D require significantly more computational time and computing power compared to 2D. In this paper, only 2D numerical models are examined. It should be noted from the outset that simplifying to two-dimensional cross-sections inherently creates significant limitations, particularly when anisotropic or faulted rock masses are under consideration.

2. Predicting the Behaviour of Underground Excavations

As part of an underground excavation design procedure (Figure 2), three key steps are usually followed (Schubert et al., 2003):

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

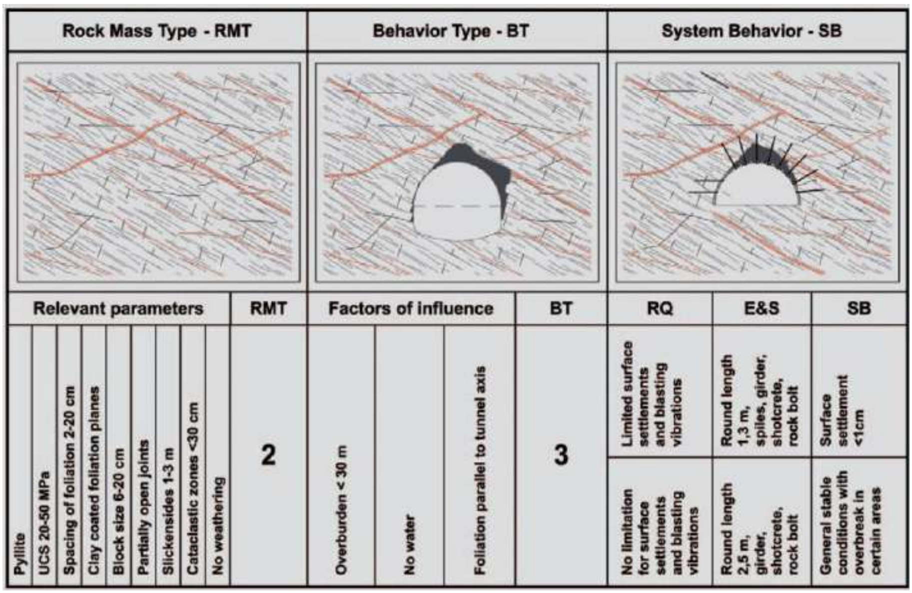
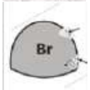
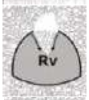


Fig. 2. Example of the underground excavation design procedure (Schubert et al. 2003).

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. Table 1 summarizes eleven general categories that most-commonly occur. Such can be conceptualized using sketches as shown in Figure 2, and later evaluated using numerical methods, which can then form the basis for determining excavation requirements and ground support systems.

Table 1. General Underground Excavation Behaviours and Potential Failure Mechanisms (after Schubert et al. 2003; Oke et al. 2013).

Behaviour Types (BT)			Description of Potential Failure Mechanisms during Excavation of the Unsupported Ground
Stable	1		Stable ground with the potential of small, local, gravity-induced falling or sliding of blocks.
Discontinuity-controlled block fall	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.
Shallow failure	3		Shallow stress-induced failure in combination with discontinuity- and gravity-controlled failure.
Voluminous stress induced failure	4		Stress-induced failure involving large ground volumes and large deformation (i.e. squeezing ground).
Rock burst	5		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.
Buckling	6		Buckling and anisotropic strain of rocks with a narrowly-spaced discontinuity set, frequently associated with shear failure.
Crown failure	7		Voluminous overbreak in the crown with progressive shear failure (also termed chimney type failure).
Ravelling ground	8		Ravelling of dry or moist, intensely fractured, poorly interlocked rocks or soil with low cohesion.
Flowing ground	9		Flow of intensely fractured, poorly interlocked rocks or soil with high water content.
Swelling ground	10		Time dependent volume increase of the ground caused by physical-chemical reaction of the ground and water in combination with stress relief.
Frequently changing behaviour	11	N/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).

Hoek and Brown (1980) presented concepts, which have been developed over 40+ years to relate intact rock strength (UCS or σ_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 3). 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.

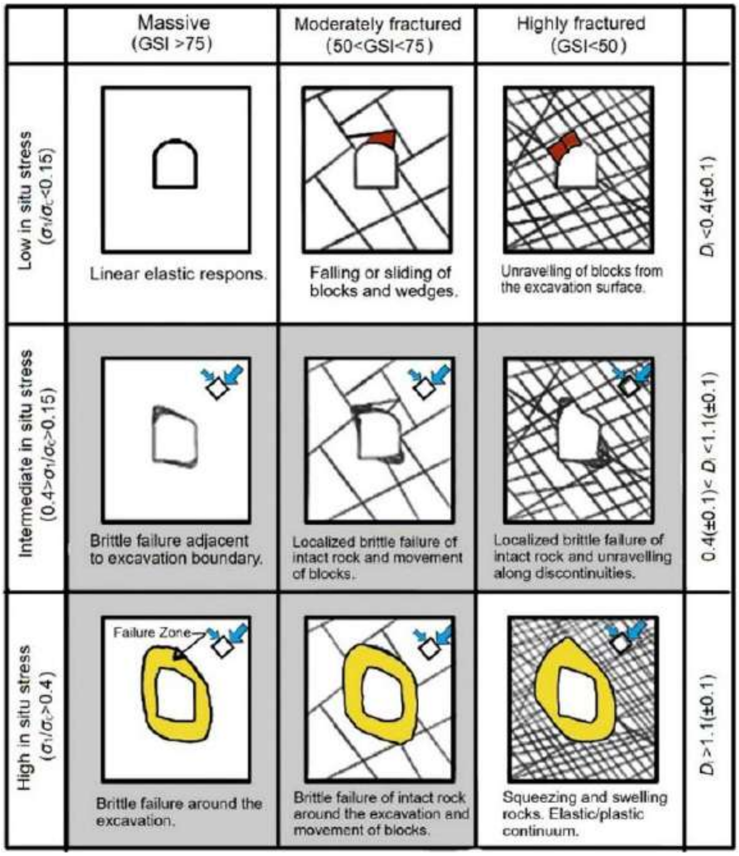


Fig. 3. Potential Failure Modes based on the Ratio of In-Situ Stress and Intact Rock Strength and GSI (after Askaripour et al. 2002; Castro et al. 2012; Hoek and Brown, 1980; Martin et al. 1998).

The continued use of rock mass classifications, such as RMR or the Q-system, for assessing excavation stability and ground support requirements remains invaluable, particularly for validating the appropriateness of numerical modelling outputs prior to the commencement of excavations (i.e. when validation will be possible).

3. Numerical Methods for Assessing Underground Excavation Stability

Case studies are presented to demonstrate the effects of different numerical methods including:

- Continuum assuming homogeneous and isotropic conditions using GSI and the Hoek-Brown Failure Criterion with 2D Finite Element Method (FEM) software, RS2 by Rocscience Inc.
- Hybrid-continuum using the Hoek-Brown Failure Criterion for intact rock and a network of discontinuities to enable heterogeneity and anisotropy with 2D FEM software, RS2.
- Hybrid-discontinuum using 2D Finite-Distinct Element Method (FDEM) software, PROROCK by Scientia.
- Discontinuum using 2D Distinct Element Method (DEM) software, UDEC, by Itasca Inc.

Over the last 20 years, the use of 2D continuum models assuming homogenous and isotropic rock mass conditions by adopting the Hoek-Brown failure criterion and GSI has become very common for tunnel and underground excavation design studies. Barton (2023) indicated that a generational change, persuaded by the ease of software use, has promoted the use of such 2D continuum modelling.

3.1. Case Study I: Drainage Tunnel in Papua New Guinea

Case study I involves the development of a 5m x 5m drainage tunnel in Papua New Guinea (Bar and Baczynski, 2018). As shown in Figure 4, the siltstone rock mass is blocky to very blocky, and 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 (σ_1/σ_c<0.12), and intact rock strength, σ_c, is in the order of 70 MPa.

Potential failure modes are generally expected to involve discontinuity-controlled block falls (considering Fig. 3), which have also been observed in an existing exploration tunnel.

Fig. 4. 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).

Numerical methods were used to assess drainage tunnel behaviour using two approaches using 2D FEM software, RS2, with a summary of material properties shown in Table 2:

- Continuum assuming homogeneous and isotropic conditions (CHI) using the estimated GSI and the Hoek-Brown Failure Criterion. Young's Modulus (E_m) for the rock mass was estimated to be 9 GPa based on the approach by Hoek and Diederichs (2006). Hoek-Brown Damage Factor, D, was set to zero (note: this significantly improves strength and stiffness, i.e. reduces modelled displacements).
- Hybrid-continuum (HC) using the Hoek-Brown Failure Criterion for intact rock (GSI set to 100) and a network of discontinuities to enable heterogeneity and anisotropy with 2D FEM software, RS2. E_m was estimated using Hoek and Diederichs (2006) to be 40 GPa (i.e. equal to the intact rock modulus, E_i).

Table 2. Material Properties for Drainage Tunnel Numerical Models.

Numerical Method	UCS (MPa)	m_i	GSI	D	E_i (GPa)	E_m (GPa)	ν	Joint Stiffness (MPa/m)	
								k_n	k_s
CHI	70	10	45	0	40	9	0.26	N/A	N/A
HC	70	10	100	0	40	40	0.26	100,000	10,000

Elastic numerical modelling outputs are shown in Figure 5 and indicated vastly different results between the CHI and HC numerical methods:

- Behaviour type predicted by CHI was a potential for voluminous stress induced failure (BT 4, Sq.) whilst HC predicted discontinuity-controlled block falls (BT 2, Wg.).
- CHI predicts a modelled 'plastic zone' (σ_1 negative to zero) that is approximately five times deeper around the tunnel perimeter compared to the HC model.

- CHI models displacements up to ten times greater on the tunnel perimeter compared to the HC model. Similarly, at a depth of approximately 1 m behind the side walls and below the floor, CHI models at least five times higher displacements compared wo the HC model.

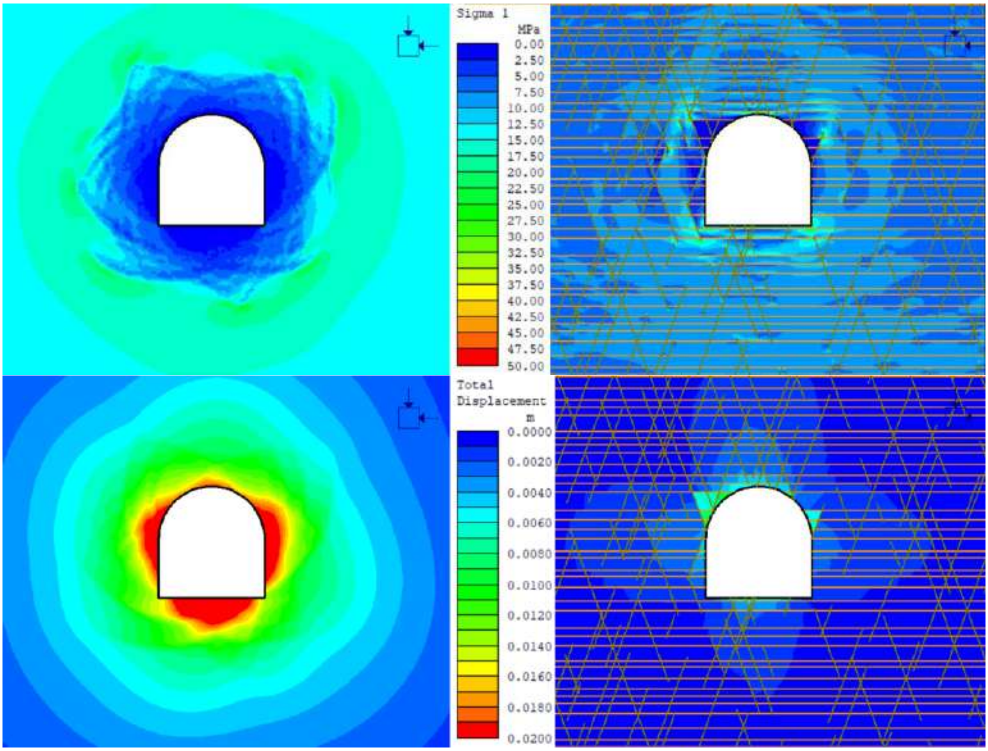


Fig. 5. Drainage Tunnel (5 m wide x 5.5 m high) 2D FEM Model Results. Top Row: Maximum Principal Stress (σ_1). Bottom Row: Total Displacement. Left: CHI (continuum, homogeneous and isotropic) model outputs showing yielded zone with depth of 2-3 m around the tunnel perimeter. Modelled behaviour type: potential for voluminous stress induced failure (BT 4, Sq.) with over 20 mm displacements up to a depth of approx. 1 m behind side walls and below floor. Right: HC (hybrid-continuum) model outputs showing minimal yielded zone of less than 0.5 m on side walls and locally unstable blocks at the tunnel shoulder. Modelled behaviour type: discontinuity-controlled block fall (BT 2, Wg.) with less than 3 mm displacements at a depth of approx. 1 m behind side walls and roof.

Ground support requirements based on the CHI model (combination of shotcrete, mesh and bolting) would be significantly higher than predicted by the HC model, which indicates only a need for spot bolting. Based on the observed excavation performance in the existing exploration tunnel (Fig. 4), the HC model outputs are significantly more realistic than the CHI model.

3.2. Case Study II: Mine Stope in Australia

Case study II involves an unsupported mine stope in an Australian gold mine. As shown in Figure 6, the stope has experienced local block falls from the metasandstone within the hanging wall. The rock mass is generally blocky with the foliation dipping at approximately 45 degrees (i.e., shallower than the ore body and stope dip), typically with two orthogonal joint sets. GSI is normally in the range of 60 to 70. Ground support and stope sizes are typically designed using Q’, which range from 10 to 40.

The stope is located 500 m below surface with regional horizontal stresses approximately twice the vertical stress ($\sigma_1/\sigma_c \approx 0.26$), and σ_c is in the order of 100 MPa. The development drives are 5.5 m wide x 5 m high, and the distance between the top and bottom (extraction) drives is 40 m (stope height).

Potential failure modes are generally expected to involve localized brittle failure of intact rock and movement of blocks (discontinuity-controlled block falls), which have been observed.

Fig. 6. Blocky metasandstone on extraction level. Block falls observed from hanging wall of unsupported stope.

In a similar manner to Case Study I, CHI and HC models were used; however, plastic numerical modelling was undertaken with shear strength reduction (SSR) to estimate Factor of Safety ($SRF_{critical}$). The plastic modelling outputs in Figure 7 have vastly different results:

- CHI model predicts crown failure, i.e. Factor of Safety less than one for large span on the hanging wall of the stope.
- HC model predicts a marginally stable hanging wall with a Factor of Safety of 1.1. The potential for discontinuity-controlled block fall for kinematically unstable blocks exists.

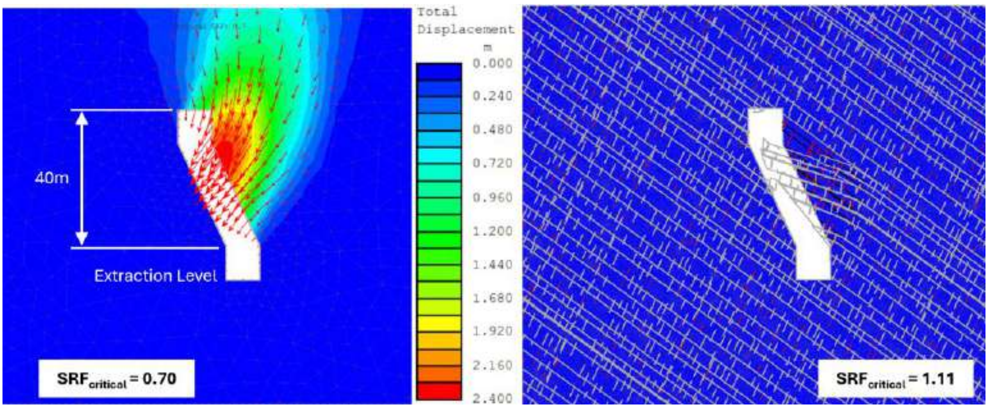


Fig. 7. Unsupported Mine Stope 2D FEM Model Results showing Total Displacement. Left: CHI (continuum, homogeneous and isotropic) model outputs show a large volume overbreak in the crown, i.e. crown failure (BT 7, Ch.), which may be a result of chimneying or progressive shear failure with a Factor of Safety of 0.7 (i.e. unstable). Right: HC (hybrid-continuum) model outputs show an exaggerated mesh distortion to illustrate discontinuity-controlled block fall (BT 2, Wg.) potential in the hanging wall; however, with the exception of kinematically unstable blocks, the hanging wall appears to be marginally stable with a Factor of Safety of 1.1.

If the stope were designed with CHI, the stope span, and therefore, height between development drives, would need to be significantly reduced to achieve stable conditions. Reduced distance between development drives incurs significant additional costs and may result in an uneconomical mine design. The HC model appeared to be more realistic and was validated with actual stope performance (Fig. 6).

3.3. Case Study III: Cavern in Norway

Case study III reviews a 62 m span cavern excavated at Gjøvik in Norway that was initially analysed with a discontinuum approach using UDEC and model outputs were later validated with monitored displacements from extensometers (MPBX) which were installed from surface into the roof of the cavern (Barton et al. 1994). This modelling included sequenced excavation and support installation.

The cavern was constructed near surface within a gneiss rock mass that is typically blocky with Q values ranging from 1 to 30, with a weighted mean of about 9, i.e. *fair* quality rock. Based on available data, GSI is likely be in the range of 70 to 80. Stresses are very low and intact rock strength is high. Based on Figure 3, expected ground behaviour is discontinuity-controlled block falls.

Figure 8 shows modelled displacements replicating the UDEC models of Barton et al (1994) using elastic continuum and hybrid-continuum approaches using RS2 and FDEM or hybrid-discontinuum using PROROCK software.

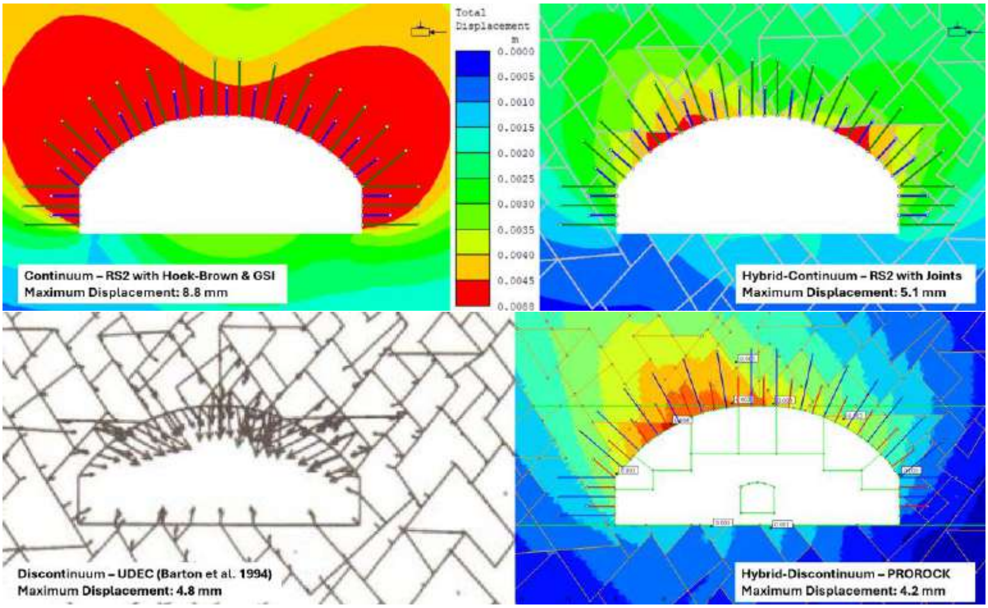


Fig. 8. Gjøvik Cavern (62m Span) 2D FEM, FDEM and DEM Elastic Model Results showing Total Displacement or Displacement Vectors. CHI (continuum, homogeneous and isotropic) model outputs show a significantly larger zone of displacement than the HC, FDEM and DEM models.

In a similar manner to Case Studies I and II, the CHI (continuum, homogenous and isotropic) model outputs show a significantly larger zone of displacement than the HC, FDEM and DEM models. The HC (hybrid-continuum) also shows a slightly larger zone of displacements compared to the FDEM and DEM models. In terms of absolute, modelled displacements, CHI predicted approximately twice the displacements of HC, FDEM and DEM models, which all had quite similar results.

4. Discussion and Key Findings

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.

Continuum modelling approaches offer a simple and fast means of predicting ground behaviour. By way of example, in three case studies presented, the homogeneous, isotropic, continuum models using the Hoek-Brown failure criterion presented in this paper each took less than one hour to develop and a few seconds to compute results. The hybrid continuum and FDEM models to several hours to develop, and perhaps a few minutes to compute.

When applying continuum approaches and the Hoek-Brown Failure Criterion, an engineer or practitioner should be considerate of:

- Limitations of GSI and the Hoek-Brown Failure Criterion as described by Marinos et al (2005) and Hoek and Brown (2019).
- The simplification of engineering geological characteristics of the rock mass such as the complex assemblage of bedding, foliation, joints and other discontinuities is simplified into a single number (GSI), which assumes homogeneous and isotropic strength and stiffness. As demonstrated in the three case studies, this simplification often results in:
  - Model predicting an incorrect behaviour type or inappropriate failure mechanism.
  - Over-estimating the depth of yielding and over-estimating total displacements (potential requirement for additional support, and therefore, additional cost).
  - Under-estimating Factor of Safety (potential requirement for additional support and reduced excavation spans, and therefore, additional cost).
- GSI and Damage Factor, D, have a profound effect on the estimated rock mass modulus, E_m, when applying the approach by Hoek and Diederichs (2006). Estimation of E_m using different empirical methods yields vastly different and variable results (Bar and Schubert, 2023).
- Limitations in continuum approaches for modelling absolute displacements since it is represented as a continuous displacement field (Sjöberg, 1999) in which ‘detachment’ cannot occur. As such, continuum models often predict ‘larger zones of displacement’.
- The above limitations are also applicable to Mohr-Coulomb and other failure criteria.

Discontinuum approaches including hybrid continuum, FDEM and DEM, offer a more comprehensive representation of engineering geological conditions and generally provide more realistic outputs in terms of modelling ground behaviour, failure mechanisms and predicting displacements. However, each model provides results for only a single assemblage of discontinuities, i.e. several iterations of possible assemblages should be modelled to ensure ground behaviour is adequately assessed.

With limitations in site investigations and inherent uncertainty in ground conditions, practitioners should always remember that *all models are ultimately wrong, but some are useful*, and that wherever possible, calibration or validation with actual ground behaviour should be a key point of focus.

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