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The importance of rock engineering for deep open pit slopes: Some considerations

R. Ulusay
Hacettepe Univ., Dept.of Geological Engg., Ankara, Türkiye
resat@hacettepe.edu.tr
0000-0003-1930-7611

Abstract

When compared to underground mining, open pit mining is a more cost-effective mining method allowing high grade of mechanization and large production volumes. Much more need due to the exponential growth in the world population with easily accessible mineral resources becoming more scarce, increasing commodity prices, and improved production capabilities, leaded to much deeper and larger open pit slopes than have previously been considered and implemented. Therefore, particularly in the last few decades, depths of open pits approached and exceeded 1000 m. At these increased depths the risk of slope failure becomes greater, dependent upon the imposed design, and makes engineers question whether the analytical assumptions previously used on lesser slope heights are still valid. Formation of slope damage features, such as brittle fracture, rock mass dilation and bulging, are the main factors causing deformation of deep open pits and high natural rock slopes. In addition, strength of slope forming rock masses and discontinuities is reduced by the progressive accumulation of slope damage and cause a decrease in slope stability. In this keynote differences and similarities between the stability of "deep-large" and "classical (shallower)" open pit slopes are discussed in terms of the main factors controlling slope stability such as rock type, discontinuities, effect of degradation, rock damage, hydrogeological conditions, rock mass strength, geotechnical model, effects of in-situ stresses and earthquakes, and importance of movement monitoring based on the recent practice and using some examples selected from practice.

Keywords

Deep open pit, Slope stability, Failure mechanism, Slope monitoring, Rock damage.

1 Introduction

Slope design, regardless of the dimensions of the slope, is an integral part of open-pit mining engineering. The slope angle is also a key component that determines whether a proposed or existing project can remain economically viable. The first studies concerning the geotechnical design of open-pit slopes were carried out in the late 1960s and early 1970s. Initially, slope design was conducted according to the principles of soil mechanics and other branches of materials engineering. At that time, only a few open pits had slope heights slightly exceeding 300 m, and pit designs were commonly based on the rule of thumb "take the overall slope angle as 45° and then observe" (Stacey 2009). This approach often led to widespread slope instabilities in open pits, resulting in injuries and/or fatalities as well as severe economic losses and long-term interruptions in ore production.

By the late 1980s, parallel to the increase in global population and the growing demand for ore production, deposits that had previously been considered uneconomical for open-pit mining due to their low grades and deep occurrence began to be exploited. Furthermore, several existing open pits weredeepened. A plot showing the historical record of open pit slope height and overall slope angle, including data from Sjöberg (1999) (orange circles, representing open pits up to 1999) together with more recent data compiled by the author from the various sources (blue triangles), is presented in Fig. 1.This plot indicates that, except for a few pits, the depth of the majority of open pits was between 300 and 400 m until the early 2000s. However, after 2000, mining operations began to be carried out in open pits exceeding 500 m in depth, with some reaching depths of up to 1350 m. Open-pit slopes excavated to depths of several hundred meters were regarded as "deep slopes". However, it is important that these operations are conducted in a safe and economic manner. This means that competent designs and techniques should be adopted to support the ever growing mining capacities. However, the attainment of depths exceeding 500 m in some modern open pits (Figs. 1 and 2) has resulted in the occurrence of ground stresses that are high even by underground mining standards, along with highly complex slope behaviours and instability mechanisms (e.g. Sjöberg 1999, 2013; Stacey et al. 2003; Elmo et al. 2007, 2022; Lorig 2009; Tuckey et al. 2013; Cambio et al. 2019; Donati et al. 2020). Compared with soil-like materials, rock masses are discontinuous in nature due to the presence of joints and other discontinuities. As in-situ stresses increase with depth, rock material failure within the slope-forming rock units becomes more pronounced, leading to damage accumulation and a reduction in strength.Although the slope design process described in many publications is generally similar for all mine slopes, the prediction of ground performance in deep open pit slopes exceeding 500 m in height involves relatively greater uncertainty due to the factors such as high stress, intact rock breakage and time-dependent behaviour, and therefore, such open pits are considered as deep/ultra-deep open pits (Robotham 2011).

The complexity of slope failures due to increase in depth of slopes makes it increasingly difficult to answer key questions in slope design, such as: (a) what types of slope instabilities could affect the pit? (b) what is the likelihood of a major instability developing? (c) what could be the potential consequences of such failures? and (d) can the operation withstand these consequences? (Karzulovic 2004). Therefore, compared to shallow open pits, the design of deep and large open-pit slopes inherently involves a significantly higher degree of uncertainty. Moreover, when compared to classical open pits, limited experience gained from deep open-pit designs, together with the inherent knowledge gaps and limitations of conventional analytical methods, further affect the process of assessing the stability of deep slopes. Therefore, this requires a better understanding of catastrophic and expensive slope failures in deep open pits.

In this keynote, differences between "deep-large" and "classical (shallower)" open pits are discussed in terms of the main factors affecting the stability of open pit slopes, such as rock type, degradation and discontinuities, formation and effect of damage in rock slopes due to in-situ stresses, hydrogeological conditions, and strength of rock masses and discontinuities. Then the role of geotechnical model, influence of earthquakes and in-situ stresses, slope movement monitoring, which are particularly more important for deep open pits, are also highlighted with some examples selected from practice. The methods of analysis, such as kinematic, limit equilibrium and numerical methods used in slope stability assessments for deep and conventional open-pit mines have not been included within the scope of this keynote, since these methods constitutes a separate topic in their own right.

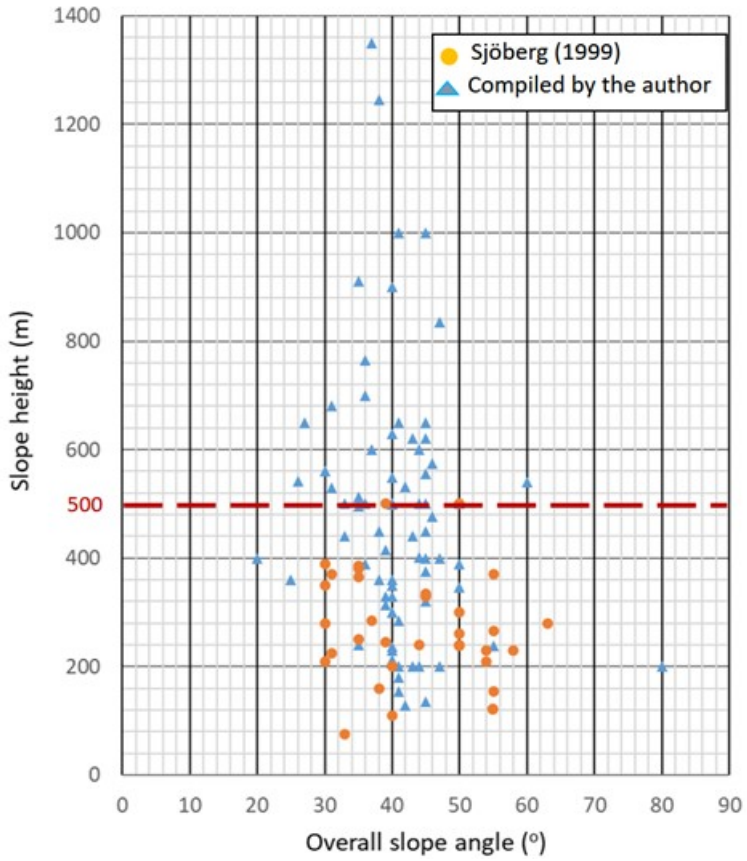


Fig. 1.Plot showing the increase of the open pit slopes based on the data on overall slope angle and slope height compiled from open pits in different countries (drawn using the data from Sjöberg (1999) and most recent data provided from different sources by the author).

(a)



(b)



Fig. 2. Examples of deep/ultra-deep open pits: (a) Bingham Canyon Copper Mine, USA (current depth: 1350 m; Photo courtesy of Rio Tinto Kennecott ROV &Survey Teams, Ergun (2026)), (b) Chuquicamata Mine, Chile (current depth:1000 m) (from Contreras et al. 2019).

2 Effects of main inputs on slope stability

2.1 Effect of rock type

Because of their different origins and nature, rocks possess distinct intrinsic geological properties. For example, in soft and weak rocks such as schists and shales, the principal factor controlling slope stability is the lithology of the rock itself. In contrast, in hard rocks such as granite and limestone, stability is mainly governed by the dominant sets of discontinuities.The mineralogical composition of rockscontrols the mechanical properties and the behavior of the rock mass andalso determine the strength of the rock mass.

Another important rock slope instability is experienced in **B**lock-**I**n-**M**atrix **ROCK**s (Bimrock), which are complex geological mixtures such as mélanges, conglomerates, faulted and sheared rocks, where blocks of varying rock types and origins are embedded in a finer-grained matrix (e.g. Medley, 1994, 2012; Figs. 3a.1-a.3)). Bimrock behaviour is particularly common in open pits excavated within the following geological environments, such as ophiolite belts, subduction melanges, serpentinite-mixed rock zones, tectonic breccia zones, large-scale fault zones and porphyry copper deposits containing volcanic breccia and agglomerate. In such materials, the types of slope instabilities, as well as the shape and location of potential failure surfaces, depend on the frequency, size, and orientation of the blocks (Fig. 3b). Due to the significant spatial, lithological, and mechanical variability of these materials, the accurate geological and geotechnical characterization of bimrocks is essential to avoid unforeseen and costly surprises during slope excavation. In a hypothetical example given in Fig. 3c for the problem related to mischaracterization of a bimrock in a slope case, the geotechnical investigation for a main road the exploration borings were teminated about 2 m into sandstone bedrock. The engineer concluded that the instability was shallow and composed of clay and boulder colluviums sliding on the contact with the underlying sandstone bedrock (Fig. 3c.1). After the excavation started, it was seen that the instability was actually a deep-seated earth flow in pervasively sheared melange, rather than a shallow soil mass sliding on top of the bedrock (Fig. 3c.2). Although some works have been conducted and some approaches have been recommended, there is still no general consensus on the approaches and guidelines for site characterization of bimrockss, and therefore, further studies and combination of their results with existing experiences are necessary.

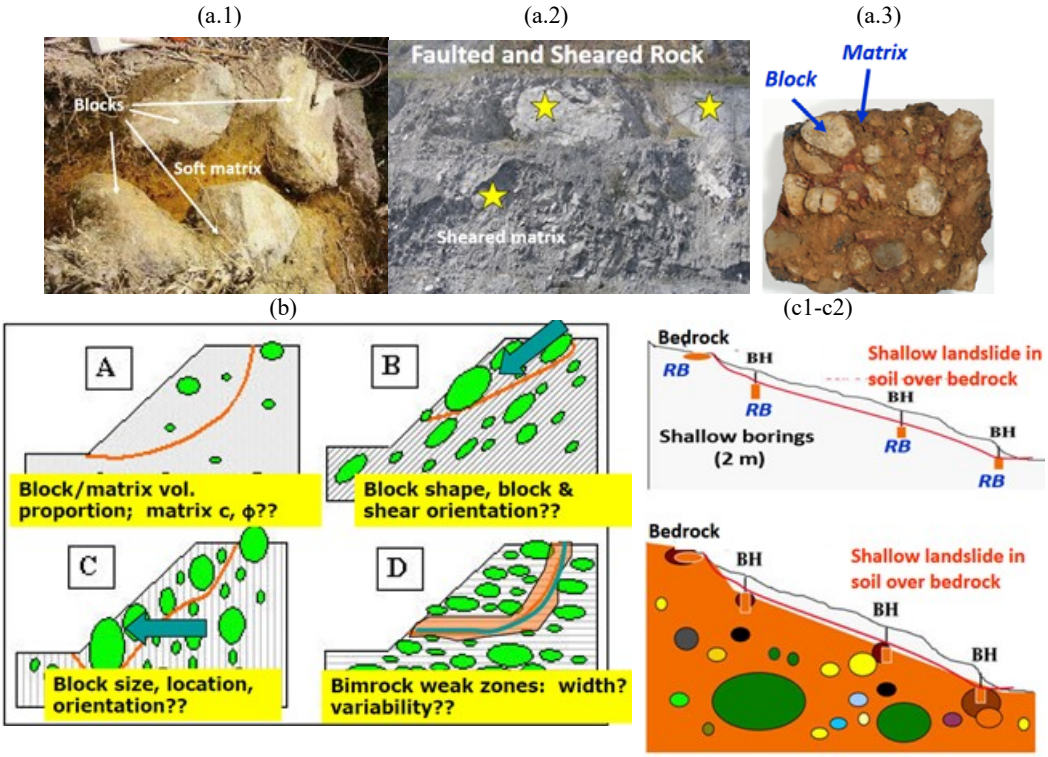


Fig. 3.(a) Views from three typical bimrocks (a.1. coarse blocks in soft matrix, a.2. faulted and sheared rock, a.3. conglomerate; Medley 2012), (b) effects of block proportion and orientation on slopes and failure surfaces excavated in bimrocks: (A) very low block proportion-analysis similar to that for soil slopes, (B) failure surface influenced by the orientation and nature of matrix shearing,(C) large blocks which influence tortuous failure surface that negotiate around blocks and strength to matrix, (D) block-poor zones within generally block-rich are weaker and more likely to fail (Medley 2012), (c) a hyphothetical example on mischaracterization of bimrocks (c.1. before and c.2. after borings) (Medley 2008).

2.2 Effect of degradation

Rocks may undergo degradation when they are exposed to atmospheric conditions and/or hydrothermal fluids through rock mass. The degradation process is commonly known as deterioration in the forms of weathering and alteration depending upon the physical and chemical process involved. Weathering transforms hard rocks into soft rocks which maintain the structure of the intact rocks, but are characterized by higher void ratios and reduced bond strengths. Soft rocks are transformed into thick granular soil mantles generally called residual soils. The degradation process also influences the joint spacing and discontinuity filling material in the form of clay and results in drastic changes in strength and deformability characteristics of the slope forming rock mass.

Significant progressive deterioration of rock slopes may occur in engineering time, often giving rise to the need for unplanned maintenance and constituting safety hazard, and the physical and chemical weathering effects can be predominant in controlling the modes of rock slope failure. For example, open sheet/exfoliation joints, which are formed by stress relief as a result of erosion of igneous plutons, tend to run sub-parallel to the rock head with a tendency to daylight in the cut slopes resulting in a structurally controlled instability such as planar failure (Fig. 4a). If the slope forming rock mass has transformed into a soil due to intense chemical weathering (grades V or VI) or alteration, the slope will fail in the form of circular sliding as commonly observed in soil slopes and this kind of failure can be more dangerous for deep open pits as seen in Fig. 4b. Differential weathering, which is a common source of rock slope instability particularly along steep slopes, generally occurs in cases where slopes consist of inter-layered hard (e.g. sandstone, limestone) and soft rock (e.g. shale, claystone, tuff and mudstone) units. As differential weathering of softer rocks progresses, undercutting can lead to loss of support and collapse (falls or topples) of the overlying material. Large-scale, sub-vertical discontinuities and over-break fractures may promote failure by intersecting undercut surfaces and/or reducing block size within the rock mass (Fig. 4c) (e.g. Neiman 2009, Admassu et al. 2012). Deterioration, however, is rarely given much attention at the design stage; emphasis is on the avoidance of deep-seated failure.

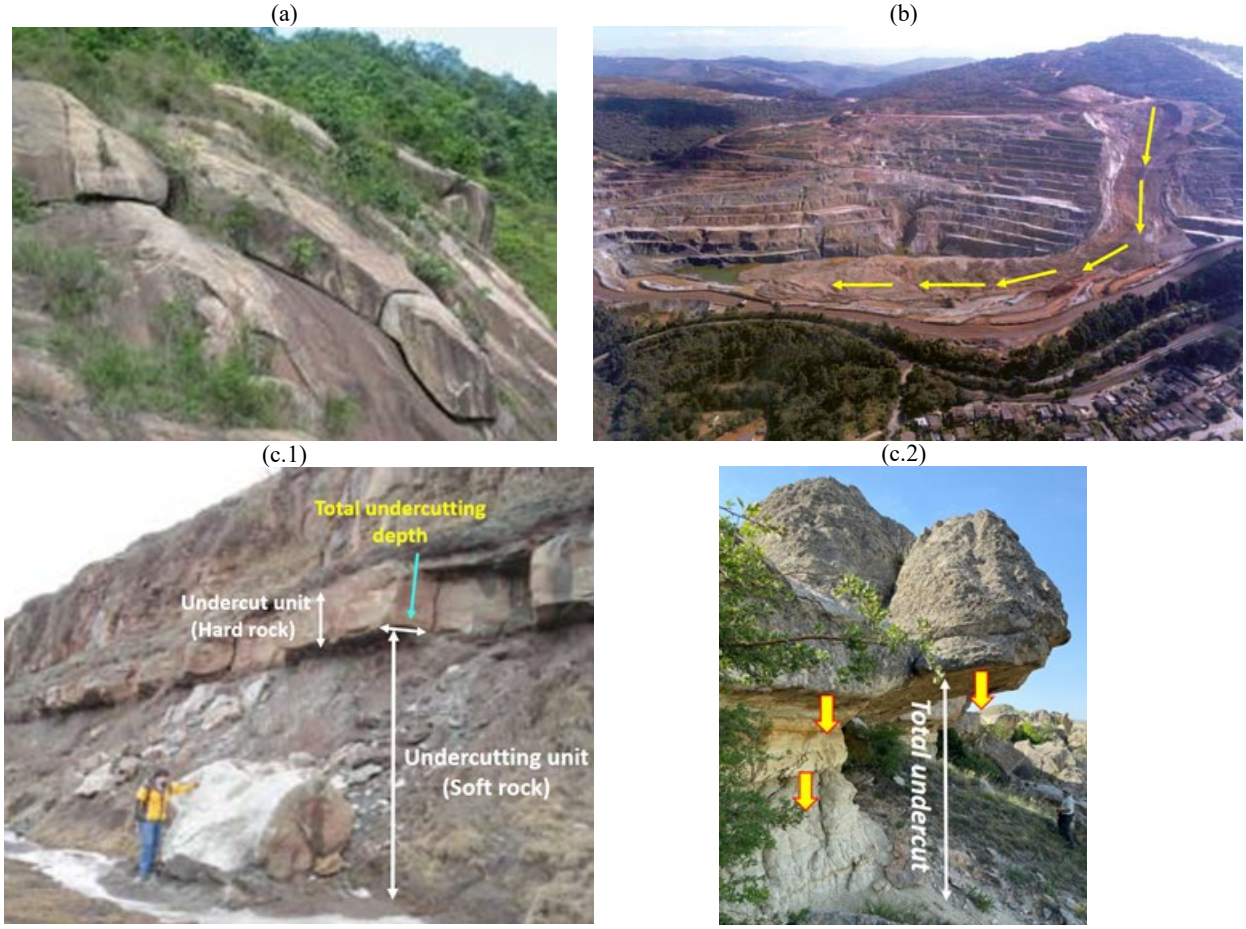


Fig. 4. (a) Partial movement and dilation of sheeting slabs typical of early stage of failure in Hong Kong (Hencher et al. 2011), (b) failure of weathered zone material in an open pit (Martin and Stacey 2013), (c.1-c.2) examples for undercutting-induced rockfalls due to differential weathering in USA (Admassu et al. 2012) and in Cappadocia, Türkiye (Photo: R. Ulusay), respectively.

2.3 Effect of structural features and scale

In rock masses divided by discontinuities, the most significant factor controlling slope instability is different types of discontinuities such as bedding planes, joints, faults, and schistosity. The adverse interaction between discontinuities and the slope orientation plays a major role in the development of slope instabilities. Depending on the degree of structural control exerted by discontinuities, four main types of instability models are generally recognized. The first group includes failures that develop along discontinuities or their lines of intersection, such as “planar”, “wedge” and “multi-planar” failures, and those occurring along columnar structures dipping opposite to the slope direction called “toppling failure” (Fig. 5a–d). Multi-planar failure is a type of structurally controlled instability in which sliding occurs along two or more intersecting discontinuity planesand unlike simple planar failure, movement is governed by a combination of planes that together form a kinematically feasible sliding surface. Structurally-controlled failures are commonly observed both in shallow and deep open pit mine slopes and multi-planar failures can involve large rock volumes and may develop progressively. The second group represents circular (rotational) failures, which are partly passes through the rock material and partly along discontinuities, without structural control, similar to those observed in soil slopes, where the rock mass is highly fractured by closely spaced discontinuities (Fig. 5e.1), or extremely weak or highly weathered or altered (Fig. 5e.2). Rotation and translation of individual blocks in the rock mass would help to create a failure surface. Mapping of natural discontinuities can be conducted according to sampling methods suggested by ISRM (1981, 2007), independently of whether the discontinuity parameters are collected using direct physical mapping or indirect data collection by means of remote sensing techniques, such as digital photogrammetry and laser scanning techniques.

As emphsized by a number of researchers (e.g. Sjöberg 1999; Lorig 2009), the traditional assumption of low-stress conditions in slopes implies that structurally conrolled failures mentioned above dominate in rock slopes, while this may be true for slopes with large scale structural features, it does not necessarily apply to a rock mass with many, but smaller, discontinuities. For a high slope, the rock mass will appear to be very fractured, as the length of individual discontinuties is small compared to the dimensions of the slope. This effect of scale is shown in Fig. 6a-c. for 30 m, 90 m and 500 m high bench slope, interramp slope and overall slope with angles of 70^o, 30^o and 50^o, respectively. In these figures, a 2-D cross-section of the slope is shown with two hyphotetichal joint sets with the following characteristics: Joint A (length. 10±1 m; rock bridge: 3±1 m, spacing: 3 m) and Joint B (length. 8±2 m; rock bridge: 5±2 m, spacing: 3 m; spacing: 7 m). Fig. 6 suggests that a rock mass can thus be described as being a closely jointed when the joint spacing is small in relation to the scale of the entire slope. From this it is evident that the failure mode of a large scale slope can differ significantly from that of a small scale slope (Sjöberg 1999). As shown in Fig. 6d, a rock mass than might appear “blocky” for an underground opening could appear “disintegrated” for a large and deep slope (Lorig 2009) and these examples higlight the importance of scale effect in rock slopes, particularly for deep open pits.

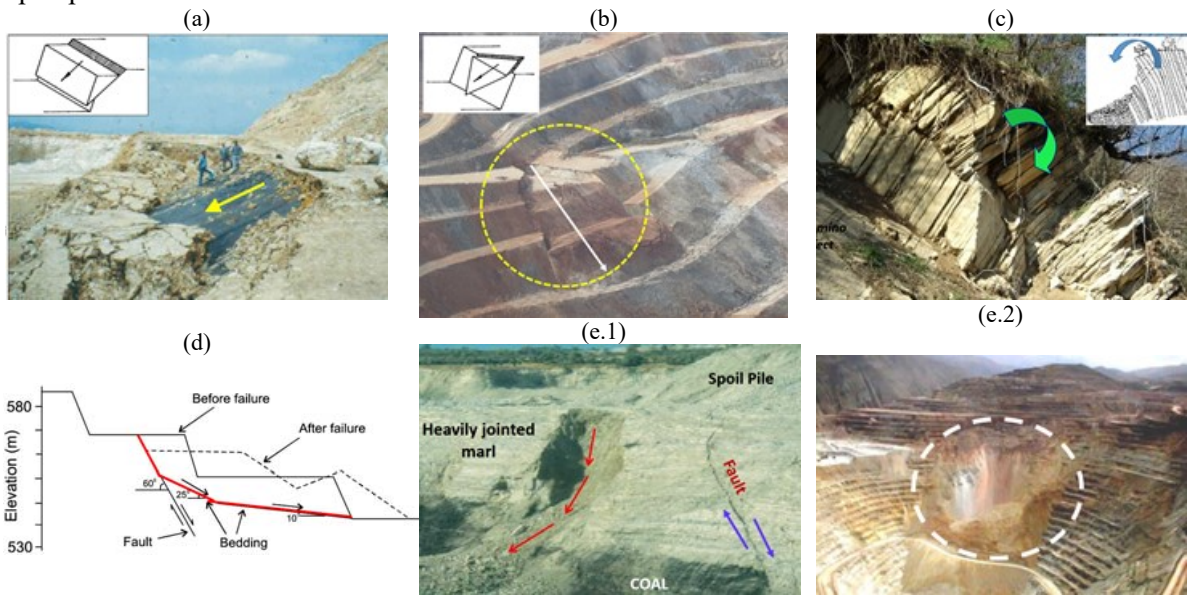


Fig. 5**.** Five main structurally-controlled failure modes occurring in rock slopes: (a) planar, (b) wedge, (c) toppling and (d) multi-planar failure, and circular failure in (e.1) a heavily jointed rock mass (a, d, e.1- Ulusay 1991; b, c - Photo: R. Ulusay) and (e.2)failure in a highly weathered east wall slope at Cuajone open pit in 1999 (Peru) (Hormazabal et al. 2013)

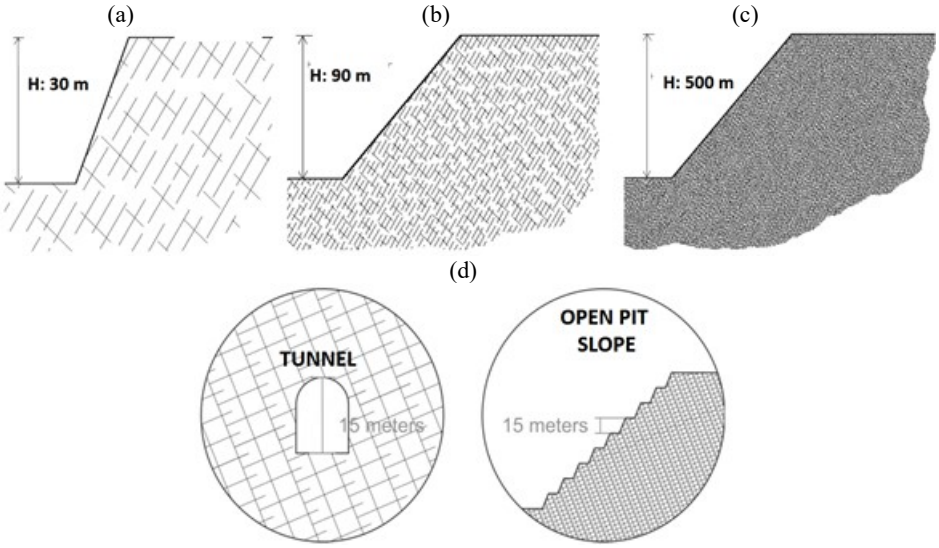


Fig. 6.Discontinuity patterns for (a) bench, (b) inter-ramp and c) large scale overall rock slopes with different slope angles of 70^o, 50^o and 50^o and two different pre-existing joint sets (slightly rearranged from Sjöberg 1999), (d) illustration of how the same rock mass could appear differently for an underground excavation and a rock slope (Lorig 2009).

2.4 Rock damage in deep open pits

Today, slopes of deep open pit mines can reach depths in excess of 1 km and as a result of this progress, they are subjected to deformations which are inevitably associated with the formation of slope damage features, such as rock mass dilation and bulging, brittle fracture and rock falls. The progressive accumulation of slope damage, in turn, causes strength degradation, and consequently the risk of instability increases. Due to a combination of geological units with different characteristics, the intersection of a greater number of structural elements, higher magnitudes of in-situ stresses, and increased deformation with depth, more complex (composite) instability mechanisms are often encountered in deep/ultra-deep open pits (>500 m). This is the most distinctive difference between deep and conventional (shallower) open-pit mine slopes. Based on the evidence from the past cases supported some form of extensional mechanics occurring with yield and failure of deeper open pit slopes.Some researches and practitioners (e.g. Hoek et al. 2000; Stacey and Xanbin 2004) considered that in addition to structurally controlled failures, deep open pit slopes may also involve composite defect failure mechanism.

Long- and short-term deformation of rock slopes is associated with the formation of slope damage features, and consequently, the instability models in deep open pits are strongly dependent on the scale of the slope. Based on the findings from Ok Tedi Mine in New Papua Guinea, Little (1999) and Baczynski (2000) proposed a model for deep and large slope instabilities with complex failure surfaces covering interaction of five possible zones of different strength that can act in the development of some instability at an inter-ramp or global scale.As schematically illustrated in Fig. 7a, in deep and ultra-deep open-pit slopes, sliding may occur not only along unfavourably oriented discontinuity planes, but also through shear failure within the intact rock bridges (intervals of intact rocks between the tips of adjacent and non-persistent discontinuities), and by the dilation of step-like arrangements of steep and sub-horizontal joint systems within the rock mass and increase in rock falls. In this conceptual illustration, Donati et al. (2020) emphasized that such damage induced fracture connectivity on secondary permeability, pore water pressure and shear strength along failure surfaces is critical. An example showing how the presence of rock bridges could manifest even within a rock mass characterized apparently fully continuous and interconnected fractures is given in Fig. 7b. The prism failure mechanism proposed byPrandtl (1923) stability analysis of the curved or irregular surface geometry by Mencl (1966) first time. In this method, the stiffness of the sliding mass influences the development of kinematic freedom and the transfer of driving force from an upper active block, through a Prandtl transition wedge and into a lower passive block (Fig. 7c). Eberhardt et al. (2004) and Fisher and Eberhardt (2007) found that a Prandtl zone of yielded rock mass developed at the base of the sliding surface and propagated towards the upper active wedge and it provides an accurate description for the development of kinematic freedom, respectively. Based on the analyses, considering “volumetric rock bridges” and using distinct element code UDEC for the assessment of stability of model slopes Tuckey et al. (2013) concluded that it may be especially applicable in the case of highly fractured rock masses, or in very large open pit slopes, where jointing can be considered effectively homogenous.

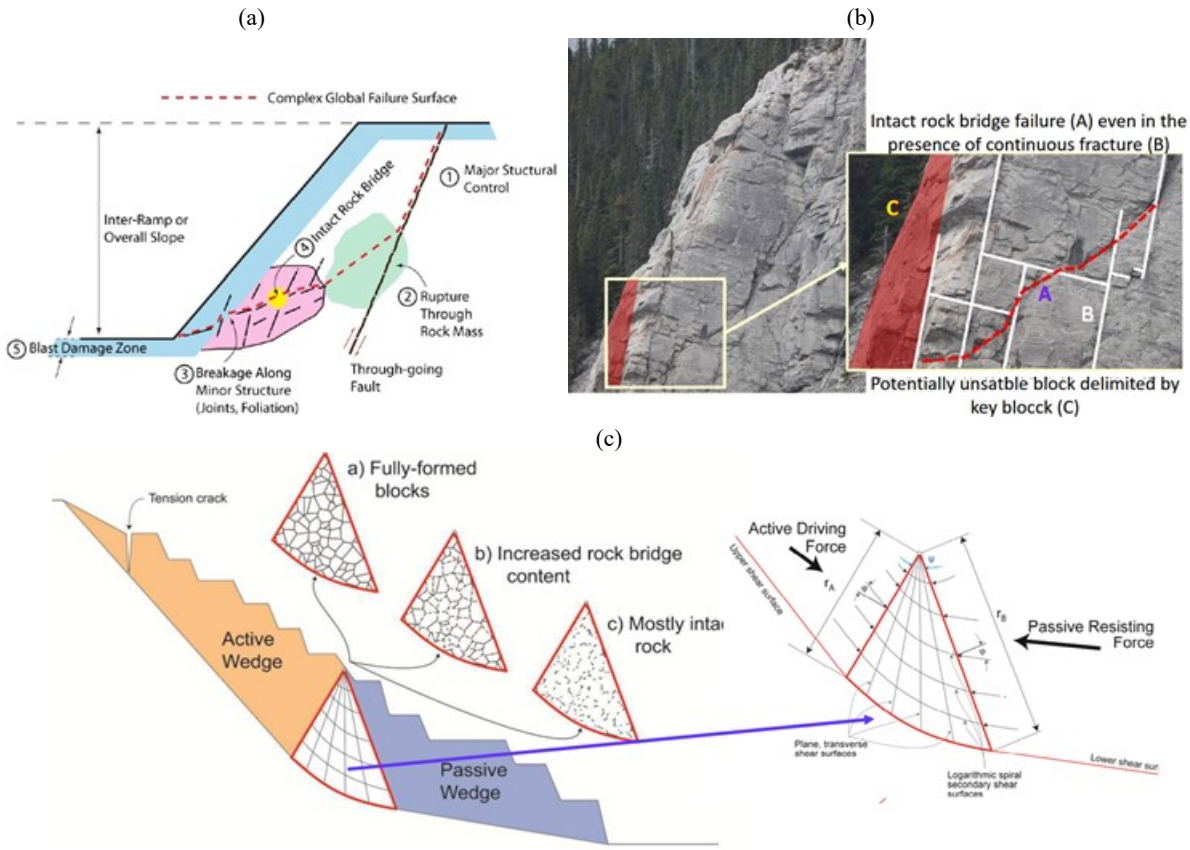


Fig. 7. (a) Conceptual illustration of a composite failure through a deep pit slope (modified by Tuckey et al. (2013) from Little (1999) and Baczynski (2000)), (b) example showing how ongoing intact rock bridge failure may occur even in the presence of apparently continuous fractures (modified from Elmo et al. (2018) by Elmo et al. (2022)) (c) Prandtl wedge applied to bi-linear slope failure; the Prandtl Prism functions as a transition zone, where the direction of driving stresses from the active upper block are transferred to the lower passive block (Tuckey 2008).

The Bingham Canyon Mine, located in the Utah State of USA (see Fig. 2a), is a major producer of copper, gold, silver and molybdenum since 1906 and is currently the deepest (1350 m) open pit mine in the world. A huge slope instability, called Manefay Slide, occurred in the upper northeast wall of this pit on April 10, 2013 which was a largely unconfined nose-shaped geometry (Ross 2017)(Fig. 8). Williams et al. (2021), who evaluated theManefaySlide after it occurred using modern InSARtechniques, mentioned that the mechanism of this slide has been studied, reviewed and back-analysed with numerical models by experts, however, there has only been a single publication by Ross (2017) released with first-hand knowledge of the mechanics and it established a consensus that the failure is a complex, active-passive failure.



Fig. 8 Views from the Manefay Slide occurred in Bingham Canyon Mine, Utah, USA (Sources: http://www.mining.com; Williams et al. (2021))

The geology of the mine is complex and varied and includes sedimentary rocks folded, faulted, intruded by igneous materials and subjected to alteration (Porter et al. 2012). Although previously a number of much smaller and simpler wedge and circular failures have developed in this mine, Ross (2017) mentioned that due to its complexity the Manefay slide which was different than any of the previous wedge and circular failures developed in this pit, it could not be identified with standard 2D analyses. In summary, the mechanism of this slide is explained by Ross (2017) as given below.
(i) when the movement started it caused a reduction in strength between the beds of sandstone and siltstone additional movement became easier,
(ii) in addition to reduced cohesion, clayey contacts called gouge with low permeability created perched water table and resulted in a saturated zone,
(iii) the active block was moving down the dip of Manefay bed and this movement was being resisted by the massive wall west of the active block, which was solid rock and had no place to move (Fig. 9a)
(iv) the strength of the active block was exceeded by the weight and stresses of the active block, the passive block failed in a catastrophic manner and was ejected from between the immovable wall to the west (Fig. 9b),
(v) the 1st block moving changed the area of the mass tend to fail and created a new active block (gray colored area in Fig. 9c). But much of the 2nd active block included old mining waste from earlier decades. The stress on the 2nd passive block became critical and 40 minutes after the 1st failure the stress broke the intact rock along the estimated failure plane as shown in Fig 9d. In his book, Ross (2017) emphasizes that the ManefaySlide was a complex failure at a mine with intricate geology and geologic structures and it was also true for all deep and large open pit mines that should consider the possibility of a large-scale active/passive failure mechanism as a part of the geotechnical analyses and highwall designs.

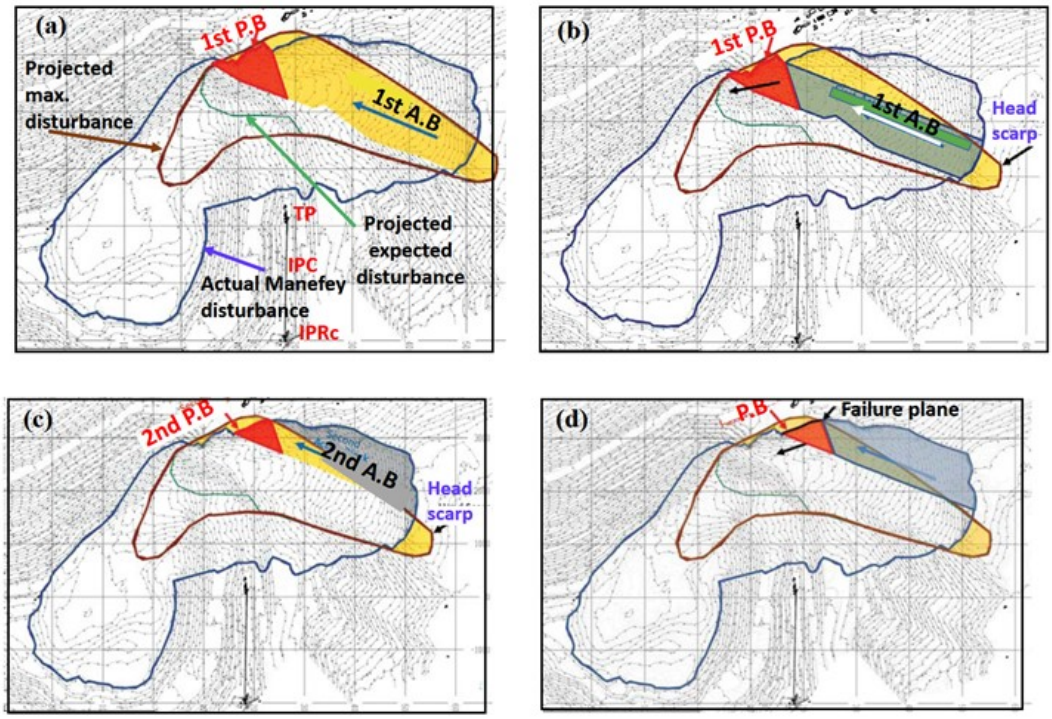


Fig. 9 (a) Manefay Slide general layout and active block (A.B) and psssive block (P.B) - pre-failure, (b) 1stfailure, (c) 2nd active block, (d) 2nd failure (slighty rearranged from Ross (2017))

Quantificaton of discontinuity persistence and intact rock bridges within rock slopes is a critical component of rock slope stability assessments and requires both the use of new data collection techniques and rigorous data interpretation through fracture network analysis (Elmo et al. 2007). Jennings (1970) is the first researcher who documented the importance of intact rock bridge fracturing for slope stability assesments. More recently, the Discrete Fracture Network (DFN) approach using the probability density functions to represent discontinuity orientation, spacing and persistence has been developed and was shown as a useful numerical tool for the use of synthesizing fracture network models obtained from digitally and conventionally mapped data and its use increased particularly in the last decade (e.g. Elmo et al. 2007, 2014, 2015, 2018, 2022; Stead and Eberhardt 2013; Tuckey and Stead 2016; Rogers et al. 2009, 2014; Dershowitz et al. 2017; Donati et al. 2020; Montiel et al. 2020; Ojeda et al. 2023; Ambah et al. 2025).

The DFN approach is based on the characterization of each discontinuity set within a structural domain using statistical distributions of fracture properties mainly orientation, length (persistance), intensity, termination percentage and spatial variation. Since direct investigation of rock structures in rock masses in 3D is not possible, discontinuity data sampled from mapping of exposures in variably oriented outcrops (2D) and boreholes (1D) are used in DFN approach to generate shotcastic discontinuity model which shares statistics of the samples and allows for the incorporation of deterministic discontinuities such as faults or highly persistent joints (Elmo et al. 2013). Natural discontinuity mapping can be conducted using the traditional outcrop mapping (scan-line survey/linear mapping), rectangular window mapping (ISRM 1981, 2007). In addition, indirect data collection with the use of more advanced remote sensing techniques such as terrestrial digital photogrammetry and laser scanning techniques (LiDAR), and also unmanned aerial vehicles (UAV) to characterize the rock masses in open pits considerably increased (e.g. Elmo et al. 2013; Stead et al. 2012, 2019). Such 3D models are also employed to map and characterize slope damage features, such as tension cracks (Stead et al. 2019; Donati 2019), brittle fractures (Sampaleanu et al. 2017; de Vilder et al. 2017); and characteristics of blast damage and intensity, and seepage intensity (Lupogo 2017; Tuckey and Stead 2016; Vivas et al. 2015). An example of DFN fracture model and DFN stereogram for the Los Pleambres open pit (Chile) are given in Fig. 10. In literature (e.g. Sjöberg 1999; Donati et al. 2020), it is recommended that since at bench scale the modes of failure in open pits are generally structurally controlled (without combination of the rock mass with discontinuities), unless the rock mass has low quality, DFN approach is used for inter-ramp and overall slope scales.

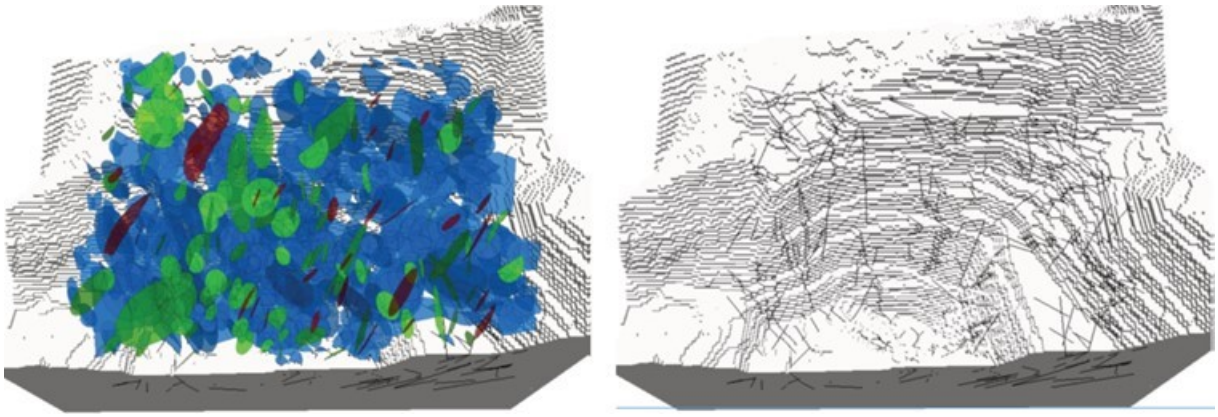


Fig. 10 An example of DFN model for an open pit mine (Montiel et al. 2020).

2.5 Selection of shear strength for discontinuities and rock masses

2.5.1 Shear strength of discontinuities

Accurate determination of the geomechanical parameters of discontinuities and rock masses which are the key inputs for slope stability analyses, is essential for correctly evaluating the stability of rock slopes. The use of empirically suggested parameter values based solely on lithology, discontinuity surface characteristics, or rock mass classifications is no longer considered suitable approaches, and therefore, experimental methods are preferred.

Sjöberg (1999) have highlighted the importance of scale on the strength of the rock mass in open-pit slopes and it is schematically shown in Fig. 11a. The reduction in strength with increasing scale (i.e., when the slope includes more benches) results from the greater number of discontinuities within the rock mass. Therefore, the strength of discontinuities is of primary importance in structurally-controlled slope failures. The degree of roughness must be considered when selecting the appropriate failure criterion for determining the shear strength of discontinuity surfaces. For planar and smooth discontinuity surfaces and discontinuities with thick filling (shearing only of the filling), the "Mohr–Coulomb failure criterion" (Fig. 11b) is the most widely used linear criterion independent of normal stress. It has two components: peak and residual strengths. Hence, a design decision must be made regarding which set of values to use. Although using residual shear strength parameters (c_r, ϕ_r) ensures a conservative (safer) design, it may also lead to uneconomical results. Therefore, it is a common practice to use residual shear strength parameters for fault surfaces that have undergone shearing during their formation. Another approach is to use residual strength values in the analyses for discontinuities that show a rapid reduction in shear strength toward residual levels after very small displacements in shear stress–shear displacement curves. In addition, results of back-analyses of failed rock slopes can be compared with laboratory-determined shear strength parameters obtained from

samples representing the actual failure surfaces. Discontinuities with rough and/or undulating surfaces behave differently under shear loading: initially, asperities are sheared off, and once the surfaces become smoother, shear movement continues at lower strength levels. Hence, the Mohr–Coulomb criterion is not applicable to such discontinuities; their failure envelopes are curved and depend on normal stress (σ_n). For such discontinuities, the "Barton's (1976) criterion" is used to determine their shear strength (Fig. 11b). In slope stability analyses, it is necessary to calculate equivalent cohesion and friction angle from the curved failure envelope.

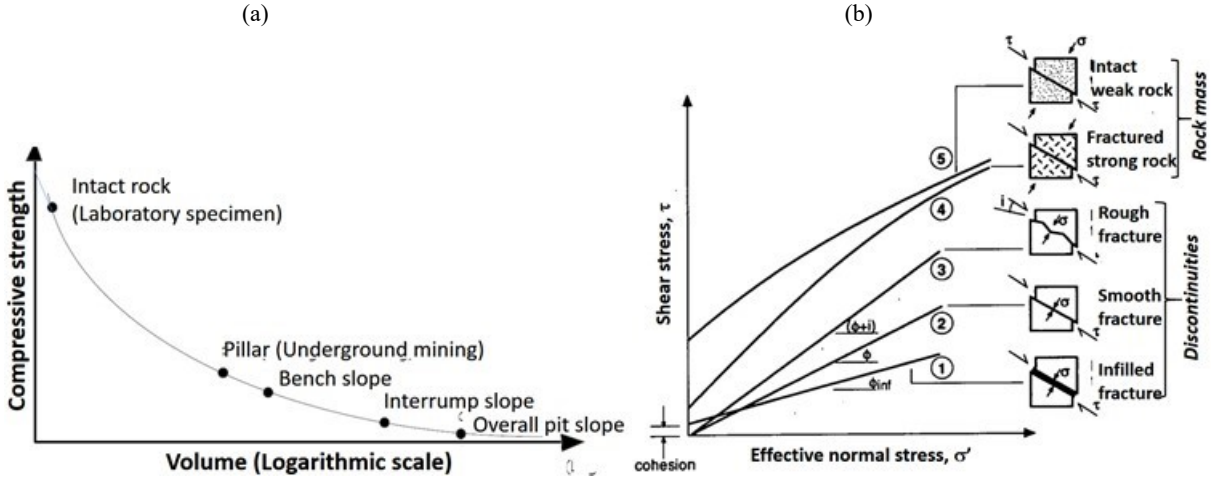


Fig. 11 (a) Scale effect on rock mass compressive strength in terms of rock slopes (Sjöberg 1999), (b) relationships between normal and shear stresses for different discontinuity surfaces and the corresponding failure criteria (Transportation Research Board 1996)

2.5.2Shear strength of rock masses

In rock masses, divided by multiple discontinuity sets with different orientations, the overall behaviour is controlled by both the discontinuities and the intact rock material. Therefore, strength and deformability properties of rock masses should ideally be determined using samples that include both the intact material and discontinuities. However, obtaining meter-sized samples from such environments and preparing them for laboratory testing is generally impossible, and suitable large-scale testing equipment is not available. Laboratory tests on core samples are not representative of rock mass, in-situ testing on rock mass cannot be practical and economic for all cases. On the other hand, even representative values of large scale rock strength can be obtained from the back-analyses of failed slopes, this can only be possible for slopes failed through rock masses. As emphasized by Renani and Cai (2022), since rock masses represent highly variable and complex systems of intact rocks and discontinuities of varying mechanical properties and geometric configurations, which have been subjected to different in-situ stress regimes and geological processes, determination of their mechanical characteristics is a difficult issue.

In response to the lack of suitable failure criterion for rock masses, Hoek and Brown (1980a, b) suggested an empirical nonlinear rock mass failure criterion to estimate the strength of rock masses to be used in analysis and design of rock engineering structures, such as tunnels, slopes and others. The Hoek-Brown (H-B) criterion is based on the following assumptions: (i) rock mass exhibits an isotropic behaviour, (ii) intermediate principal stress has a negligible effect on strength of rock, and (iii) strength of jointed rock masses is not sensitive to the rate of loading (Hoek and Brown 1980a). The originators of this criterion also suggested that it can be used for determining the strength of rock masses consisting of intact rock blocks separated by intersecting discontinuities in low confinement environments where the behaviour is controlled by block sliding and rotation without significant degree of intact rock failure. The criterion has been commonly used for the above purposes over the past forty-five years and repeated revisions were also made to the criterion (mainly Hoek and Brown (1988) - Updated criterion; Hoek et al. (1992) - Modified criterion; Hoek and Brown (1997) - Generalized criterion). Since the original criterion is too optimistic for slope design, the generalized criterion was updated in its 2002 and 2018 editions (Hoek et al. 2002; Hoek and Brown, 2019) and the concept of undisturbed and disturbed to provide a method for downgrading near surface rock masses and RMR to estimate H-B parameters were introduced in the updated criterion. The "*a"* parameter was introduced in the modified criterion (Hoek et al. 1992) to produce a criterion with zero tensile strength. In the generalized criterion (Hoek and Brown 1997), Geological Strength Index (GSI)

replaced to RMR of Bieniawski (1989) as a geological data input and the linear regression method for estimating the equivalent Mohr-Coulomb cohesion and friction angle was introduced. In the 2002 edition of the criterion, new derivations of relations between the parameters m_b(reduced value of material constant m_i),s and a (material constants) and GSI were provided and the disturbance factor (D) was introduced to account for blast damage and stress relaxation in slopes. The guidelines given in the 2002 update of the criterion mostly remained same in its 2018 update (Hoek and Brown 2019).

Although it is widely used in practice, repeated updates to this criterion have led to insufficient awareness among practitioners of certain issues, such as "the criterion is only based on tri-axial test data on Panguna andesite from an open pit in Papua New Guinea (Hoek and Brown 1980a, b) and no additional test data on rock mass strength was used for supporting its updates", "its origin, scope and evolution", "uncertainty and influence of parameters", and "scale effect". The main limitations and/or difficulties of the failure criterion are briefly discussed in the following paragraphs.

Until the publication of the 2002 update of the criterion, the switch at GSI=25 was arbitrarily and validity of this boundary value has not been confirmed by case studies. Although the parameters s and a are sensitive to the GSI value, according to Hoek and Brown (1997), the exact location of this switch has negligible practical importance. Based on the stability analysis of a hypothetical drained slope consisting of rock masses with the GSI values between 20 and 30 using the generalized H-B criterion, Sönmez and Ulusay (2002) indicated that a strength gap appears between the failure envelopes of rock masses with GSI values of 25 and 26 (Fig. 12a), suggesting that this situation probably arises from the equations employed for the estimation of a and s at GSI<25 and GSI>25. In addition, the strength envelopes of the rock masses with GSI values smaller and greater than 25 intersect each other at certain normal stress levels (Fig.12b) . In other words, this situation suggests that a weaker rock mass may possess higher strength than that of a stronger one when the normal stresses corresponding to the intersections are exceeded. To overcome these difficulties in the generalized H-B criterion, some approaches and improvements to the estimates of rock mass parameters have been suggested by Sönmez and Ulusay (2002) and the validity of the proposed improvements has been checked by their application to different rock masses and to hypothetical and real slope cases.Their analyses showed that s should be estimated from the suggested equation which considers the effect of rock mass disturbance and GSI values between 5 and 100, regardless of any switch value for the GSI. In addition, it was also validated that the boundary of GSI < 25 suggested by the criterion should be replaced by GSI < 30 to overcome the difficulties related to the estimation of the parameter a (for GSI > 30 a = 0.5; for GSI < 30 a= 0.65 - (GSI / 200)).

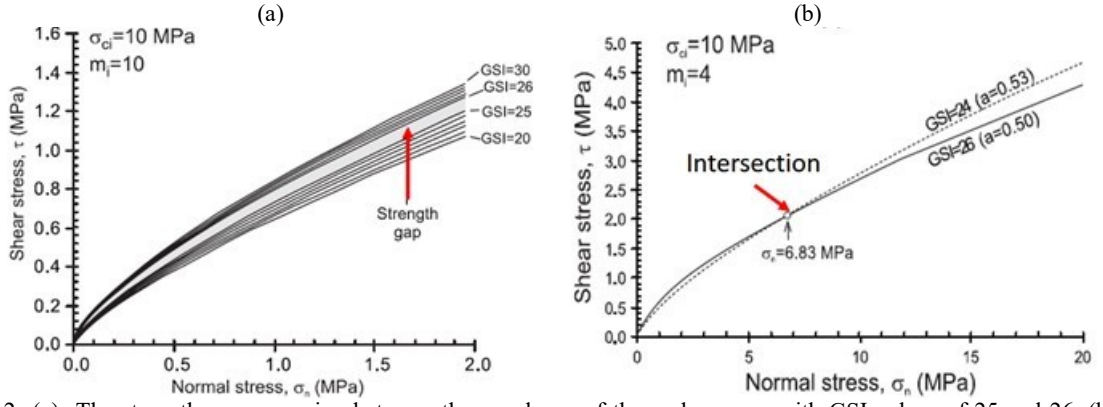


Fig. 12 (a) The strength gap appearing between the envelopes of the rock masses with GSI values of 25 and 26, (b)) an example illustrating the effect of m_i on intersection of the failure envelopes of a rock masses when the generalized H-B criterion is used (Sönmez and Ulusay 2002)

The guideline for estimating the disturbance factor, which shows some similarities with that proposed by Sönmez and Ulusay (1999, 2002), was proposed by Hoek et al (2002) who left decision on the severity of disturbance to the users. However, when compared to the guidelines for the disturbance factor (D) suggested by Hoek et al. (2002). the disturbance factor (d_f) suggested by Sönmez and Ulusay (1999) provides various degrees of disturbance severity (considering the type of excavation, such as mechanic excavation, blasting) based on the disturbance effects described by Kendorksi et al. (1983).

The original qualitative GSI chart (Hoek 1994; Hoek et al. 1995) was later modified by Hoek and Brown (1997), who added GSI contours to the chart without meaningful numerical axes to determine a basis for positioning on the chart. Then the categories of "laminated and sheared weak rocks" and "intact-massive rock" were included in the GSI chart by Hoek et al. (1998), Hoek (1999) and Hoek

and Marinos (2000), respectively. Due to the qualitative nature of the GSI charts suggested in the updates of the criterion, there are some subjectivity and uncertainty in their application. In other words, it is possible to estimate different GSI values from the charts for the same rock mass by different persons depending on their personal experience. Therefore, some researchers have provided approaches to the quantification of GSI chart with numerical axes and these chronologically include Sönmez and Ulusay (1999, 2002), Cai et al. (2004), Russo (2009) and Renani et al. (2019). Due to lack of the parameters with related interval limits to describe discontinuity surface conditions of the discontinuities and the rock mass structure in the GSI System, two terms namely, "structure rating (SR)".based on volumetric joint count (Jv) and "surface condition rating (SCR)" estimated from the parameters such as roughness, weathering and infill, were suggested by Sönmez and Ulusay (1999, 2002) in the quatititative GSI chart they recommended. The surface conditions given on the X-axis of the GSI chart recommended by the originators of the H-B criterion are only based on descriptions which can cause incorrect assessments. For example, slickendisiding (in "very poor" surface condition) should be "highly weathered and with clay filling". But this description does not consider that rougher discontinuity surfaces may have clay filling and be weathered (Barton 2025).Therefore, the quantitative GSI charts based on separate rating values selected for each dicontinuity surface condition provide estimation of more representative GSI values.

A quantitative GSI chart using RQD and R (adopted from RMR rock mass classification system of Bieniawski (1989) was recommended by Hoek et al. (2013) and they mentioned that this quantitative chart is applicable to tunnels with spans of 10 m and slopes with a maximum height of 20 m and lower GSI values should be selected for larger scales. This suggestion also causes an uncertainty for practitioners. Based on the two case studies where it has been possible to compare the results of the available quantification schemes with back-analysed strength from large scale open pit instability, Duran (2016) reported that the quantitative GSI charts of Sönmez and Ulusay (2002) and Russo (2009) have provided the values of GSI to those provided by assessing RMR_{89} and utilizing the equivalance suggested by Hoek et al. (1995). Duran (2016)also recommended that review of further case studies to allow refinement of the quantification schemes and to assess their applicability to large scale open pit slopes are needed. Hoek et al. (1998) also proposed a new rock mass category to accomodate thinly foliated or laminated, folded and predominantly sheared weak rocks of non-blocky structure. But previously Hoek (1994) emphsized that the H-B criterion is applicable to "intact rock" or to "heavily jointed rock masses" which can be considered homogeneous and isotropic. Since the strength and deformability characteristics of such rock masses are governed by the displacements along numereous very thinly spaced pre-sheared and slickensided foliation planes, introducing of this new category into the GSI chart seems not to be realistic. Therefore, this category was not included into the quantitative GSI charts proposed by Sönmez and Ulusay (1999, 2002). But this rock mass category was used by the H-B failure criterion until 2013 when its originators acknowledged that the failure of such rock masses should not be included within its applicability range, and they therefore, excluded both this category and the category of massive (intact) rock masses from the GSI chart.(Hoek et al. 2013). In addition, Marinos and Hoek (2000) proposed a GSI chart for heterogeneous and tectonically deformed sedimentary rocks and its extended version was published by Marinos and Carter (2018). On the other hand, GSI charts for ophiolites (Marinos et al., 2005) and tectonically undisturbed molassic rocks (Hoek et al., 2005) were also proposed to including tunelling projects in northern Greece. In the GSI system, very poor rocks are generally defined by GSI value below 27 (Morales et al. 2004) or 25 (Hoek and Brown 1997). In order to better characterize the poor rock masses without disregarding its original concept, the modified GSI chart, which can be used for poor and very poor rock masses with GSI ranging between 6 and 27 and is integrated with existing GSI chart, was also recommended by Osgui et al. (2010) as an assistant tool for GSI estimation for such rocks.

The H-B constant m_i, which defines the non-linear strength envelope of rock material, depends on the frictional characteristics of the minerals involved by rock material. Therefore, its accurate determination is a great influence in the estimation of strength of brittle rock materials. Hoek and Brown (1980a-b, 1997) suggested regression analysis of results obtained from the numereous tri-axial tests at least five sets of tests under different $\sigma_3$values. Heok and Brown (1980a) also proposed a guideline to estimate m_i as a range only based on rock type and its latest version with a more detailed lithological classification was proposed by Hoek (2007). Since the m_i values are given in a range in this guideline, it is difficult to select exact m_i values. In the absence of tri-axial tests, there are different methods, such as R-index (the ratio between σ_c and σ_iHoek and Brown 1980a,b; Mostyn and Douglas

2000; Richards and Read 2011; Mehrishal et al. 2015), crack stress initiation model, which is a special form of R-index (Cai 2010), UCS-based model (Shen and Karakus 2014; Vasarhelyi et al. 2016), tensile strength (TS) model (Wang and Shen 2017) and combination (UCS-TS) method (Arshadnejat and Nick 2016) are available to estimate m_i. However, some of these prediction methods are based on limited data and limited rock types. Recently, Karakul and Ulusay (2022) developedϕ-based method using three σ_1 and σ_3 data pairs from tri-axial tests and Davaranpah et al. (2022) statistically analysed a database and introduced new material constants for igneous, sedimentary and metamorphic rocks. Both studies revealed that the ranges of calculated and predicted m_{ii}values are much larger than those of the m_i ranges in the m_i table proposed by the originators of the failure criterion and it is generally still inappropriate. These two recent studies on m_i given above indicated that R-index approach with new material constants provides the best fit for all studied rocks and is not influenced by rock type (Davaranpah et al. 2022), and the ϕ-based method proposed by Karakul and Ulusay (2022) gives the best estimation for m_i and only 2 out of 121 sets of data fall out of the lines between m_i(±calculated) /± 10°.Readers are also encouraged to consult other sources concerning evaluations of the H-B failure criterion (e.g. Renani and Cai, 2022; Barton, 2023, 2025; Aydan et al., 2012).

In his paper on mechanical properties of rock masses, as one of the two originators of the H-B failure criterion, Brown (2008) strongly recommends the following issues when using this criterion: (i) it is not intended to replaced and depend for its successful application on, careful thought, detailed appraisals of site geology, carefully conducted site characterization and laboratory test programs, and a detailed evaluation of mechanics of the engineering problem being addressed, (ii) both the criterion and its derivatives should not be used as "black box" or "cook book" approaches, (iii) as done in some cases, $m_i$values should not be taken from tables in final design, instead it should be determined from required tests, and care should be paid to the selection of parameters, such as disturbance factor (D). The author of this paper considers that the above comments by Brown (2008) are very useful and important.

2.5.3 Shear strength of bimrocks

Main factors affecting the overall mechanical properties of BIMROCKs are: (a)mechanical properties of matrix, (b) Volumetric Block Proportion (VBP), (c) shape and size distribution of blocks,and (d) orientation of blocks relative to failuresurfaces. Due to the great spatial, lithological and mechanical variability of bimrocks, neglecting the contributions of blocks, choosing instead the strength and deformability of the weak matrix may result in too conservative and expansive for many bimrocks in terms of slope design (e.g. Medley 1994, Lindqusit 1994, Kim et al. 2004, Wakabayashi and Medley 2004, Medley and Zekkos 2011).But Lindquist (1994) showed that the strength of bimrocks depends on volumetric block proportion (VBP) and when VBP ≤ 25% it is considered that the material is soil or weak rock, and when VBP > 75% there is little further addition in strengthbecause blocks start to touch each other (Fig.13a). Based on the experimental studies on artificial bimrocks, some authors(e.g. Lindquist 1994, Sönmez et al. 2006, Kalender et al. 2014) proposed empirical strength criteria assuming that bimrocks are homogeneous and isotropic rock masses with equivalent mechanical properties which can be defined according to their block proportions and strength parameters of matrix. One of these empirical criteria is given in Fig. 13b as an example.

In order to overcome the size limitation of laboratory specimens, in-situ shear box tests (BIM Test) on shale-limestone caotic complex Bimrock from an open pit in Italy was also conducted by Coli et al. (2011). The in-situ testing apparatus cosists of a flat frontal steel plate with two horizontal bottom and of two side plates, and all the plates are 80 cm wide, 50 cm high and 1.5 cm thick and is connetced to a hydraulic jack (Fig. 14a). A constant displacement of 0.05 mm/sec was applied during the test. In general, the failure surfaces show tortuous paths, characterised by a step profile and by abrupt local slope variations due to the presence of protruded calcareous blocks (Fig. 14b). Due to unknown saturation and pore pressure, the upper sliding area was divided into slices in order to calculate the shear strength parameters, and a pair of c and ϕ parameters was calculated for each test specimen with the aid of limit equilibrium method of analysis performed on each cross-section of the sliding surfaces, and using the results VPB-ϕ and VPB-$c$graphs were drawn (Fig. 14c.1-c.2). As seen from these graphs, ϕ increases with the increase in VBP with a high coefficient of determination (R^2= 0.85) (Fig. 14c.1), while cohesion shows a strong decrease from test P2 to test P1. This sudden drop in cohesion is observed at the VBP threshold of 20-25% which represents the threshold value between a matrix-controlled and a block controlled behaviour according to previous studies (e.g. Lindquist 1994, Kim et al. 2004, Sönmez et al. 2006). However, Coli et al. (2011) remarked that the bimrock's shear

parameters obtained through the in-situ shear tests are not effective but operative parameters under natural conditions, and very likely, partially drained conditions; therefore, the proposed comparison has the only purpose of giving an indication about the respective magnitudes.Napoli et al. (2021) aimed statistically to investigate and compare the stability of theoretical slopes in the most widespread bimrocks and used model bimrocks obtaining blocks with variable shapes, dimensions, arrangements and contents. Their results based on fifteen configurations indicate that block shapes and orientations significantly affect the stability of slopes in bimrocks only when the block contents are greater than 40%, and the blocks inclined 0°to the horizontal provide the most tortuous and irregular failure surfaces and, consequently, the highest safety factors.

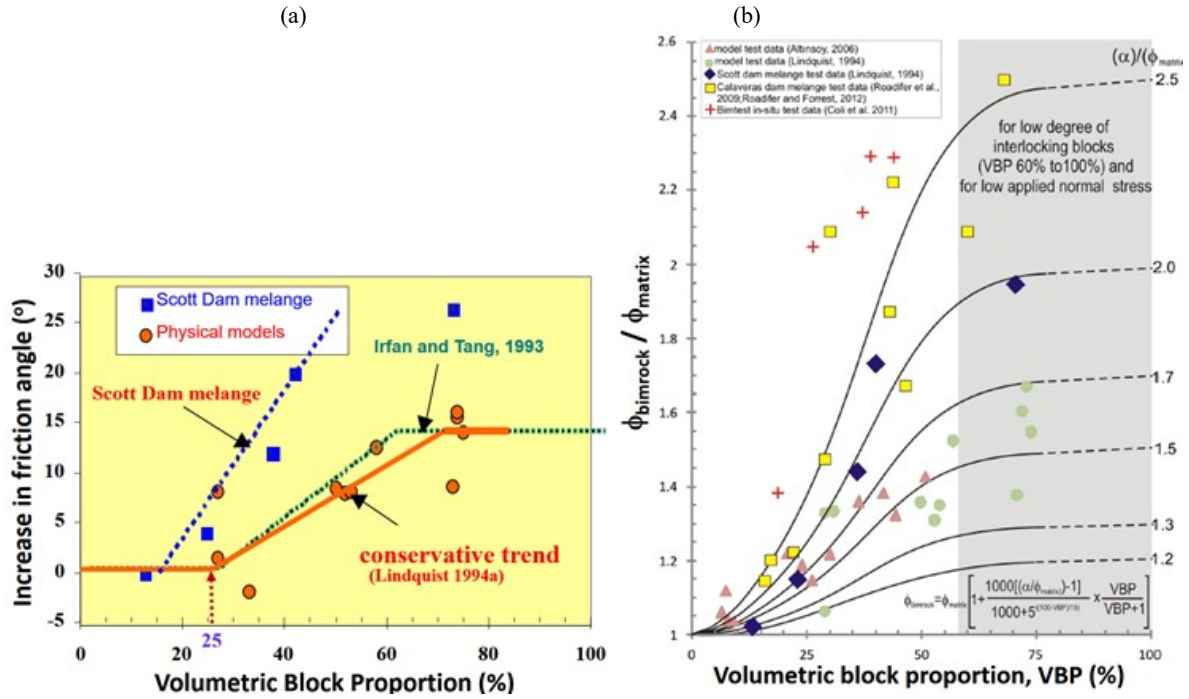


Fig. 13 (a) Strength of bimrocks increases with volumetric block proportion (Lindquist 1994), (b) the graphical presentation of the empirical approach for predicting $\phi_{bimrock}$ (Kalender et al. 2014)

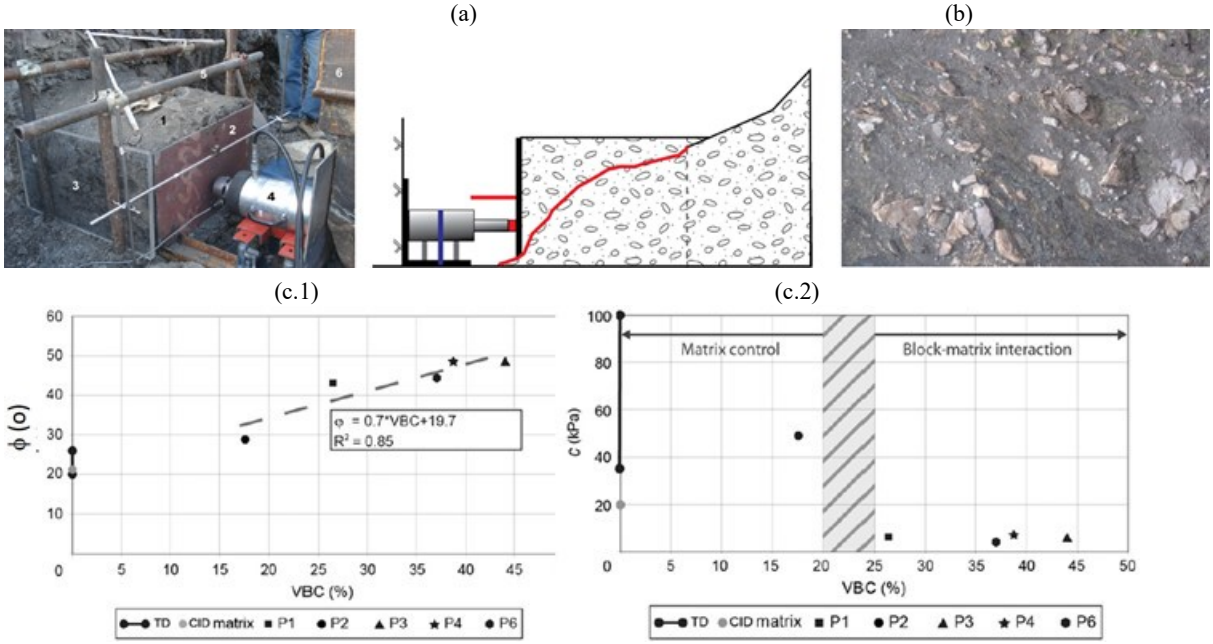


Fig. 14 (a) In-situ shear test apparatus used for a bimrock and schematic representation of the test and failure surface, (b) shale–limestone chaotic complex bimrock. and (c.1) correlations between friction angle and VBP, and (c.2) cohesion and VBP (Coli et al. 2011)

By considering that there is still no general concensus on the approaches/methods for the site characterization and determination of geomechanical properties of bimrocks, the author of this keynote recommends the followings for future studies on bimrocks: (i) laboratory tests on larger sized specimens should also be conducted, (ii)in case of huge blocks greater than the size of the test device, the dimesion of the in-situ BİM shear test cannot be a practical way in all cases, (iii) since real

bimrocks generally contain non-spherical blocks, assumption of block sphericity has limitation, therefore, future studies on other block shapes (such as elipsoids etc.) are neccessary,(iv) influence of structural features in bimrocks should also be investigated, (v) in laboratory experiments and numerical solutions, matrix is considered as a homogeneous rock mas, however, when it is anisotropic (such as laminated shale or slate)matrix anisotropy should not be ignored, (vi)since there existonlya few study addressing the behaviour of bimrocks under seismic loading (their dynamic properties), this issue also needs further studies.

2.6 Hydrogeological conditions and conceptual hydrogeological model

Pore water pressures acting within discontinuities reduce the effective stress, which in turn decreases the shear strength of discontinuities and/or the rock mass, adversely affects slope stability. Therefore, the hydrogeological conceptual model, which complements thegeotechnical model of a rock slope, can be critical to the slope design. By considering this, a clear understanding of the water pressure distribution and reducing any pressure is an essential controlling factor and should not be overlooked especially in the design of large and deep slopes. This model is essential for clearly understanding the distribution of pore water pressure and for determining appropriate measures to reduce it. The importance of hydrogeology has rapidly gained momentum particularly 2000s to meet the requirements of deeper and larger open pit mining projects. The advent of new monitoring systems, and particularly the use of grouted-in vibrating wire piezometers (VWP) has contributed to a practical understanding of the factors that control the behaviour of water pressure in the pit slopes (Read and Stacey 2009).

Mining can cause changes to the hydraulic properties of the slope forming materials due to: (a) changes induced by blasting, creating an over-break zone (5-40 m thick) and fluid-to-solid hydro-mechanical changes at deeper elevations; (b) litho-static unloading by removal of the overburden may result in an expansion of the fracture aperture and pore space and depending on these pore pressure reduces. However, reduction of high pressure seldom occurs in fractured rock because transient flow of water occurs towards depressurized area to equalize the pressure drop; (c) deformation resulted from drainage and reduction of pore pressure; (d) changes the hydraulic properties of the slope forming materials due to slope failure (Beale 2013). Except for rocks with hydraulic conductivity coefficients of 10^{-8} m/s or greater, the traditional practice of simply drawing a phreatic surface in slope cross-sections and assuming hydrostatic pore pressures below it is now considered an incorrect approach for deep and large open pits (Read and Stacey 2009). In addition, the research by the Large Open Pit (LOP) project indicated that effect of deformation in pore pressure in fractured rocks became important in zones where the permeability is less than 10^{-8} m/s (Read and Stacey 2009; Galera et al. 2009). As illustrated in Fig. 15a, depending on the depth of the pit, particularly in deep open pits, pore pressures may vary, and partially saturated zones may exist. If the geology is characterized weak points in a slope, they may create future instability and should be identified early in the investigation program, so that hydrogeology can be focused on these areas, instrumentation can be installed to measure pore pressure and geotechnical parameters at the same interval in the same drill hole (Fig. 15b). Therefore, the hydrogeological conditions cannot be represented by a single, uniform groundwater level. For this reason, the involvement of mining hydrogeology experts is of great importance in assessing the stability of open pit slopes.

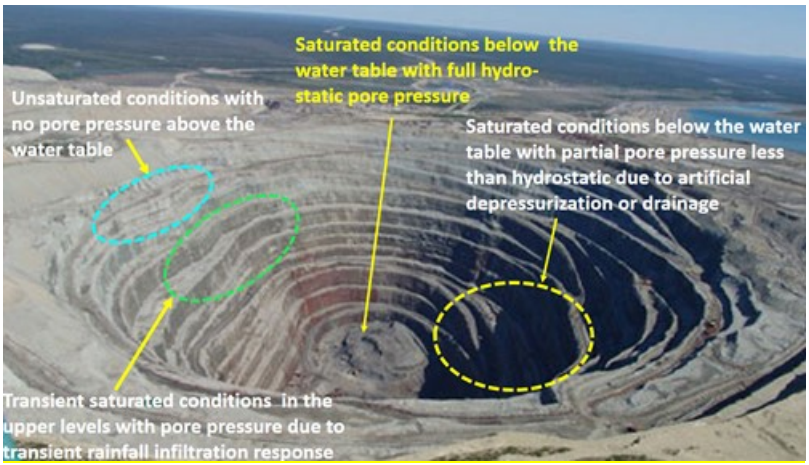
For the geotechnical models, the development of a good groundwater conceptual model is an important part of the rational design of deep and large open pits. A conceptual groundwater model is a realistic, defensible, albeit simplified representation of the hydrogeological system (Beale et al. 2013). By collecting and evaluating data obtained through modern hydrogeological measurement and monitoring techniques, a hydrogeological conceptual model can be developed and integrated with the geotechnical model. Based on regional and mine scales, the conceptual hydrogeological model considers the three regimes given below (Beale et al. 2013): (a) geology (stratigraphical units and their variations, locations and interactions, geological structures), (b) hydrology (natural and artificial recharge to the groundwater system, natural discharge from the groundwater system, groundwater-surface water interaction), (c) hydraulics (hydraulics of the main geological units, groundwater flow paths).Such a model should also be used for designing appropriate drainage solutions. Two examples of hydrogeological conceptual model are given in Fig.16.

The following types of groundwater control are commonly used in open pit slopes:

(i) Dewatering (pumping) wells: They are used general mine dewatering and slope depressurization. When a general lowering of the groundwater level in permeable slope forming material is needed vertical pumps are used outside the crest or within the pit area.

(ii) Horizontal and steep angle (or vertical) drains: They are commonly used to relieve groundwater pressure behind the pit slope. Sub-horizontal drain holes can be considerably effective in hard rock slopes if they are long enough to reach the rocks mass in the vicinity of potential failure surfaces. In large slopes, in order to achieve this goal, drain holes with a length of 200 to 300 m may be required. Vertical or angled drains can be used where a large hydraulic gradient is developed within the slope.
(iii) Drainage tunnel: They are constructed behind the slope or underneath the pit and their use particularly increased in large open pits. Their construction and operation without interfering mining and their construction earlier in a mine and providing active depressurization are the main advantages of this control method.
(iv) Cut-off system (slurry wall, grouting of permeable fracture systems): They are used to reduce the amount of water reaching the pit with the aid of reducing the permeability of a particular rock unit or a zone along the flow path.
(v) Removal of the water using in pit-sumps: It can be applied when the rock mass in the pit slope has relatively low permeability.

(a)



(b)

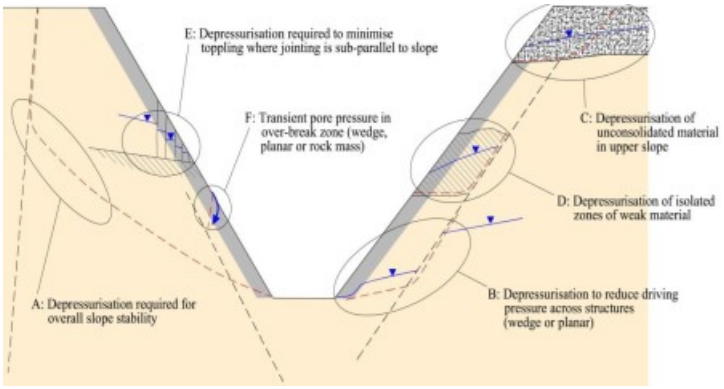
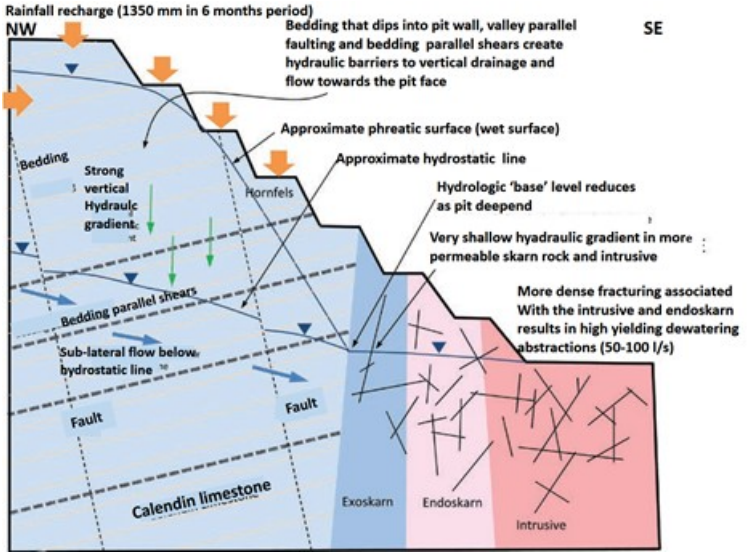


Fig. 15(a) Possible hydrogeological conditions at different depths of a deep open-pit slope (different groundwater conditions are shown on photo of an open pit (*mining-technology.com*)), and (b) focusing the hydrogeology program on potential failure modes (source: LOP project; Beale 2013).

In most large open pits, a combination of the above measures is used in different sectors or at different stages of the mine life. A brief information for the groundwater control conducted at Kışladağ gold open pit (most recently 440 m deep) in Türkiye (Furtana et al. 2023) is given in this paragraph as an example. Although the east side of the pit shows good drainage towards the pit sump, the west side shows slow drainage due to orientation of the joint sets and the youngest intrusion with lowest hydraulic conductivity. Two flight submersible pumps (250 m^3/h maximum pumping capacity of 150 m head) are used in the sump (Fig.17a). In 2012, three pumping wells were drilled, and yield of water was between0.2- 5 lt/s, and five test wells were drilled in 2020 for the assessment of hydraulic conductivity of various parts of the pit. Trial advance pumping wells at the pit bottom were drilled to help dewatering at the pit floor and three of them were converted to monitoring wells. After the implementation of the monitoring and test results, long horizontal drains with length of longer than 250 m were drilled (Fig. 17b) in the southwest of the pit to increase the drawdown rate and to improve the pit slope stability and the current monitoring methodology using four VWPs sensors inside the pit worked well (Fig. 17c). The drains exceeded expectations in the SW of the pit and maximum

drawdown of 50 m was observed in monitoring wells (Fig.17d). The results of this dewatering application revealed that there is no pore pressure on production levels of the Phase 4pit until the 650L which improved the stability of the Phase 3 pitwalls.

(a)



(b)

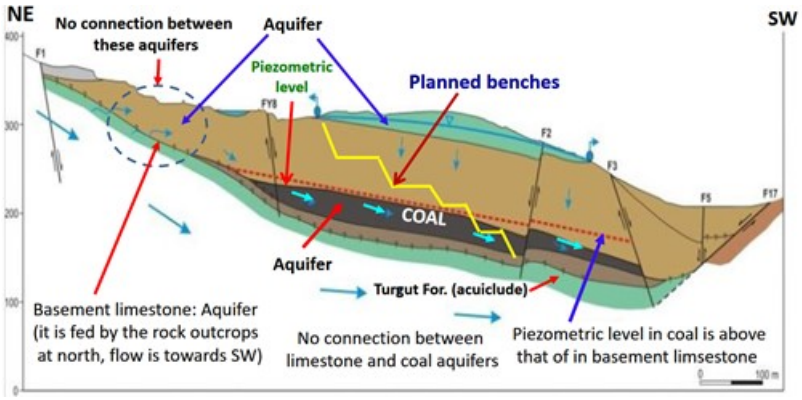
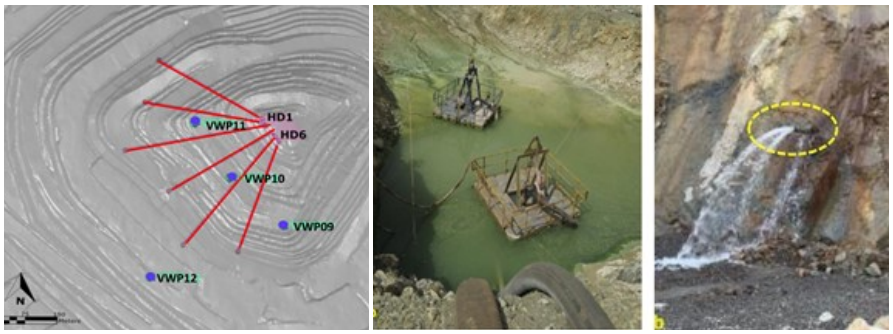


Fig. 16 Hydrogeological conceptual models prepared for (a) the Antamina pit in Peru (Pottie et al. 2013) and (b) a lignite open pit in Türkiye (arranged from Ulusay et al. 2013)

(a) (b) (c)



d)

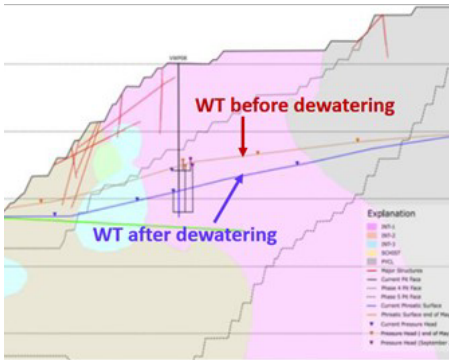


Fig. 17 (a) Wells (VWP) and drainnholes (HD) , (b) location of pumps at 650L,(b) groundwater flow from the horizontal drain HD-08, (d) vertical cross-section of HD02 showing water tables before and after dewatering (Furtana et al. 2023)

3 Geotechnical model and geotechnical domains

The objective of open pit slope design takes into consideration an analysis of the overall slope stability of open pit wall, inter-ramp angle and height, and bench design, and provides an optimal excavation design that leads to the steepest possible slopes while ensuring safety, ore recovery and financial returns are maximized. The formalized geotechnical model has been largely developed about over the past 50 years and is the basis for all open pit slope design and consists of following four sub-models (components) (Stacey 2009): (i) geological model, (ii) structural model, (iii) rock mass model (properties of materials and discontinuities), and (iv) hydrogeological model (Fig. 18).

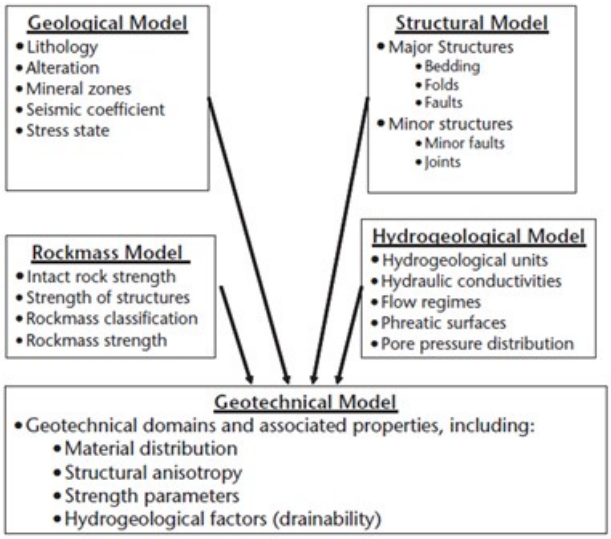


Fig. 18 Component information and output from the geotechnical model (Read and Stacey 2009)

The *geological model*provides presentation of a 3D distribution of the types of the lithologies forming the pit walls and the degree and type of weathering and alteration which can affect material properties. Core logging is also emphasized as a critical source of data for subsurface model support and logging process is also recommended. This model also helps better understanding of the regional geological setting and the genesis of the mineralization and is generally prepared by the mine geologists who primarly focus on the mineralizaton and may have a somewhat different view of the slope forming geological units.For example; waste is of little interest to the mine geologistand hence rock types may be poorly differntiated even their mechanical properties, structure and fabric may vary. In addition, although a small amount of highly altered feldspars may be of significance to the ore genesis, alteration does not necessarily weaken the rock, silicification can increase the rock strength. Therefore, the slope designer should be fully aware of the basis and terminologyused in the geological model and discuss it with the mine geologist when necessary.

The *strucural model*is divided into two sub-models, the major structures (faults, folds and shear zones at the inter-ramp to overall pit scale) and structural fabrics or discontinuites (joints, bench scale faultsgenerally at bench scale). Major structures should be continuous both along strike, down dip and persist over multiple benches. Since structural fabric has limited persistence but close spacing, attention should also be paid in design at bench scale and inter-ramp stacks. Major structures should be characterized in terms of size, nature and infillings by structural geologists. However, structural fabric is generally investigated by rock engineer and/or engineering geologist, and coordination between both professions is necessary during the construction of the major structural model.

The purpose of the *rock mass model*is to determine the engineering properties of the rock mass, structures and intact rock to be used in the stability analyses for the pit design. The most commonly used rock material properties are index properties (such as point load strength index, porosity, unit weight, P- and S-wave velocities) and mechanical properties (uniaxial and 3-axial compressive strengths, tensile strength, Young's modulus and Poisson's ratio). In open pit slope stability assessments, determination of shear strength of structural defects (faults, joints, beddings etc.)by both laboratory and field shear tests. In addition, thestiffness of structural defects for numerical modeling purposes are also determined.Rock mass classification for the slope forming materials is also considered in this model. Although the laboratory methods for testing small scale samples are well defined and standardized, the extension of the results to deep slopes several hundred meters high

needed some advances. The empirical H-B criterion of which limitations are discussed in section 2.5.2,became a tool to assess the strength of large rock masses both for slopes and underground openings. The two rock Laubscher criteria, “rock mass strength” and “design rock mass strength”, were proposed for use in only underground mining (Laubscher and Jacubec 2001). However, as a part of Large Open Pit (LOP) project, which is an international research and technology transfer project on the stability of rock slopes in open pit mines (lopproject.com), some advances have been made in development of more rigorous approach to the determination of rock mass strength DFN and Syntetic Rock Mass (SRM) (Potyondy and Cundall 2004) models. Most recently, as an alternative to methods based on RMR assessments, CNI (Call & Nicholas Inc.) (Cylwik et al 2022) proposed a new empirical rock mass failure criteria related to the strength of the rock mass directly to the degree of fracturing in the rock mass through a combination of the intact rock strength and the natural fracture strength as afunction of RQD. However, due to some data limitations, this criterion couldn’t find a wider application yet. It is also noted that back-analysis of open pit slope failures and even of stable slopes can be usefuul in the determination of material properties.Building the *geotechnical model* recommended by Read and Stacey (2009) is a step-by-step process of bringing successive layers of individual or combination of datasets into a 3D solid model using a modelling system.

A *geotechnical domain* is a volume of rock that shares similar geological, geomechanical and hydrogeological characteristics. Instead of the entire open pit as a single block, design engineers divide it into the domains to apply specific design parameters and account for structural controls to ensure pit slopes remainsstable while maximizing ore recovery. A geotechnical model showing the geotechnical domains at an open pit in Africa is given in Fig. 19a as an example.In addition, the importance of the geotechnical model is illustrated by a hypothetical example shown in Figure 19b. In this example, weathered dacite occurs at the upper levels, underlain by fresh and intact dacite, and a fault dipping in the same direction as the slope is present at the rear. In addition, a weak chloritic zone intersecting the fault exists at the base. The shear strength parameters of these units and weakness planes are characterized by the relations:$c_1 >> c_2 \approx c_3$ and $\phi_1 >> \phi_2 \approx \phi_3$.When only the weathered daciteis considered, the slope, behaving like a soil mass, may exhibit a circular-type failure within this material. However, it is also possible that the rear fault connects with the weak chloritic zone dipping toward the excavation, forming a multi-surface sliding mechanism. This is because the shear strength of both weakness zones is lower than that of the weathered dacite.In such a case, it would be appropriate to analyse both possible instability types separately and to decide based on the analysis results which failure mechanism is the most critical. The stabilization measures should then be designed accordingly.

(a) (b)

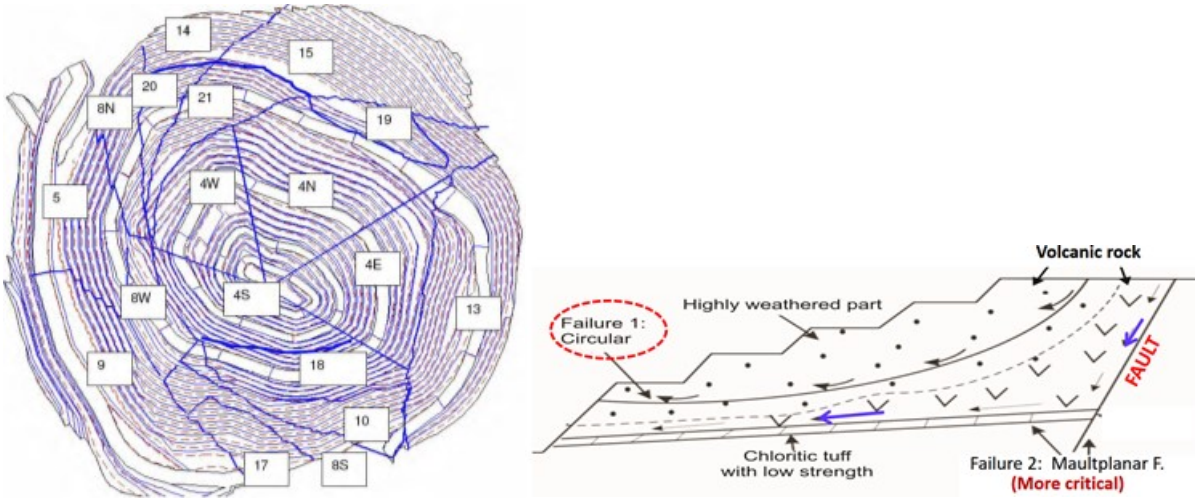


Fig. 19 (a) Geotechnical design domains in relation to the final pit shell of the Cadia Hill Open Pit (Pothitos and Webster (2006), after Franz (2009)), (b) a hypothetical cross-section forming the basis of a geotechnical model for an open-pit slope drawn by the author.

4 Influence of in-situ stresses on deep open pits

In-situ stress is the stress prior to any excavation, results from the weight of the overburden and tectonic stresses in the region, and all rocks are subjected to in-situ stresses. Open pit excavations cause changes in the existing stress field due to stress relief, and these changes can have significant

consequences for the behaviour of rock masses. In rock slopes, while pit walls generally become distressed close to the slope face, stress concentrations develop at the toe of the slope. In general, the following assumptions are made for open pit slopes: (a) the slope-forming rock mass is influenced only by gravity-induced stresses, and (b) these stresses are not large enough to exceed the strength of the rock mass and cause failure (Read and Stacey 2009).Therefore, it is thought that in-situ stresses are small and not measured. In addition, all analysis methods of limit equilibrium are based on gravitational loading and lateral stresses are not included in the analyses. Although neglecting pre-mining in-situ stresses may be acceptable for shallow open pits, it has been shown to be invalid for deep open pits in hard rocks. But the main difference between shallow and deep open-pit slopes is that rock mass damage due to in-situ stresses develops much more extensively in deep open pits. As excavation proceeds, stress redistribution occurs and the rock mass is subjected to successive stress variations. Therefore, in increasingly deepening open pits, measurement of in-situ stresses became important, particularly where the stresses at the bottom of the pit are likely higher than the strength of the rock.

The Palabora mine, where open pit operations began in 1966, is an example of a slope failure resulted from change in stress conditions (Fig. 20a). The development of block cave operation was decided in 1996. While the slopes of this open pit mine has been completely stable during the open pit mining to a depth of 800 m, a large failure with dimensions of 800 m high and by 300 m long behind the NW wall of the pit occurred during the transitioning of the open pit to underground mine (Fig. 20b). The caving caused a change in the stress environment which resulted in development of a major failure. Based on the study by Moss et al. (2006), there has been interaction between the open pit and the cave, initially the impact of the open pit on the cave was to concentrate stress within the ground between pit floor and cave, and increase stress level resulted in cave propagation by causing existing fractures to extend and formation of new fractures.

(a) (b)

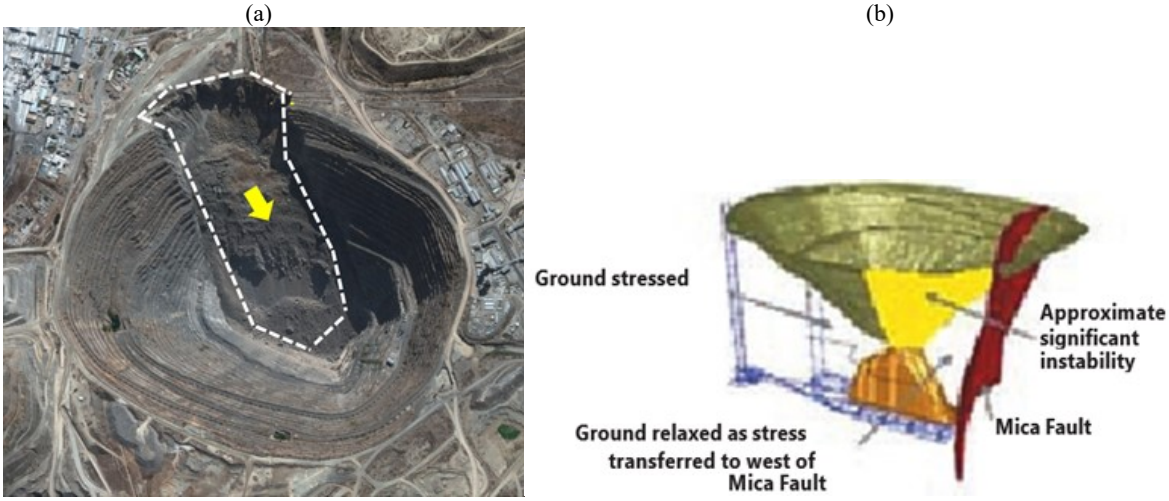


Fig. 20 (a) Quickbird image of the NW wall failure at Palabora mine, S. Africa (Woo et al. 2012), (b) zone of slope instability in this pit (Moss et al. 2006)developed during the transitioning from open pit to underground mass mining.

In very deep and ultra-deep open pits, the stress state around the excavation becomes increasingly complex, with zones of both very high and very low stresses occurring within the rock mass. Measurement of vertical stress is generally estimated by the equation "$\sigma_v = \gamma z$" where γ is the unit weight of the rock, and z is the depth from the surface. Measurements of vertical stresstaken from mining and civil engineering projects around the world generally confirm the validity of this equation (Hoek et al. 2009). however, as shown in Fig. 21a, significant scatter is observed in these measurements. Determination of horizontal stresses ($\sigma_h = k\sigma_v$; k is the ratio of the average horizontal stress to the vertical stress) is much more difficult than that of vertical stress. Measurements from different parts of the world have shown that the coefficient k tends to be high at shallow depths and generally increases with depth (Hoek and Brown, 1980a, b). Sheorey (1994) proposed an elasto-static thermal stress model for estimating k values globally, as given in Eq.1 which is used only as a preliminary approximation. Sheorey's model is a simplified one, and it should be noted that the tectonic history of a site plays a significant role in determining in-situ stress conditions.

$$k = 0.25 + 7E_h\,(0.001 + (1/z)) \qquad (1)$$

Where z depth (m)
E_h average deformation modulus of the upper crust (GPa)

Stress changes induced by successive excavation stages can lead to tensile failure of the rock, rock mass damage, and crack development, all of which negatively affect the strength of the rock. Results of simple numerical analyses performed for a deep (500 m), 40° homogeneous rock slope with two different σ_h/σ_v ratios (0.5 and 1.0) (Fig. 21b) have shown that, as the stress magnitude increases, fracture development parallel to the pit boundary also increases, and these fractures occur within the rock material between natural discontinuities.

Seismic data confirms that the deeper a slope is, the more stressed and therefore the more it fractures. Today, in very deep open pits, where high stresses are present, micro-seismic activity monitoring is used to evaluate the damage processes affecting pit slopes. Excavation-induced fracturing and crack propagation in the rock mass generate high-frequency seismic waves, which can be recorded by accelerometers and geophones. The micro-seismic monitoring performed by Lynch and Malovichko (2006) at Navachab gold mine in Namibia during period of unloading and resultant high micro seismic events to monitor a possibly problematic shear zone lying roughly sub-parallel to the slope showed that fracturing and most of the events occurred beneath the level of the pit floor (Fig. 21c). This combined with any absence of surface movements, indicates relative stability of the shear zone.

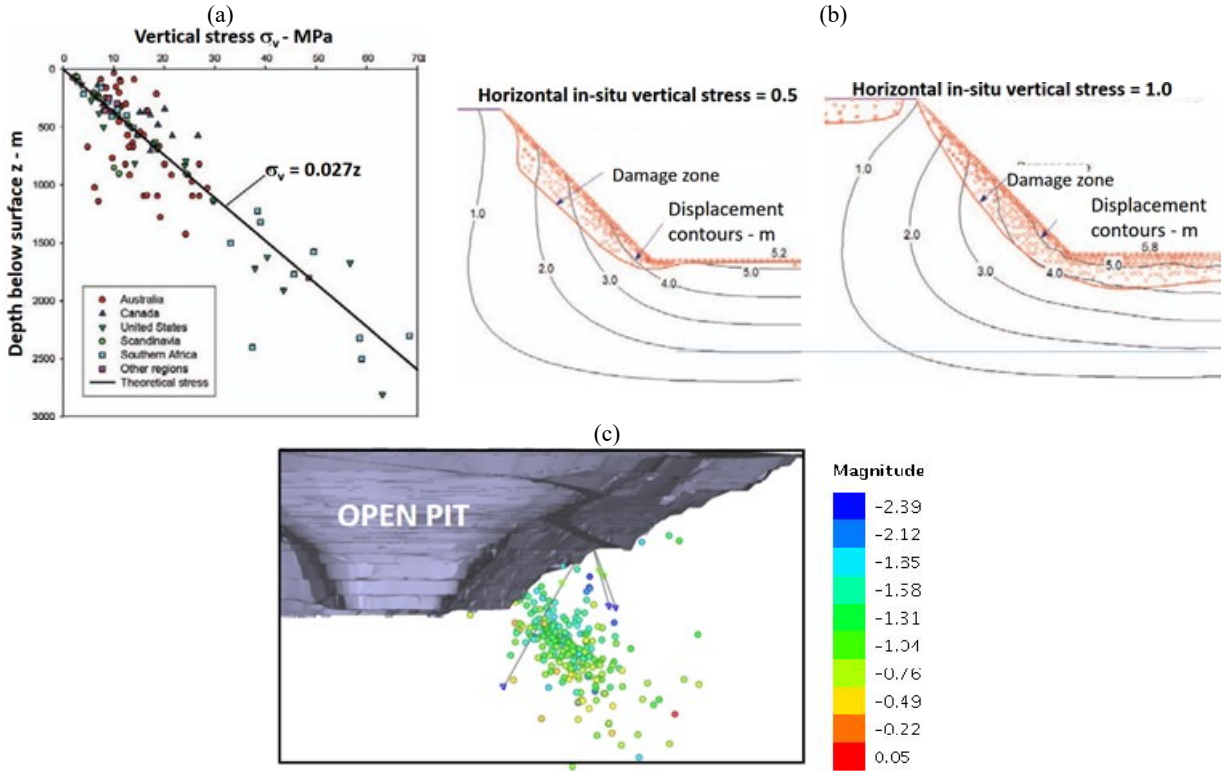


Fig. 21 (a) Measured vertical stresses from various parts of the world (Hoek and Brown 1980b), (b) predicted damage zones and displacements for slopes with *k* values of 0.5 and 1.0, respectively, in a 500 m deep and 45° homogeneous rock slope (Hoek et al. 2009), (c) seismic activity induced by overburden removal at an open pit (Lynch and Malovichko 2006).

The main direct methods for measurement of in-situ stresses are overcoring, hydraulic fracturing and, flatjack techniques, drillhole deformation gauge and CSIRO-type cell. Detailed descriptions of these tests are given in ISRM Suggested Methods (ISRM 2007) and also in Amadei and Stephansson (1997). In addition, the use of Acoustic Emission (AE) test to monitor when core is loaded to the point where new damage occurs (Villeascusa et al. 2002) increased in 2000s.It is also noted that numerical modelling has become the preferred tool for rock mechanics analysis in almost it's all fields of application today. These techniques, both 2D and 3D, also found an area of application for the assessment of stress effect on open pit slopes (e.g. Lorig 1999, Stacey et al. 2003, Sjöberg 2013). Hoek et al. (2009) discussed that the uncertainty of other parameters, such as geology, rock mass strength and groundwater conditions, has a larger influence on slope behaviour than variations in stress. However, these investigators recommend that careful consideration is required for cases with high horizontal stresses, where underground excavations are located close to a pit and for deep open pits. But based on the literature, due to the very limited number of in-situ stress measurements in open pit mines (e.g. Obara et al. 2000, Wang et al. 2025) validation studies are considerably limited, and therefore, in-situ stress measurements for deep open pits should be conducted and will be useful for the safety of these mines and for validation studies.

5 Influence of earthquakes on deep open pits

Earthquakes cause stress changes in slopes. Combination of static stresses with the dynamic stresses resulted from earthquakes may exceed available strength along a potential sliding surface and cause failures in the form of slides and rockfalls. Read and Stacey (2009) mentioned that there are no recorded cases of large scale earthquake-induced open pit slope failures which disrupted mining operations. However, more recently there have been some earthquake-induced damages were reported in open pitsin China (2008, M7.9), Chile (2010 M8.8; 2015, M8.3) and Papua New Guinea (2018, M7.5) and, except some mining infrastructure damage in the first three earthquakes and natural slope failures in the last one, none of those were significant to be recorded as causing injuries, pit slope failures and interruptions in production (Estrada and Weir 2023a). In addition, there are anecdotal reports, not officially published, of earthquake-induced open pit slope failures (Estrada and Weir 2023b; Ross 2017) in Chuquicamata Mine (Chile) prompting the adoption of advance monitoring and mitigation measure and Bingham Canyon Mine (USA) where ground movement and slope instabilities occurred and caused seismic activity. Based on te study by Azhari and Özbay (2017), who developed a database for investigating the circumstances under which dynamic loading if earthquakes cause slope failures in deep open pit mines with a long lifetime, it was concluded that open pit mines may strill experience reportable if struck by earthquakes with local depth of less than 40 km, epicentral distance less than 100 km and magnitude gretar than 6.0. In addition, their results also showed that even though the valley shape geometry and absence of top weathered layer make open pit mines less prone to earthquake-induced failures, excessive pressure in the pit slopes still carries risk of earthquake-triggered failures in open pit slopes. Factors affecting pit slope performance under earthquake loads and their consequences are summarized in Table 1.

Table 1 Factors affecting pit slope performance under earthquake load (arranged from Estrada and Weir (2023b)

Factor	Impact on slope	Consequence
Topography	Topographical amplification increased in topographical amplification at the surface and particularly slope crest and man-made topographical alterations (i.e. deep/large pits) amplifies ground motion at the crest.	Amplifies ground motion and may be detrimental if mine infrastructure is located close or at pit crest.
Slope geometry	Slope geometry (concave or convex) has a significant impact on amplifying the ground shaking of slope ridges. For improving seismic performance, natural terrain curvature of open pit is modified.	Pit slope geometry may scatter seismic waves reducing the impact of the earthquake shaking.
Rock material & mass properties	Pit slopes with contrasting lithologies or weathered material may experience increase earthquake shaking at be more risk of instability.	Slopes with stiffness contrast or inweak rock may subject to amplification
Water	Pore pressure may increase the probability of failure on a pre-existed failure surface. Dewatering in mining makes open pits less prone to failure.	Water management (pore pressure and surface water control, dewatering) may make the pits less prone to failures.
Open pit design	Typically open pit slopes are designed with a FOS ≥1.2 which may accommodate some level of earthquake loading.	FOS for static design may ccommodate some level of shaking

Based on the results from some generic analyses carried out by Toh (2010), the peak ground acceleration (PGA) at the pit crest could be amplified up to 17% from the “flat ground” PGA, with greater amplifications (over 35%) of spectral accelerations at other periods greater than zero seconds. Fig. 22 shows an example of accelerations within a 300 m high slope, where accelerations within a slip surface extending from crest to ¼ slope depth relatively constant, but average acceleration within a slip surface over the entire slope height is much lower than the maximum acceleration at the crest.

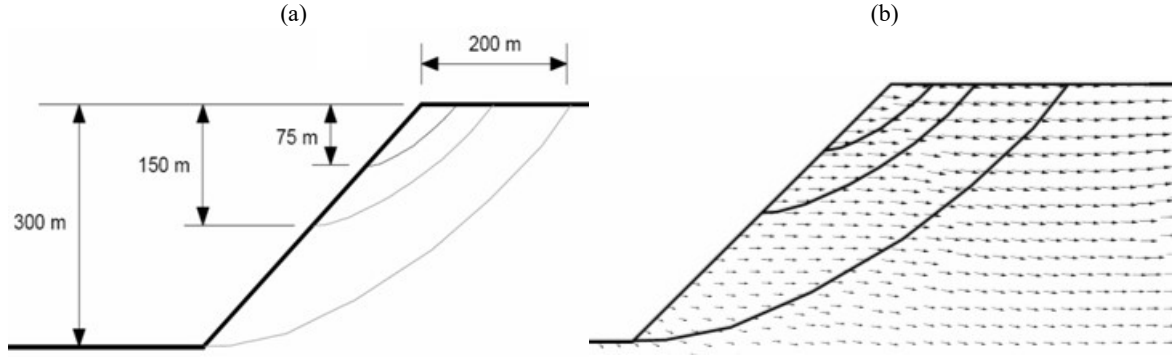


Fig. 22 (a) Potential slip surfaces for which topographical and scale effects were investigated, (b) acceleration magnitude vectors at the instant of maximum crest acceleration (Toh 2010).

In practice, in addition to pseudo-static approach (Seed 1979), numerical dynamic calculations are employed for slope stability evaluation of open pits by using different softwares such as Plaxis, ANSYS, Slope/W, UDEC, 3DEC, and Newmark's deformational method (Newmark 1965).The pseudo-static approach is the most commonly used method for open-pit slopes and provides the factor of safety (FOS) as an indication of whether a slope may fail during an earthquake. But it involves some limitations and assumptions, which are not discussed in this paper. For example, in this approach, the applied horizontal force (as a proportion of gravitational force) is assumed to be constant throughout entire slip mass, both with depth and laterally. Toh (2010) mentions that this assumption could not be considered appropriate for ovearall slopes of open pits, particularly deep/large open pit slopes, and maximum average accelaration that is experienced by a mine slope will be different from the PGA. Although more inputs, such as earthquake ground motion characteristics and dynamic material properties, are necessary for dynamic analysis, they can overcome the limitations of the limit equilibrium analysis for deep/large hard rock slopes. However, Toh et al. (2013) emphasize that many inputs used for dynamic analysis are not well understood for deep hard rock slopes. It is well known that amplification caused by earthquakes at the cliff sides due to topographical effects and lateral movements play an important role on damages to mining facilities and other structures located very close to the slope crest, particularly on those of the constructed on soil like materials and soft and weak rocks. Aydan (2015) recommends that probably, slope angle reduction would be the most desirable measure, and the fundamental concept should be such that the overall stability of the slope must be attained by the self-resistance of ground (Fig. 23a). When there are some occasions, that is, it is impossible to reduce slope angle due to slope-cut height and the construction next to slopes is allowed, there should be a safety zone between the slope crest and allowable construction boundary as illustrated in Fig. 23b.

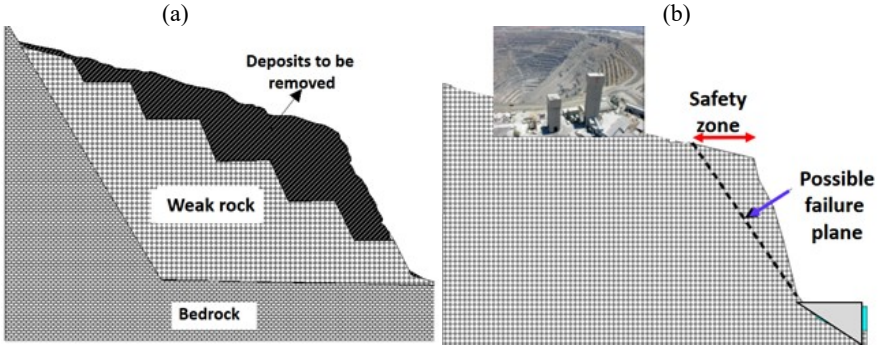


Fig. 23 (a) Slope angle reduction, (b) implementation of safety zone concept for the minşig facilities close to the slope crest (slightly modified from Aydan 2015)

There is limited available information and details of mine damages or slope failures during earthquakes. Mining operations in a few countries located at highly active seismic regions, such as Chile, Peru and Mexico, where deep/large open pit mines also exist, open pit mines are required to be designed with appropriate earthquake loads that depend on their site specific seismic hazard assessments. The necessity of accounting for seismic loading in slope stability analysis, particularly for those of deep/large open pit mines located in earthquake prone regions, increases. In other words, it would be beneficial to place greater emphasis on considering earthquake effects and the use of relevant analysis methods in the design of rock engineering structures, including deep/large open-pit mines.

6 Importance of monitoring for deep open pit slopes

Slope stability analyses and stabilization techniques are highly sensitive to data obtained from laboratory tests, the geometry of the sliding mass and failure surface, and time-dependent changes in the mechanical properties of both the discontinuities and the rock masses. Moreover, determining the ultimate stability of very deep open-pit slopes, especially during the design stage, is quite difficult using the currently available investigation data and analytical techniques. To eliminate or at least minimize the uncertainties arising from these factors, conducting intensive monitoring programs for all open pits has become an essential for management of safety and production consequences of slope failures. With increasing depths, the problems associated with rock deformation, their intensity and frequency also increased and slope failures became as one of the major concern for planning deep open pits. With the advent of many deep opencast mines, slope monitoring systems has become a vital component in planning deeper mines and to aid in mitigating hazards due to pit slope failures.

The main objectives of slope movement monitoring are:(a) ensuring safe operational conditions for personnel and equipment, (b) identifying potentially unstable zones and minimizing slope displacements, (c) determining the mechanisms of developing instabilities and providing data for remedial measures, and(d) assessing the performance of the implemented slope design. Considering that open-pit mines are becoming progressively deeper, it is now common practice to employ *multiple monitoring methods* simultaneously. The movement monitoring techniques used in open-pit slopes can be grouped into three categories: (a) *conventional method* (visual inspection-used as a means of monitoring slopes which are characterized by lower level of danger), (b) *real-time surface-based methods* which detects visible signs of instability and quantifies surface level deformations and displacements, and (c) *subsurface methods*which measures deep within the slope forming materials of the pit. Conventional techniques include visual inspection and mapping of tension crack along the slope face. Mapping and monitoring of tension cracks formed along the slopes is a traditional and a reliable tool for identification of major failures.

6.1 Surface monitoring methods

Because open-pit mining involves a dynamic excavation process and continuous slope advancement, *real-time surface-based monitoring methods*, which are also called *Advanced Monitoring Techniques*, are generally preferred. Among these, the two most commonly used techniques are slope stability radar and robotic total station. The *slope stability radar* (SSR) allows real-time monitoring of slope movements with sub-millimetre accuracy and is independent of atmospheric conditions. It continuously scans the slope face to detect surface deformations (Fig. 24a)from a stationary platform without the need for reflectors or prisms. The system scans a region of the slope and divides the area of interest into pixels. The amount of movement is measured for each pixel and compared with the amount of movement from the previous scan. When the alarm thresholds defined by the user are exceeded, the radar automatically sends e-mail or text alerts to responsible personnel. However, since the system cannot provide three-dimensional data, it does not give information on the direction or geometry of the movement and is therefore often used in combination with 3D geodetic monitoring systems, such as robotic total station.Fukuzono (1985) found that the time accelerating creep failure under gravity loading was inversely proportional to the deformation velocity. Time-of-failure (TOF) can be estimated from the inverse velocity versus time plot by extrapolating the trend to the x-axis. Rose and Hungr (2007) successfully applied this method for predicting time of slope failure in three hard-rock open pit mines. Fig. 24b shows an example about inverse velocity graph. SSR may also be for checking the performance of remedial measures against slope movement. As seen from the example given in Fig. 24c, horizontal drains were installed on the slope monitored by SSR. The upper radar imageshows slope movement (red area) due to water pressure in the slope, while the lower image represents the situation after the drains were installed indicating that movement stopped. Fig. 24d shows a typical SSR image.

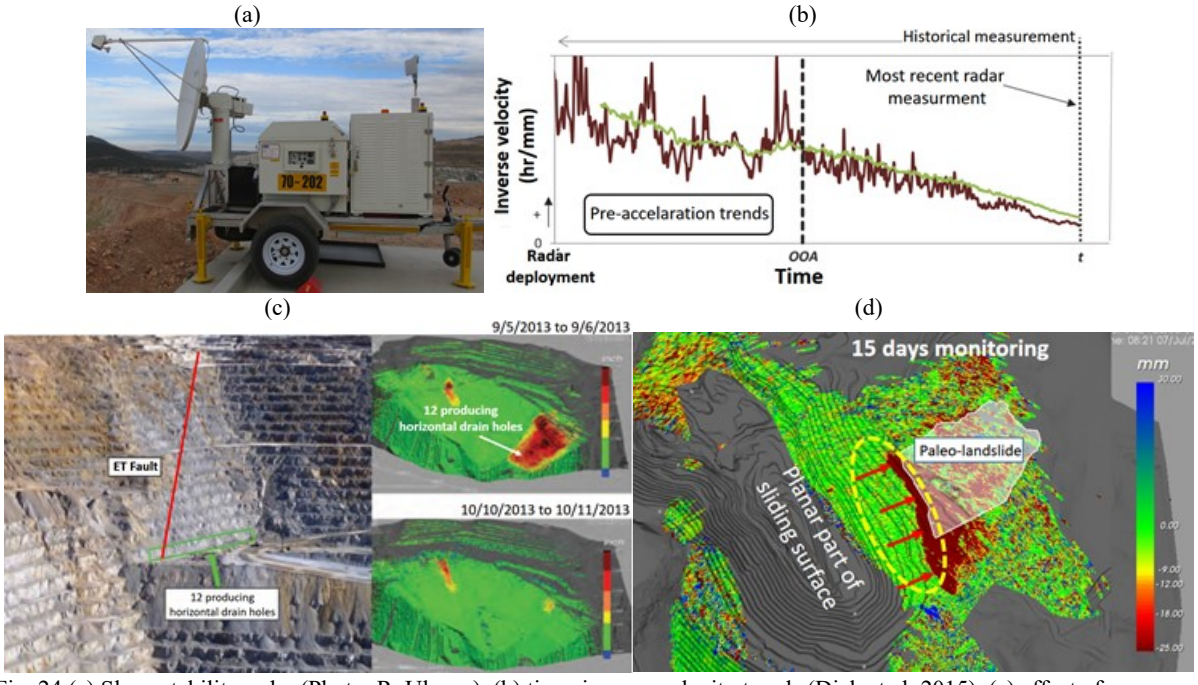


Fig. 24 (a) Slope stability radar (Photo: R. Ulusay), (b) time–inverse-velocity trends (Dick et al. 2015). (c) effect of horizontal drain holes monitored by SSR at an open pit mine (Beale 2018), (d) an SSR image from a coal open pit in Türkiye (Ulusay and Çağlan 2022)

The *robotic total station* (RTS) (Fig. 25a) measures displacements at prisms installed on various parts of the pit slopes, allowing the calculation of three-dimensional displacement, velocity, and direction of movement (Fig. 25b). Warning levels set in the system are transmitted to the mine engineers, enabling real-time assessment of slope conditions and providing an early warning function. RTS provided with GNSS receivers are increasingly being used for automated, accurate, efficient and cost effective survey for monitoring slope in deep and large open pits (Brown et al. 2007). The total station can be fixed at point for continuous monitoring of the slope at the preset interval timings or it can be shifted from one point to other point manually for monitoring the fixed prisms movement.

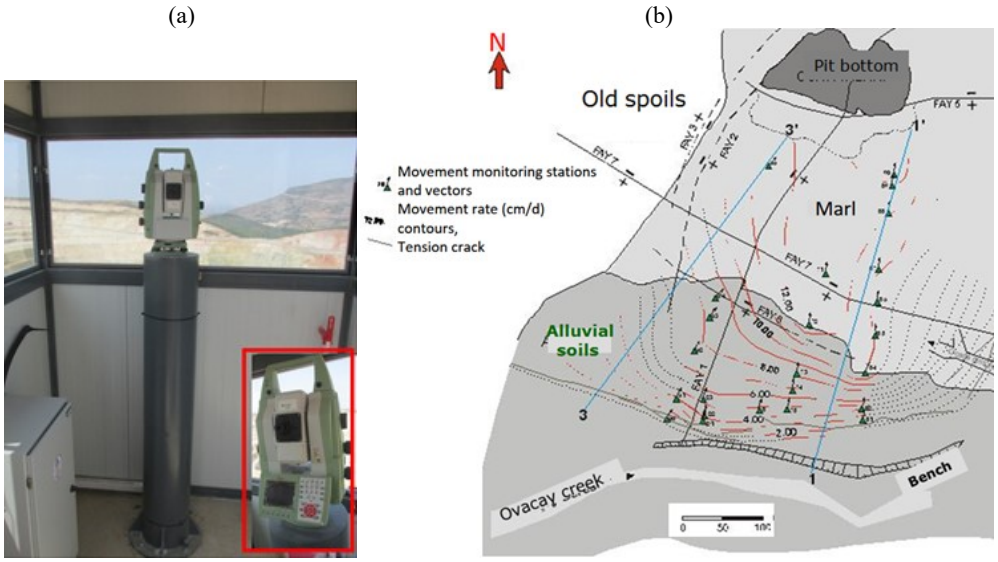


Fig. 25 (a) Robotic total station (RTS) (Photo: R. Ulusay), (b) a pit plan showing movement vectors (black arrows) based on monitoring data from TS, movement rate and tenison cracks at a lignte open pit in Türkiye (Tuncay and Ulusay 2001)

Non - Reflective LiDAR (**Li**ght **D**etection **A**nd **R**anging) *Scanning* utilizes a beam of laser lights, targeted towards the area of monitoring which returns the pictorial / digital representation of the critical area of the slope and their relative movements using the travel time of the reflected radiations (Abellan et al. 2014). They provide virtual copies of slope in minutes, like photographic images highlighting critical areas. Laser scanners are active self-contained measurement technology that generates its own light for measurement process. Though, the application of laser scanners is well proved in monitoring landslides, the method has found lesser application in monitoring open pit slopes. The application of LiDAR on mine surveying is simple, high economic benefit with rapid time saving thus providing successful slope monitoring, but restricted to weather conditions and terrain humidity, and recently automated LiDAR systems with precision as high as 1-5 mm have become available (Eberhardt et al. 2020).

Satellite radar, also known as *InSAR* (**In**terferometric **S**ynthetic **A**perture **R**adar), is a technology widely used in mining for geotechnical monitoring, applied to open-pits, waste dumps, dams, industrial slopes, and underground mines (subsidence). This method detects slow deformations, covering over a large area. With this, InSAR enables strategic monitoring, with the detection of deformationprocesses in their premature state. Datasets provide 1-2 mm resolution coverage of ground movement for most of the earth and the method has large-area application and no ground instruments or site work is needed and can detect movements where risk was not previously suspected. As the limitations of this method; its measurement frequency is limited by satellite passing from 2 to 12 days and it serves as a supplement to, rather than a replacement for, local monitoring methods such as prisms and SSR.

Digital photogrammetry is a simple remote sensing technique to monitor slope movements. Its objective is to identify wide displacement behavior of a target inferred by the comparison of photographed images using digital camera. Using such digital photographs, it is also possible to estimate the geometric properties of the objects such as shape and position, the progress, extent and cause of slope deformation (Ohnishi et al. 2006). Recent developments in the method have provisions for the generation of 3D models from the terrestrial photographs and the method is extensively applied for monitoring large scale structures with a greater accuracy. The method provides to survey the inaccessible areas and highrock faces in rapid time providing remote characterization tool for rock slope practitioners.

As a remote sensing method, *Infrared Thermography* (IRT), is used to detect areas of instability in open-pit slopes by analyzingthermal radiation patterns.IRT measures the infrared radiation emitted by surfaces, allowing for the generationof heat maps that visualize temperature variations which often manifest as negative temperature anomalies, can serve as indicators ofprecursory slope failure. One of the main limitations of this method is signal attenuation due to atmospheric conditions, which can causesignal degradation over long measurement ranges (Le Roux et al. 2025).

6.2 Sub-surface monitoring methods

Due to their ability to measure localized angular deformation and detect subsurface displacements, *borehole inclinometers* are widely used for slope stability monitoring to collect information on the locations of deep-seated failure surfaces. This equipment uses sensors and probes to detect differential movements in real time with sub-millimeter accuracy. In open pit slope monitoring, the probe inclinometer is the most commonly used type of inclinometer in which the probe travels along guide grooves in a plastic tube grouted to the drill hole. As a more recently developed monitoring device, *Time Domain Reflectometer* (TDR) is also used for the same purpose of inclinometer and detects the location of shearing across a grouted borehole, but not the magnitude of deformation. TDR uses the working principle of transmission lines to detect localized strain and deformation by measuring the reflection of electromagnetic pulses sent along a coaxial or fiber optic cable embedded within a borehole in a slope (Le Roux et al. 2025). The method provides high resolution and real time slope stability monitoring. TDR cables are gaining popularity due to its advantages over inclinometers, such as less installation costs, deeper hole depths, rapidly and remote monitoring and possibility of immediate deformation and complex installation.

Evident indicators of slope failure, such as tension cracks, can be monitored using *wireline extensometers*, which measure the dilation of cracks by detecting changes in displacement of a wire anchored to the unstable part of ground. Changes in the length of the wire are measured manually, however, development of modern sensors, such as vibrating wire displacement transducers, has digitized this process, allowing for remote and continuous slope monitoring (Osasan and Afeni 2010).

The groundwater monitoring for open pits is conducted using boreholes and piezometers which measure the total groundwater head at any given location. Types of the piezometers are standpipe piezometers, horizontal piezometers andvibrating wire piezometers (VWP) (single or multi-level). Most commonly used piezometers in open pit mines are standpipe piezometer and VWP. Widespread adoption and standard use of VWPs for monitoring pore pressure in open pit mine slopes and deep groundwater management gained momentum in the 1990s and accelerated rapidly in the early 2000s.

In modern deep and ultra-deep open pit operations, *integrated use of different monitoring systems*has proven effective in reducing risk levels and numbers of fatalities. Two typical examples on this issueare given from the Bingham Canyon Copper Mine (Utah, USA), which has recently reached a depth of approximately 1.4 km. The first major failure, called Manefay Slide that occurred on April 10, 2013 (see Fig. 8), was identified by a multi-sensor monitoring (ground-based system) network consisting of GPS, SSR, RTS, InSAR, LiDAR and wirelineextensometers without no injuries or fatalities. Only some equipment damage was reported, as the run-out distance of the sliding material had been slightly underestimated (Williams et al. 2021). The consensus is that its failure mechanism is a complex failure being along the faulted and clay-filled Manefay beds(Ross 2017). Similarly, another failure, called Leo Failure, occurred onMay 312021 and affected 457 m high southeast portion of the same open pit. This failure, movement was also detected by multiple slope monitoring methods including SSR, RTS, GPS, thermal camera and InSAR, and there were again no injuries or losseswith the aid of slope monitoring studies (Ergun et al. 2023).

7 Conclusions

Evaluation of rock slope stability requires the combined use of a geotechnical model, which includes rock type, discontinuities, weathering characteristics, hydrogeological conditions, material behaviour, and rock engineering methods**.**

At present, knowledge of instability mechanisms in deep and large rock slopes excavated in hard, brittle, and jointed rock masses remains limited. As open pits continue to deepen and complex instability mechanisms involving combined failure surfaces become more common, the current trend is to combine conventional and numerical analysis methods with advanced monitoring techniques**,** while also making use of back-analysis results from pit slopes that have already experienced

instability. In particular, compiling the strength parameters of rock masses derived from completed and future back-analyses of deep and large-scale slope failures into a database would be valuable for assessing deepening slopes that have not yet failed.

Rock masses are rarely homogeneous and isotropic and modes of failure are not always circular or semi-circular. Therefore, all potential modes of failure should be identified to be able to estimate the most suitable rock mass strength parameters and to pursue the most appropriate method of stability analysis.In open pit slope stability investigations, particularly for deepening large open pits, prime attentions should be paid to the evaluation of slope damage.

The integration of different slope stability analysis techniques could represent a major advancement. Within this framework, the development of more powerful numerical programs capable of realistically modellingcombined failures along discontinuities and through intact rocks. Further research is also needed to improve the characterisation,modelling and strength estimation of bimrocksin slopes.

In terms of earthquakes, attention to seismic-induced rock slope instabilities remains limited. Therefore, it would be beneficial to focus more on earthquake inducedslope failures and to incorporate dynamic effects into slope stability analyses.Attention should also be paid the amplification effect due to earthquakes on mine facilities located very close to the crest of the open pit slopes.

Particularly in modern deep and ultra-deep open pit operations, it should be remembered that integrated use of different monitoringsystemsis more effective in reducing risk levels and numbers of fatalities. In addition, Artificial Intelligence (AI) and machine learning can analyse large datasets, identifying early warning signs of instability.

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