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Challenges in Multi-Scale Hard Rock Behavior Evaluation at Deep Underground Excavations

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ABSTRACT

As a consequence of rapid growing trend of resource extraction in world, depth of excavations for resource exploitation increases. Eventually excavations faces with transition from low stress to high stress condition. In this paper, comprehensive aspects on rock behaviour at deep underground excavation were investigated. The state of art of rock behaviour at micro- meso- and macro-scale were discussed and relevant challenges along with achieved knowledge, experiences, and research results were presented.

At micro-scale, research results revealed that, apart from chemical bonding, rock behaviour significantly influenced by deficiencies such as; particle-crystal boundaries, heterogeneity, pores and micro-cracks, which reduces the rock strength 2-3 order of magnitude. Granite SEM images proves the deficiencies between crystals, micro-cracks and pores at each crystal, and weakness and foliation of mica components. When stresses applied on specimen, new tensile cracks nucleated and initiated from the edge of existing micro-cracks, and rate of crack propagation depends on the differential stress level. At meso-scale, true triaxial testing makes it possible to apply different stress paths in the ranges of ground in situ stresses, concentrated stresses and even dynamic loads. Careful assessment of the full stress–strain curves of the true triaxial test results of granite and conventional triaxial test results of Marble shows that rock mechanical properties such as magnitude of linear elasticity, ductility domain, peak strength value, ranges of brittleness, and residual strength level significantly differs with changing confining stresses. The rock stress – strain behaviour variation were categorised to four distinct stages consisting; 1) Elastic-stable micro-cracking, 2) Stable - unstable micro-cracking, 3) Unstable micro-cracking-brittle failure, and 4) Brittle failure-residual strength. The ranges of rock behaviour at each stage with different confining stresses were illustrated, which could be used as input for mechanical parameters in design analysis. At macro-scale, counteraction between 'Rock Mass Composition (RMC)', 'Active Stress Condition (ASC)', and 'Excavation Method, Size and Orientation (EMSO)' to estimate the 'Rock Mass Behaviour (RMB)' were discussed and presented as a verbal equation. To reduce the sudden failure risk, a micro-seismic monitoring system were designed and implemented for perdition and warning of failure and evacuation in timely manner. To verify the presented approaches, rock mass behaviour and failure mechanisms were illustrated in a deep gold mine in Western Australia.

To manage the ground behaviour; considering the static and dynamic loading and interlocked nature of rock masses at deep underground excavations, the ratio of "Ground energy demand" to "support energy absorption capacity" is mostly used for stability evaluation. Finally, it should be noted that, the geomechanics at general and deep underground geomechanics specifically is a developing field due to incapability to achieve proper ground characteristics, huge number of variables and their coupled interactions, and incompetence in analysis them properly. Therefore, the results from current analysis should not be taken as granted and always solid engineering judgement must involve in interpretation and design. It is also hoped that future development in sophisticated ground exploration technologies along with advances in computation science will assist geomechanics engineers to mature their knowledge of rock mass behaviour and safe and economic design in engineering activities.

KEYWORDS: DEEP MINING, ROCK BEHAVIOUR, UNSTABLE FAILURE, SUDDEN FAILURE, ROCK BURST, SEISMIC MONITORING

1- INTRODUCTION

Earth crust is a main source of humane needs such as water, mineral, oil and gas, and hot waters (geothermal energy). Additionally, many civil infrastructures consisting hydrocarbon, goods, waste storage are located in earth crust. The earth resource engineering activities in earth crust illustrated in Figure 1.

Review of global trends shows that (Figure 2) Industrial revolution and rapid world population increases after Second World War (WWII) (Figure 2-a) lead to excess of the energy resources consumption (Figure 2-b). Similar significant increases were occurred in earth resource extraction in which mineral extraction is shown in Figure 2-c. As consequence of substantial removal of near surface earth resource depth of exploration continuously increasing as shown in Figure 2-d. Underground excavations to reach unprecedented depths in response to global demands for mineral and energy resources as near surface resources are depleted. These trends are crucial and need to be addressed properly for sustainability of the earth's resource and environment.

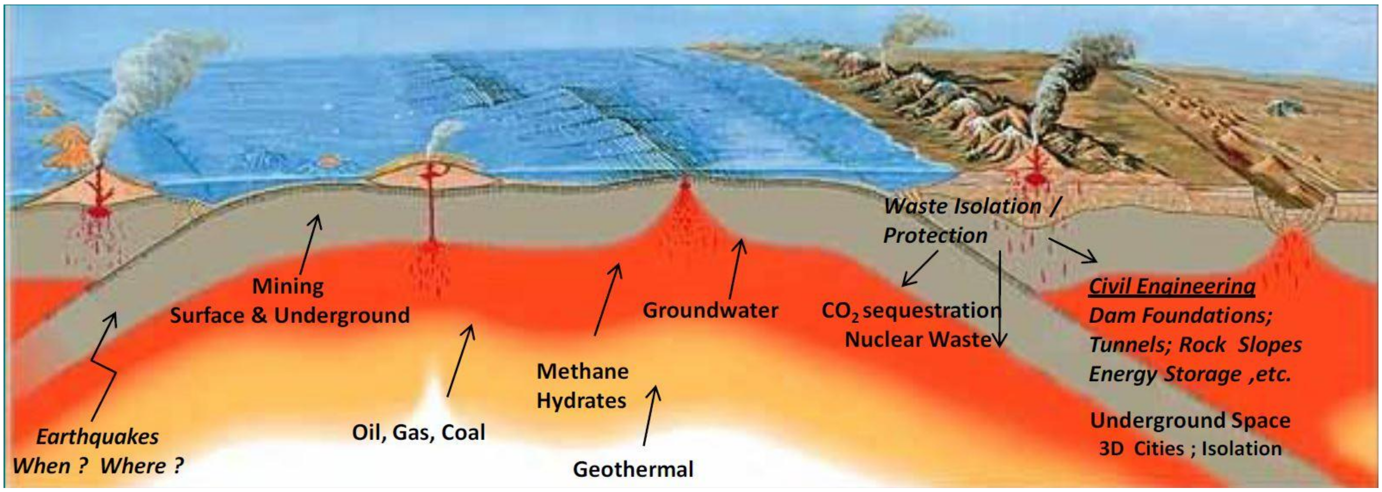


Figure 1. Engineering activities in earth crust (Modified after NEU, 2010).

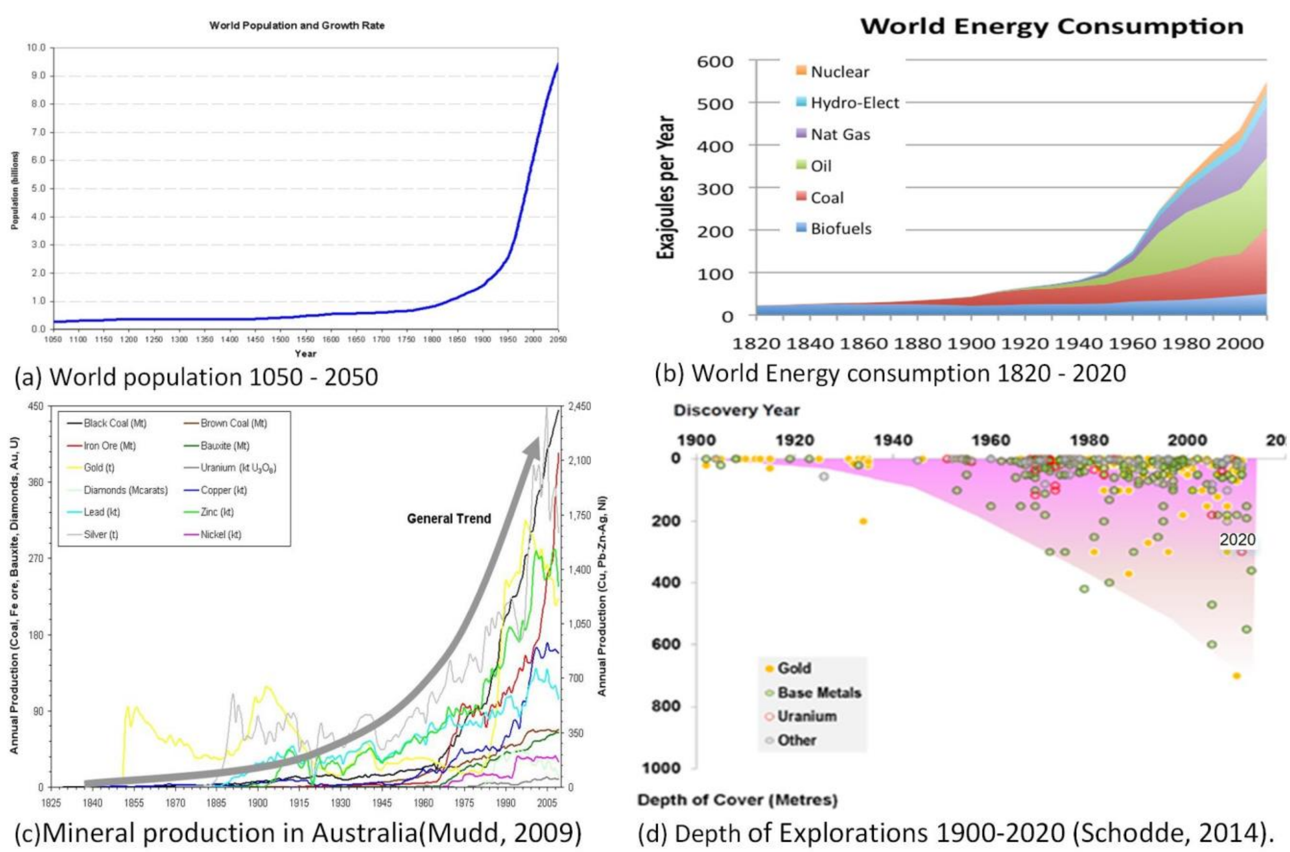


Figure 2. Global trends in (a) population, (b) energy consumption, (c) Mineral production, and (d) Exploration depth.

From the geomechanical view point, with increases in depth of excavations for resource recovery, the ground stress, and temperature will increase accordingly. Ground temperature increases with the rate of 25 centigrade per kilometre which have minor influence on rock behaviour, however with going deeper (about 40 km) it could significantly change the rock behaviour. The ground vertical stress increases with the rate of 27 MPa per kilometre and horizontal stresses varies in the range of 0.5 to 3 times of vertical stress depending on geological structures and tectonic.

Hard rock mining is experienced at a depth of about 2 km in Australia, more than 3 km in Canada, and a depth of about 4 km in South Africa highlight the importance of ground behaviour at such depths. For example, the Mponeng, TauTona and Savuka Gold Mines southwest of Johannesburg in South Africa are about 4000 m depth (Lippmann-Pipke et al., 2011). The Kidd Creek and Creighton Mines in northern Ontario, Canada are about 3000 m depth (Counter, 2014). The deepest hard rock mines in Australia are the copper and zinc lead mines in Mount Isa, Queensland at 1,800 m depth. In China, there are more than 30 metal mines in production or to be constructed with mining depths over 800 m (Dong, 2016). Similarly, transportation and hydroelectric tunnels are reaching unprecedented depths as infrastructure development increasingly expands into mountainous regions. The Olmos Trans-Andean Tunnel in Peru experienced overburdens of up to 1930 m. The Gotthard Base Tunnel in Switzerland was constructed with overburdens of up to 2300 m. The tunnels of Jinping II reached maximum overburdens of 2525 m.

Prior to any engineering activities, rocks at deep underground are under high stresses from all sides therefore basically they show high strength due to high confinement, which is far different from shallow condition. Initial consequence of deep underground excavation is removing the one or two of the confining stresses which leads to remarkable rock strength reduction (Wawersik & Brace, 1971). The second consequence deep underground excavation is stress disturbance due to stress redistribution, groundwater pressure change, and possibly seismic loading where the resultant of all those applies to rocks around underground opening. Considering the superposition of multiple loads, the behaviour of rocks around an opening largely influence the rock structure from micro- to macro- scale, depends on brittle-ductile nature, loading-unloading rate which could be significantly different from shallow excavations failure modes. In deep underground excavations special failure modes such as; spalling, slabbing, rock burst, splitting, buckling, bulking, and squeezing or combination of them occurs which is fundamentally different block fall or sliding at shallow openings. One of the main differences of failure at deep and shallow openings is duration, where mostly sudden ejection with huge energy release is in deep underground and gradual failure with low energy release in shallow openings. Many researchers such as; Ortlepp and Stacey, 1994; Kaiser and Cai, 2012; He et al. 2010; Eberhardt et al., 2017; Feng et al., 2016; Diederich, 2007.

In this paper major challenges and inducing factors in deep underground excavation and attempts for understanding the rock behaviour mechanisms through multiscale rock structure study, laboratory testing, field evidences along with worldwide examples are presented.

2- ROCK BEHAVIOUR AT MICRO-SCALE

Rocks as a solid material composed of minerals or crystals in which in smaller scale made from atoms, molecules and lattices. The particles kept together with chemical bonds, where several types of bonds such as metallic, ionic, molecular and covalent network. In essence the bonds are the matter of energy between microstructures which governs the rocks mechanical properties specifically strength, brittleness or ductility. All rocks suffers from the wide range micro-structural defect such as;

- Atomic disorder and dislocations in pure and homogenous rocks,
- Crystal lattice boundaries in crystalline and foliated rocks due to two dimensional covalent network (Figure 3),
- Heterogeneity (adjacency of weak and strong rock particles),
- Pore spaces during generation mainly due to gas escape in volcanic rocks,
- Cleavages due to overtime deformation and residual stresses,
- Micro-crack or structural defects due to stresses.

These micro structural defects are main reason behind crack initiation, propagation and failure in larger scale. The type of bonds between particles and their spatial patterns such as one, two or three dimensional bonds defines the massiveness, foliation and/or one dimensional growth of rock particles and crystals as shown in Figure 3 and Figure 4. For example, one dimensional or bonds in quartz (needle type figure 4.a), two dimensional bonds in graphite (plate or layered type in Figure 3 and Figure 4-c), and three dimensional bond in diamond and silica as shown in figure 4a-b. The rocks with Ionic and covalent network bond mostly shows brittle behaviour. Depend on the bond strength at each direction the rock mechanical heterogeneity could be determined. Figure 3 shows a three dimensional bond where, bonds at each layer are strong covalent network, while the bond between layers is weak Van Der Waals bonds. In other words, however the bonds in plane (2D) are solid, but the bonds in third direction (layers bond) is feeble. This type of bonds are common in many metamorphic foliated, volcanic and clastic sedimentary rocks. When stresses applied to these type of rocks, depends on stress orientation they behaves as anisotropic sliding, shearing, buckling and bulking as shown in Figure 4-c.

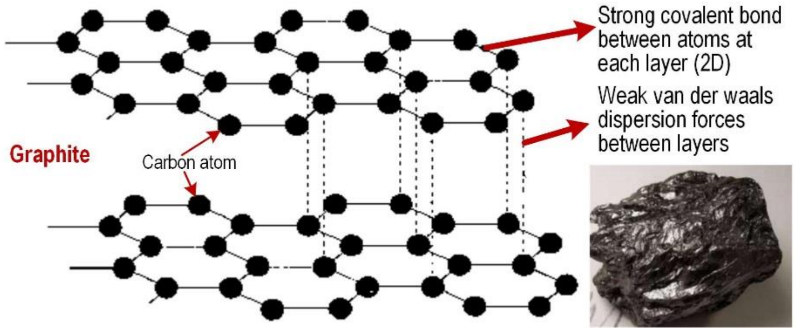


Figure 3. Two dimensional covalent network bonding structure with weak Van Der Waals bonds between layers in foliated rocks (example of Graphite).

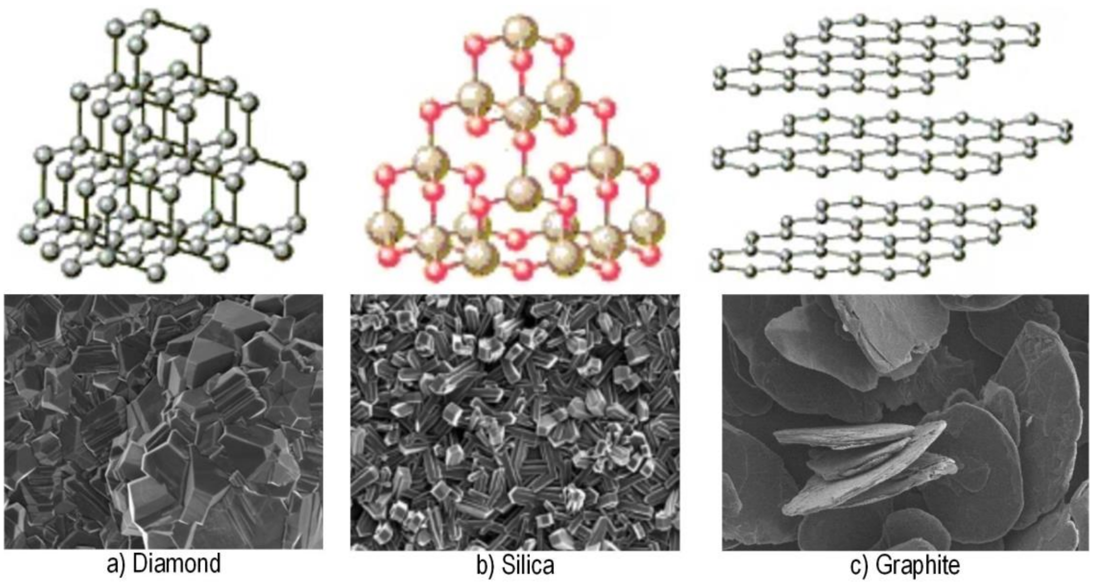


Figure 4. Atomic bonding structure and SEM image of three typical rocks; a) Three-dimensional covalent network homogenous bonding structure in Diamond, b) Three-dimensional covalent network heterogeneous bonding structure in Silica, and c) Two dimensional covalent network bonding structure with weak Van Der Waals bonds between layers in Graphite (not in scale).

Figure 4-a shows a three-dimensional covalent network bonding between carbons in diamond where the bonds are strong in all directions which makes rock isotopically resistant. However homogenous rocks are rare in the engineering activities, but strong homogenous rock failure appears as bond breakage and chipping or spalling and rock burst, and softer homogenous rock failure appears as squeezing due to stress concentration. Like as figure 4-a, figure 4-b shows a three-dimensional covalent network bonding structure between silicium and oxygen in Silica which makes it heterogeneous and consequently anisotropic behaviour against the load. Depends on the stress state and degree of heterogeneity, various types of failure such as splitting, buckling, spalling, squeezing could occur.

At microscale, rocks not only suffer from weak chemical bonding between particles, but also they suffer from voids, micro-cracks and cleavage, which dictates the fracturing orientation and failure mode. To investigate various types of micro-scale rock defects in the granite specimen was studied using Scanning Electronic Microscopy (SEM) as shown in Figure 5. Ranges of defects were captured by zooming on specific features, for instance the micro-cracks between crystals in overall structure, micro-cracks at each mineral with zooming on specific minerals, pores, and cleavages. The measured deficiencies were studied and their patterns at each mineral such as quartz, mica, feldspar and albeit were recognised. Considering the scale of the images the length of defects could be measured easily.

Figure 5-a images with 200 times magnification shows contact between quartz, feldspar, mica and amphibolite crystals in granite. Crystal bonds together over long geological times with interlocking and mineral liquids which is far weaker than the in-crystal bonds. Therefore crystal contact areas could be considered as defect in crystalline and even granular rocks. Figure 5-b images with 500 to 1000 times magnification shows deficiencies at each crystal surfaces such as micro-cracks, pores and cleavages. Figure 5-c and Figure 5-d images with 1000 times magnification shows deficiencies on quartz and feldspar crystals, where small scale foliation is evident on feldspars (Figure 5-d).

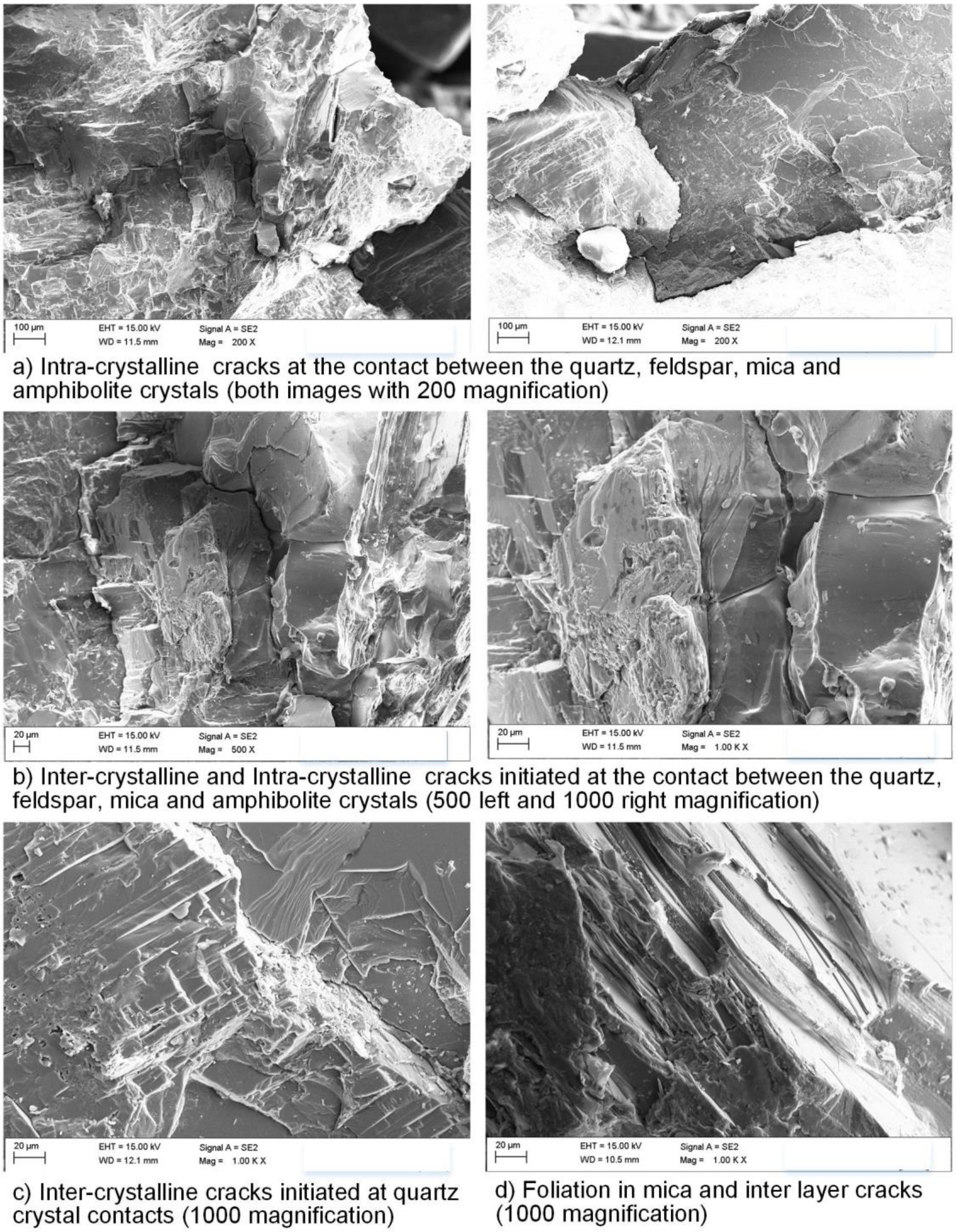


Figure 5. SEM images showing micro-structural defects in granite specimen with 200 to 1000 times magnification consisting; a) Intra-crystalline cracks at the contact between the quartz, feldspar, mica and amphibolite crystals, b) Inter-crystalline cracks in the quartz, feldspar and amphibolite, c) Cracks in and between quartz crystals, and d) Foliation in mica and inter layer cracks.

When a rock specimen undergoes loading, stresses concentrate around micro-scale deficiencies and when stresses exceeds strength of materials around deficiencies, new micro-cracks nucleated or existing micro-cracks propagates. Existing micro-cracks could be attributed to different sources as partially unveiled using SEM results as shown in figure 5 for granite specimen. He 2010 stated that; the feldspar in granite is the feeblest particle and the bonding between quartz grains is also such that it makes the specimen weaker. This declares that the stored energy during and rock sudden failure is larger than that in a uniaxial test. It is also found that, theoretical strength of rocks are between one-third and one –tenth of elastic modulus (E), while presence of deficiencies cause a reduction of rock strength to one-hundredth to one-thousandth of the Young's modulus (McClintock & Argon, 1966, Pellet & Selvadurai, 2017). Increases of loading on specimen leads micro-crack propagation, coalescing, creating fracture in whole specimen and overall failure. The rock damage initiation and development could be controlled up to specific stages of loading and deformation so called stable crack propagation, however with exceeding loading and energy level the crack development becomes uncontrollable and it leads to unstable failure (Eberhardt et al., 2017). The criteria for crack initiation and propagation were modelled by utilising energy approaches and fracture mechanics methods. The Griffith criteria for onset crack propagation with simplifying the original criterion is presented as follow:

$$\begin{aligned} &(\sigma_1 - \sigma_3)^2 - 8 \cdot \sigma_t \cdot (\sigma_1 + \sigma_3) = 0 && \text{if } \sigma_1 > 3 \cdot \sigma_3 \\ &\sigma_3 = -\sigma_t && \text{if } \sigma_1 < 3 \cdot \sigma_3 \end{aligned} \quad (1)$$

Where the σ_1, σ_3, are major and minor principle stresses, σ_t, is the tensile strength. Once new micro-cracks crack initiated, the crack development will entirely depend on triaxial stress levels and rock properties. In the loading domain where the tensile crack growth is low and can be controlled so called stable cracking mode, on contrary high rate of shear crack growth will lead to unstable cracking mode and failure. The comparison of the initiation and propagation of cracks wing from a Griffith crack under triaxial stresses were illustrated in figure 6. The figure 6 shows that slight increases in confining stress (10-20%) of the major principal stress limits, the crack length propagation by about 80%. (Eberhardt et al., 2017).

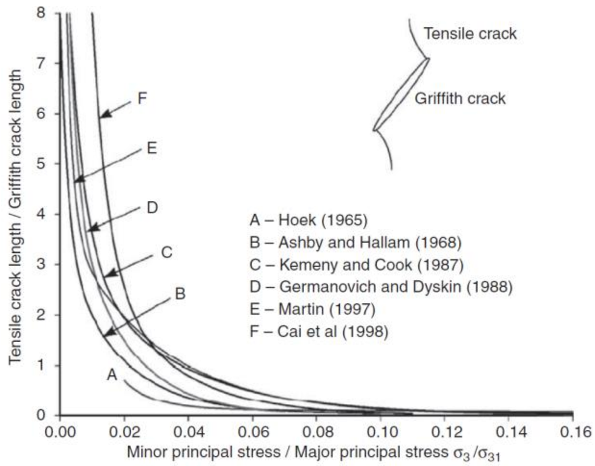


Figure 6. Correlation between stress ratio (σ_3/σ_1) and crack growth ratio (tensile crack length / Griffith crack length) compiled from experimental and numerical studies data (Eberhardt et al., 2017).

3- ROCK BEHAVIOUR IN MESO-SCALE

In the real ground condition, rocks are mainly under three different stresses as σ_v, $\sigma_{H1,}$ and σ_{H2} or three major principle stresses denoted as σ_1, σ_2 and σ_3. To investigate rock behaviour in such three dimensional loading condition, utilisation of true triaxial testing system is inevitable. Due to difficulties in testing so far, most tests were performed in conventional triaxial tests, where two confining stresses are considered equal ($\sigma_2 = \sigma_3$). In this research to apply (similar to ground) three dimensional stresses on the specimen in the laboratory, true triaxial apparatus were introduced as schematically shown in figure 7. Using the servo-control testing machine the post peak specimen behaviour is studied. A true-triaxial testing machine was designed and developed by Feng et al., 2016 to study the hard rock behaviour under both loading and unloading stress paths. The true-triaxial testing system consists of loading frames, confining cell, transducers, a data acquisition system, controllers and hydraulic systems as well as software (Figure 7). The stress path can be programmed using a developed computer code.

The stress path is used to represent the continuous states of stress in a test specimen during loading or unloading in the three-dimensional principal stress space. However the effect of the stress path on the rock strength, ductile deformation and residual strength is remarkable but it is not paid enough attention. Complicated stress paths can be designed in true triaxial tests based on the real stress condition, pre-excavation and sudden or gradual changes of stress during and after excavation. In this research granite specimens were tested under different triaxial stresses. The typical loading path for determining the granite stress–strain behaviour is shown in figure 8. To apply loading path as shown in figure 8, the hydrostatic pressure is uniformly increased by increasing the fluid pressure in the confining cell until a target value of $\sigma_1 = \sigma_2 = \sigma_3$ is achieved. Then, σ_3 is held constant while σ_1 and σ_2 are instantaneously and gradually increased to reach the desired value for σ_2. In the next step, σ_2 and σ_3 are held constant, and loading is applied in the σ_1 direction using either stress control or strain control until failure occurs (Figure 8). In testing granite specimens to determine the stress–strain curves, a combined control mode may be adopted. At approximately 70% of the peak force, which could correspond to a dilation point where the volumetric strain increases from its minimum value, the control mode is switched to deformation control in the minor principal stress direction until a complete force–displacement curve is obtained. In the elastic deformation stage, stress control is used to increase σ_1 at a rate of 0.5 to 1 MPa/s. After the dilation point, the stress control is switched to minor principal strain control; the corresponding strain rates range from approximately 1 to 10×10^{-6} /s. (Feng, 2016, ISRM 2017). Another typical stress paths were also developed by He et al. 2017 to study the sudden failure mechanisms due to rapid unloading of one of principal stresses as shown in figure 9. In this approach load is first increased consistently and slowly to reach the hydrostatic stress state ($\sigma_1 = \sigma_2 = \sigma_3$) marked by letter A (Figure 9), under the convention of $\sigma_1 > \sigma_2 > \sigma_3$ for the principal stresses. Secondly, σ_1 and σ_2 were increased step by step until approaching the stress state marked by B. then, this stress state was maintained for a certain time to allow the equilibrium state to be attained inside the rock specimen, and then σ_3 was suddenly removed from one surface of the specimen (Figure 9a) while keeping σ_1 and σ_2 at constant. Thus the sample is at the possible occurrence state of rockburst (the first unloading marked by C). Typically, the stress paths in Figure 9 have three cycles of the loading/unloading, assuming that rockburst occurs at the third cycle (He, et al., 2017).

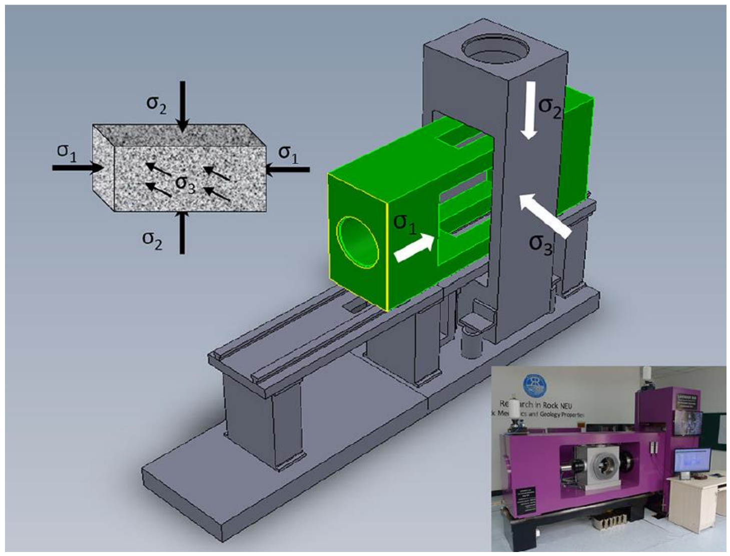


Figure 7. Schematic view of the true triaxial apparatus showing the loading directions on the specimen (Feng et al., 2016).

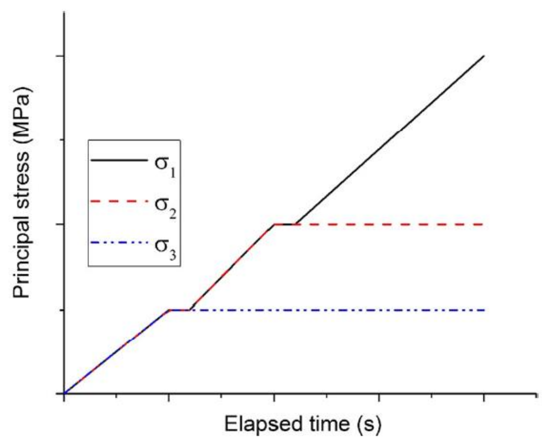


Figure 8. Typical stress paths to study the true triaxial behaviour.

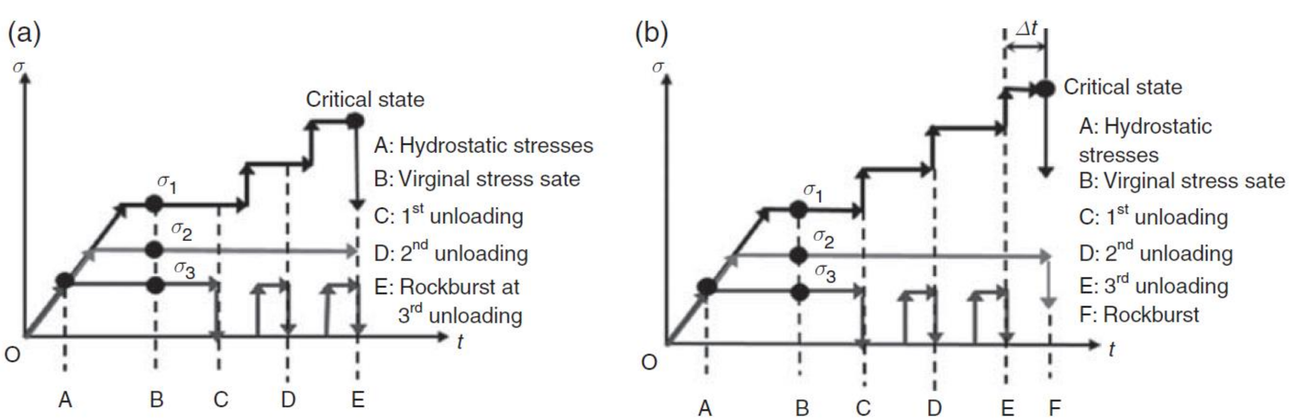


Figure 9. Stress paths; (a) stress path for instantaneous rockburst, and (b) stress path for delayed rockburst (He et al, 2017).

In this research the effect of the minimum principal stress on the stress– strain behaviour of granite specimens was studied. Tests were conducted using the same stress path explained in Figure 8. In other words, σ_2 was kept constant at 50 MPa and σ_3 was varied from 2 to 30 MPa, which corresponded to a gradual change from a conventional triaxial compression test ($\sigma_1 > \sigma_2 = \sigma_3$) to a triaxial extension test ($\sigma_1 = \sigma_2 > \sigma_3$). The specimen deformation behaviour considering the three principal strains (ε_1, ε_2 and ε_3) under differential stresses ($\sigma_1 - \sigma_3$) is illustrated in figures 10. The coordinate origin of the strain is defined as the point when the deviator stress is applied; therefore, the uniform compression stage is removed from the graphs. For a constant σ_2 (σ_2=50 MPa) at each σ_3 (σ_3=2, 10, 20, 30 MPa), the σ_1 is increased until a maximum value is reached. The strength under the triaxial extension condition ($\sigma_1 > \sigma_2 > \sigma_3$) is higher than that under the conventional triaxial loading condition ($\sigma_1 > \sigma_2 = \sigma_3$). Compared with the conventional triaxial strength ($\sigma_2 = \sigma_3$), the increases of the maximum peak true triaxial strength and the biaxial strength ($\sigma_1 = \sigma_2$) are 30 and 7 %, respectively. It is found that relatively high peak strength occurs in the same σ_3, but changing σ_2 (Figure 10).

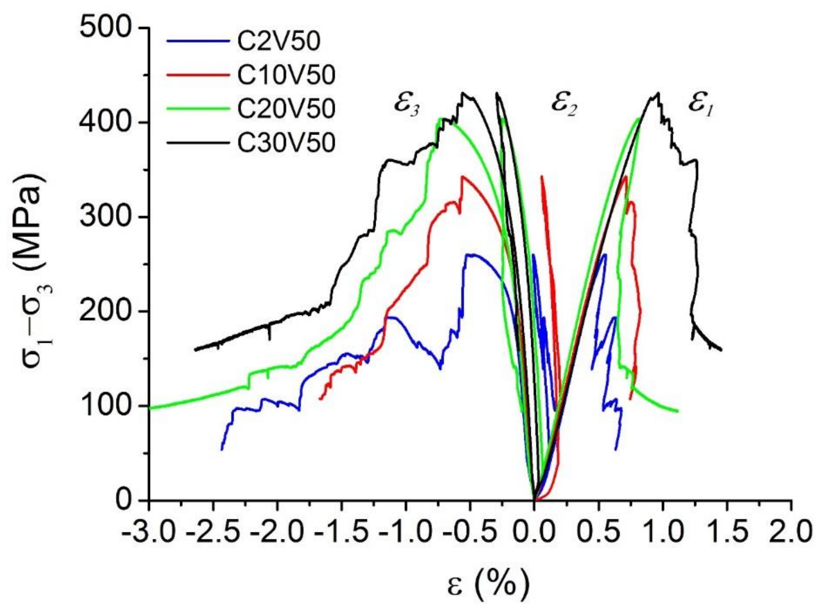


Figure 10. The stress-strain curves obtained from true-triaxial tests under different minimum principle stresses and constant intermediate stress on granite specimens.

The fracturing patterns due to failure varied according to triaxial stresses condition. The number of fractures and fracture orientation strongly depends on magnitude of principal stresses. In uniaxial test, one or few fractures develop along axial stress direction, however in high confining stresses a huge number of conjugate and oblique fractures occurs. The fracturing modes for rock specimen under constant intermediate stress and different minor principle stresses is shown in Figure 11. Under minor principle stresses of 10 MPa, three major tensile fractures (splitting) parallel to major principle stresses can be seen. When σ_3 increased to 20 MPa, specimen failed under shear and slightly oblique fracture. With increasing the σ_3 to 30 MPa two sets of conjugate fractures occurred and specimen divided to multiple pieces. As it can be seen from figure 11 the fracture inclination increases with increasing confining stresses.

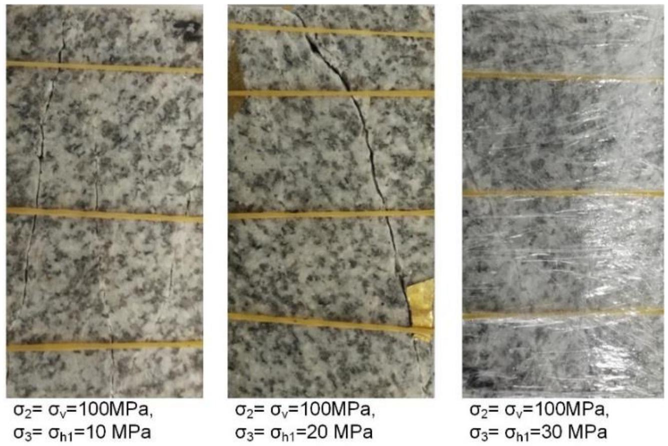


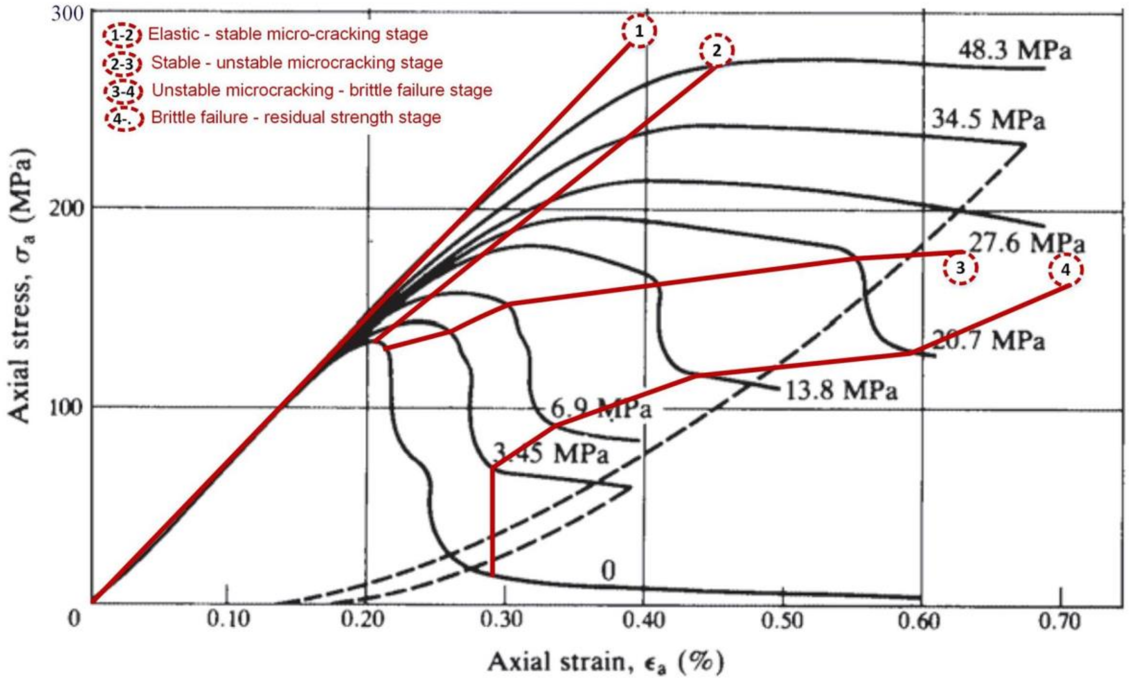
Figure 11. Granite specimen failure modes after true triaxial tests.

At deep underground where stress and confinements are high, rocks shows higher strength than its uniaxial strength. As depicted in figure 12-a at uniaxial testing condition, specimen shows low strength and high brittleness with very low or no residual strength. With gradual increases in confining stresses ductility, peak and residual strength of rocks increases and rock brittleness reduces. Stress-strain curves from conventional and true triaxial tests is shown in figure 12-a, and 12-b respectively. Based on figure 12, whole stress-strain behaviour categorised in four distinct groups, which shows rock behaviour evolution and alteration under different confining stresses and briefly explained below;

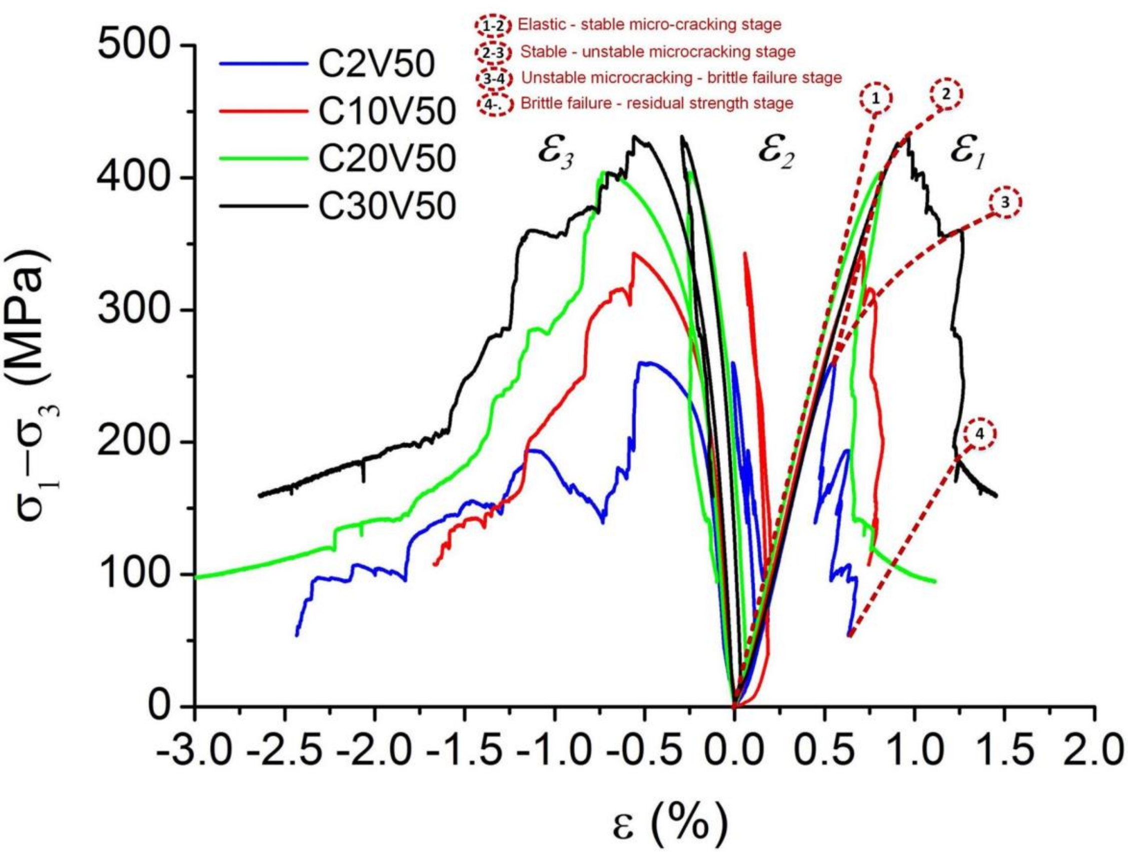
1- Elastic to stable cracking zone (1-2): based on figure 12, area between line 1 (elastic) and line 2 (stable cracking or ductile) is called stable cracking zone. As shown in figure 12, increases in confining stresses results to increase in elastic behaviour or delay in crack initiation. In this zone at first rock behaviour deviates from linear elastic behaviour and existing pores and cracks in rock are activated and possibly minor tensile crack may develop, where cracks will be closed with unloading and rock will recover the pre-test condition. At this stage, rock mechanical properties such as cohesion (C), friction (ϕ) and poisons ratio considered constant.
2- Stable to unstable cracking (2-3): based on figure 12, area between line 2 (stable cracking) and line 3 (un-stable cracking) is called unstable cracking zone. In this zone, new cracks nucleate and develop from existing pores and cracks. The length of crack development is highly dependent on the stress level as shown in Figure 6. In this stage, incremental increase in loading leads to crack propagation and calescence where the loading energy stored in specimen cause un-controllable crack development and building major overall crack in specimen. Obviously rate of crack development and length of un-stable crack development highly depend on confining stress levels. In other words, at low confining stresses stable cracking rapidly ends up with unstable cracking and brittle failure, but with increasing confining stresses length of unstable cracking increases significantly. The mechanical properties of rock reacts differently during unstable cracking, cohesion reduces because of crack opening, while friction increases due to shear cracking. Poisons ratio and Young modulus reduces in this stage.
3- Un-stable to brittle failure (3-4): at this stage the accumulated energy in specimen tends to release it on cracks and de-stabilise cracks, which leads to major brittle failure plane in specimen. Increasing confining stresses have remarkable impact on failure occurrence, brittleness mode (energy releasing – energy absorbing) and magnitude of strength loss. As shown in line 3 in Figure 12, under uniaxial test, brittle failure happens at low deformation and specimen loses almost its strength completely. With increases in confining stresses, firstly rock strength increases and failure occurs in higher stresses, secondly failure starts at higher deformation, thirdly magnitude of strength drop reduces and at higher confining stresses rock failure point disappears.
4- Brittle failure to residual strength (4 onward): at this stage fractured rock moves on each other and only frictional stress, or apparent cohesion is resisting against applied stresses. At low or no confining stresses, residual strength is totally lost or negligible but increases in confining stresses leads to increases in residual strength and at high confining stresses it vanishes.

To conclude triaxial rock behaviour we found that all rock properties such as rock strength, brittleness, ductility, cohesion, friction, poisons ratio and Young modulus are varying depending on stress conditions and they should be considered according the stress path at specific project condition. Also it is found that small increases in confining stresses can results in significant improvement of rock properties such as peak and residual strength, ductility and reduced brittleness. This concept is used in underground excavation by providing support reinforcement to absorb ground energy and limit the possibility of failure.

To model the rock behaviour under triaxial stresses, conventional Mohr-coulomb failure criteria is no longer applicable because it considers cohesion and friction increases with confining stresses. The concept of cohesion loss and friction resistance were proposed and developed over the last three decades and several models were presented as shown in figure 13 (Eberhardt et al., 2017). As shown in Figure 13, four models consisting; a) conventional Strain-Weakening (SW), b) Cohesive-Brittle-Frictional (CBF), c) Cohesion-Weakening and Friction-Strengthening (CWFS) and d) Damage-Initiation and Spalling-Limit (DISL). The Strain – Weakening (SW) model (Figure 13-a) was suggested by Hoek et al. (1995) and considers the instantaneous reductions in rock cohesion and frictional strength with yielding. The Cohesive – Brittle – Frictional (CBF) model (Figure 13-b) was proposed by Martin (1997) due to shortcomings in SW model in estimating the thickness of brittle failure around tunnel. The CBF model, assumes continuous losses in cohesion loss and gaining friction with increasing plastic strain. The Cohesion-Weakening and Friction- Strengthening (CWFS) model (Figure 13-c) was presented by Hajiabdolmajid et al. (2003) and it is similar to CBF model where, both models considers almost similar cohesion loss. The difference between CWFS and CFS is that; CWFS model considers slowly friction strengthening than CFS model, which could attribute to shear micro-cracking. Finally, Damage Initiation and Spalling Limit (DISL) model (Figure 13-d) was presented by Diederichs (2007) which is similar to CBF and CWFS models. Despite to CWFS model in DISL model initial yielding is considered due to micro-cracks initiation and damages.

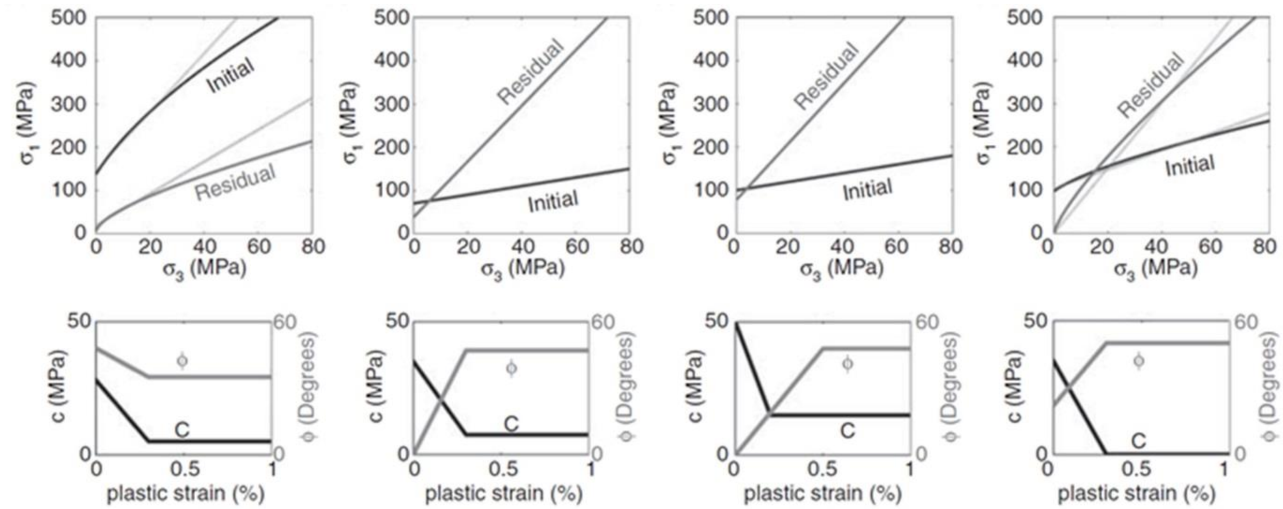


a) conventional triaxial test results on Marble (Wawersik & Fairhurst,1970)



b) True triaxial test results on granite specimens

Figure 12. Intact rock behaviour modes and their postulated transition limits on test results from a) conventional triaxial on marble specimens (Wawersik & Fairhurst, 1970) and b) true triaxial test results on Granite specimens.



a) Strain-Weakening (SW) model by Hoek et al.1995 and Martin1997

b) Cohesive-Brittle-Frictional (CBF) model by Martin (1997)

c) Cohesion-Weakening-Friction-Strengthening (CWFS) model by Hajiabdolmajid et al. (2002),

d) Damage-Initiation and Spalling-Limit (DISL) model by Diederichs (2007).

Figure 13. Variation of cohesion and friction models during yielding used for modelling depth of brittle failure (i.e., spalling) (Eberhardt et al., 2017) : a) conventional Strain-Weakening model used by Hoek et al. (1995), b) Cohesive-Brittle-Frictional model introduced by Martin (1997), c) Cohesion-Weakening and Friction-Strengthening model introduced by Hajiabdolmajid et al. (2002), d) Damage-Initiation and Spalling-Limit model introduced by Diederichs (2007).

4- ROCK MASS BEHAVIOUR (RMB) IN MACRO-SCALE

It is found that rock mass behaviour is scale dependent and according to rock mass physical characteristics, stress condition and engineering project size various behaviours are expected. Based on the existing knowledge and experiences in engineering projects around the world, the Rock Mass Behaviour (RMB) can be expresses by following verbal equation:

$$Rock\ Mass\ Behaviour\ (RMB) = Rock\ Mass\ Composition\ (RMC) + Active\ Stress\ Condition\ (ASC) + Excavation\ Method, Size\ and\ Orientation\ (EMSO) \quad (2)$$

In this verbal equation;

RMC; indicated to rock mass physical conditions such as; rock type, intact rock and discontinuities characteristics and geological structures, faults, folds and weak zones. Analysis of these factors give reliable evaluation to classify rock masses to massive, blocky, heavily jointed, and special minerals.

ASC; indicated to principle stresses resultant of all stresses consisting, in-situ stresses, groundwater pressure, induced stresses, and seismic events sourcing from natural or engineering activities. Vertical in-situ stress estimated from weight of overburden rocks which is high at deep underground excavations, and horizontal to vertical stress ratio varying in wide range from 0.5 to 4. ASC, could also represent the interlocking due to high stress condition at deep underground mines.

EMSO; indicates to underground opening excavation method, size at each sequence and orientation in relation to rock mass condition. On the other works, scale dependent rock mass behaviour (RMB) is greatly influenced by opening size to rock mass size ratio and active stresses as interlocking and yielding factor.

By reasonable integration of the verbal equation components with sound engineering judgement, ground most likely behaviour could be diagnosed. Hence, the main ground types and their probable behaviour types could be classified as follow;

- In intact or massive hard rocks; due to high stress concentration at depth, spalling, slabbing, and rock burst are main failure mechanisms, however in intermediate or soft massive rocks, squeezing is more likely to happen.
- In blocky hard rock masses, under high stresses, depending on rock mass block size wide ranges of failure modes such as buckling, bending, tensile splitting, ejection, and bulking widely occurs. Nonetheless, in such condition with low stresses, block fall, sliding and toppling most likely to occurs.
- In heavily fractured rocks, bulking, buckling, ravelling and flowing are the main failure types.
- In the ground with special minerals, rocks may show physico-chemical behaviour such as swelling, dissolution and flowing due to the spectacular conditions.

In figure 14, the classification of sudden or unstable failure mechanisms is illustrated. According to this classification; firstly, sudden failure could take place in massive intact rock or along with pre-existing discontinuities, secondly, failure can occur self-initiated strain burst due to stress concentration or rock burst triggered by seismic sources (natural or engineered seismic), and thirdly sudden failure takes different forms at different structures such as tunnels, pillars and caverns or mine stopes.

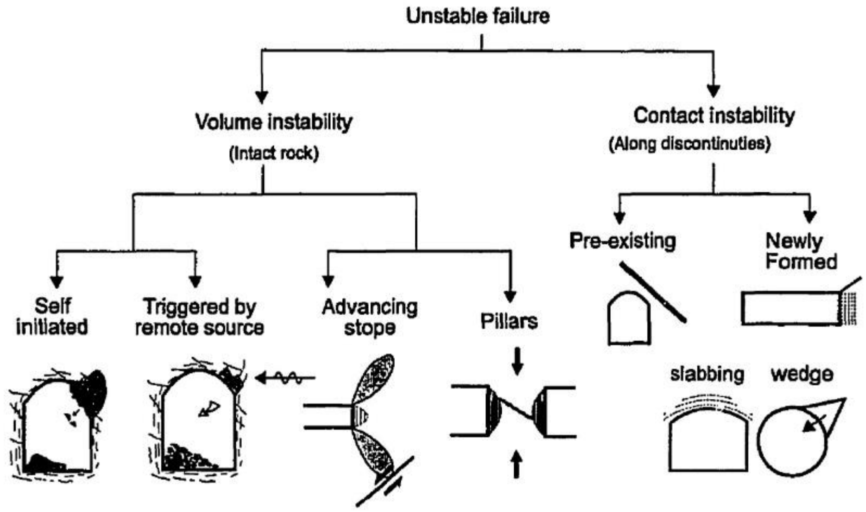


Figure 14. Unstable failure classification in underground excavations (AgLawe, 1999).

He et al., 2017 categorised the rockburst according to triggering mechanisms and the based on laboratory physical modeling results. Rockburst may happen due to; (i) in a highly stressed rock mass storing a large amount of the strain energy during tunnel excavations or face stopping phase, and (ii) less stressed and deformed rock triggered by the external disturbances such as blasting, caving, and adjacent tunneling, etc. Based on this knowledge, rock- burst phenomena under two different conditions can be categorized into two major classes as illustrated in Figure 15.

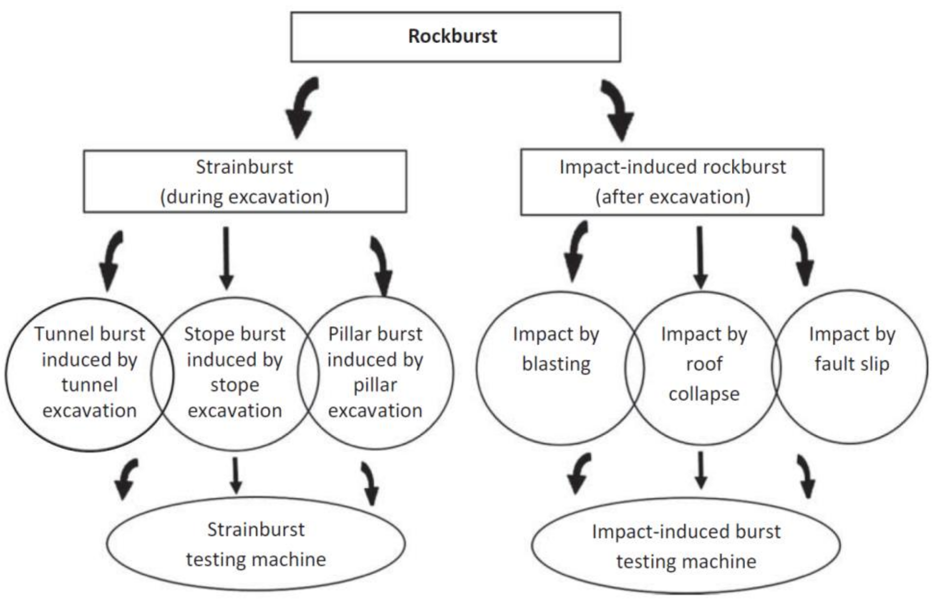


Figure 15. Rockburst classification based the triggering mechanism and experimental methods (He et al., 2017).

With these failure mechanisms in mind, it is noteworthy that in practice, failure may comprises the combination few of above mentioned mechanisms. For instance prior to major rock burst, rocks chipping and spalling take place at tunnel circumference. From the engineering point of view, rock spalling could be used as sign and precursor for major rock burst. Since rock cracking and consequently spalling are associated with noise which can be captured by micro-seismic monitoring with advancing the tunnel excavation as shown in figure 16 (Feng et al. 2017). Using the system presented in figure 16, makes it possible to predict sudden failure in deep underground excavation and prevent the probable catastrophic consequences to crew and machinery. Several cases of sudden failure detection and timely evacuation were reported in literatures.

Australia experiencing deep mining and increases in stress concentration along with seismic activities several modes of rock failure such as spalling, rock burst, buckling and bulking, splitting, squeezing were observed. A specific examples of failure modes in an underground gold mine in Western Australian is shown in Figure 16. The mine geology indicate that, it is located in complex geological structure including mafic and ultramafic volcanic. The gold deposits are hosted within the mafic stratigraphical units, and each basalt unit is defined by internal volcanic textures including coarse grain, massive, and pillowed basalt types. The mine layout and geological structure of orebody and hanging and footwall is shown in figure 17. The geological structure as shown in figure 17; the Hanging wall and footwall are basalts and orebody consists of basalt, shear zone and dolerite. The uniaxial strength of the intact basalt is about 450MPa which is categorised as vert hard rock. The hanging wall, orebody and foot wall rock quality is fair (Q=4-8) according to Q classification system. Despite such very strong intact basalts and having three discontinuity sets, rock mass heavily suffers from foliation, shear zone and faults. The major, intermediate and minor principle stresses at the depth of 600 m are estimated as 46 MPa, 29 MPa, and 18 MPa respectively. The rock mass condition, stress concentration around roadway tunnels and stopes along with seismic activities cause declines and stope instability problems such as slabbing, buckling, bulking, ejection and squeezing. Some of the captures failure mechanisms were illustrated in Figure 18.

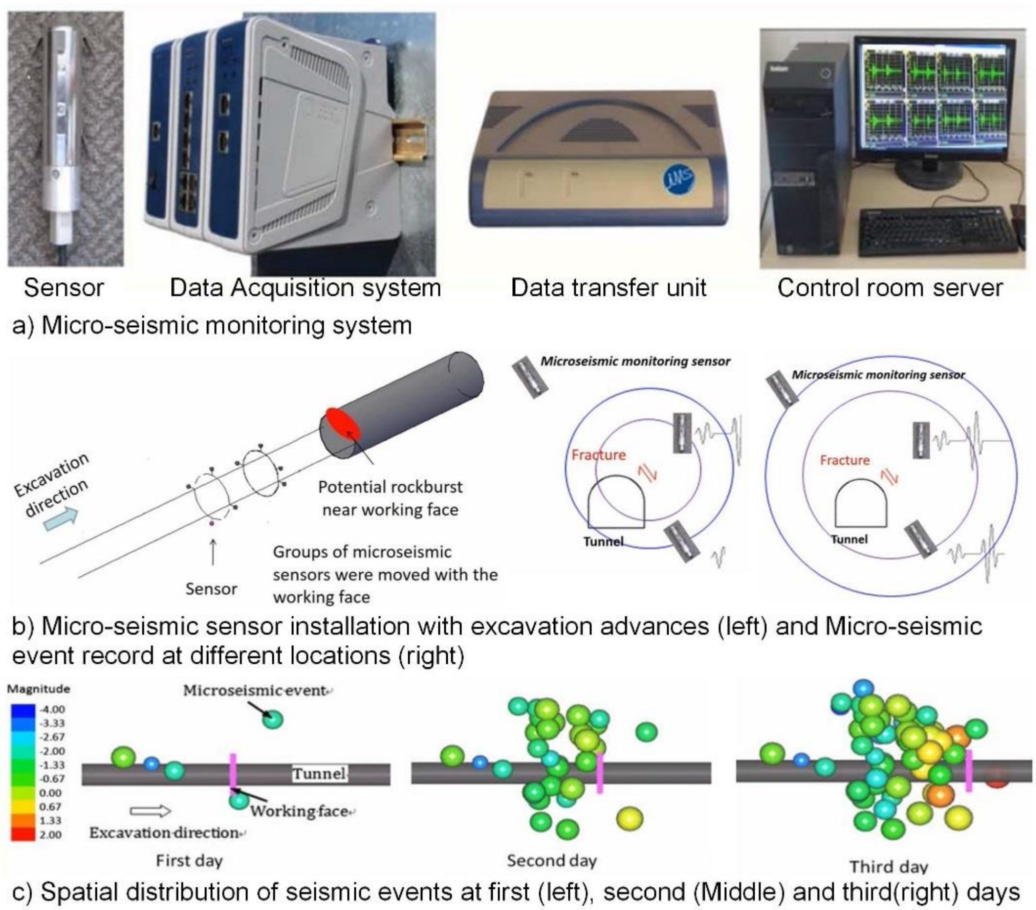


Figure 16. Field monitoring of seismic event occurrence pattern to predict sudden failure in deep and high stress excavation in hard rocks; a) micro-seismic monitoring instruments, b) sensors installation pattern along excavation advances, and Micro-seismic event records at different locations, and c) spatial distribution of seismic events at first, second and third days (Feng et al. 2017).

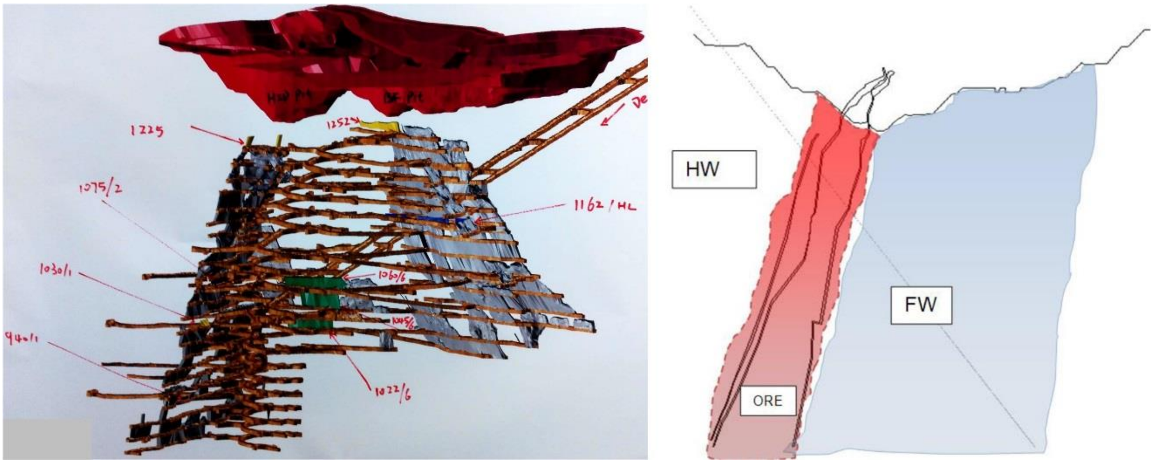
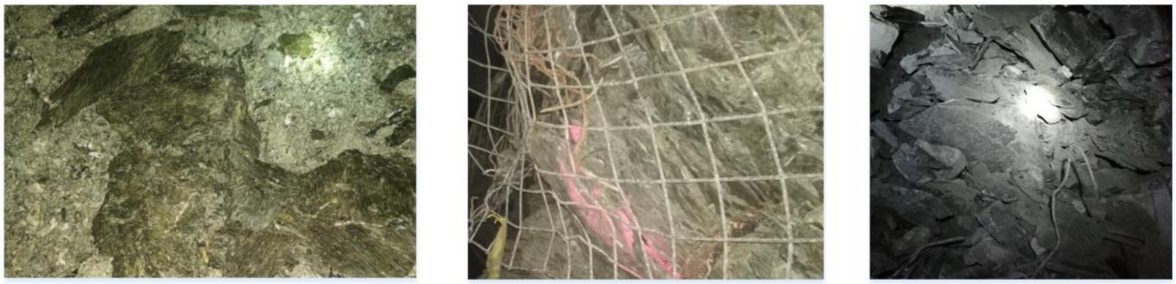


Figure 17. Left; Underground gold mine layout, and Right; geological structure of orebody and hanging and footwall.



a) Photos of foliated basaltic rocks (left) in tunnel wall (middle photo) and failed and fallen from roof (right)

b) Photos of tunnel wall undercutting and unravelling (left and middle) and buckling and unravelling (right)

c) Photos of foliated basaltic rocks bulking from side views (left & middle) and front view (right)

a) Photos of minor (left) medium (middle) and major roof collapse and support anchorage loss (right)

Figure 18. Failure mechanisms at very hard foliated basalt at depth of about 500m in underground mine.

Figure 18-a, shows the foliated nature of basalts from front and side views in decline walls and also failed, fallen to floor and divided to pieces. This indicates that such hard and foliated rock, could possibly shows shearing between contacts in parallel stresses to foliation or shows buckling induced stresses.

Figure 18-b, shows the undercutting and unravelling of the foliated basalts in decline walls due to removal of the minor principle stress, buckling and unravelling of the thick basalt layers due to stress concentration on the walls.

Figure 18-c, shows that induced stresses on decline walls cause buckling and bulking of the rocks, which consequently leads to frictional bolts failure. Similar phenomenon also seen as floor heave in other declines.

Figure 18-d, shows that induced stresses and rock mass structure lead to minor, moderate and major failure in decline roof. The major roof failure destroyed the installed support system including frictional rock bolts and mesh.

5- GROUND ENERGY DEMAND AND SUPPORT ENERGY ABSORPTION BASED DESIGN STRATEGY IN DEEP EXCAVATIONS IN ROCK

Stresses on rocks around underground excavation could be originated from different sources such as ground principle stresses, stress concentration due to excavation, rock strength reduction due to the unloading in one (tunnel) two (junction or pillars) and more importantly natural or engineered seismic events leads to rapid loading. The stresses and deformation concentrated cause energy accumulation in rocks and if the accumulated energy exceeds depending on stress path, rock likely to experience stable and unstable cracking, and in absence of sufficient support system may suddenly fails and falls to residual strength as explained in figure 12. For quantitative evaluation, of energy accumulation under dynamic condition, there are two main kinds of energy which are the kinetic energy caused by a seismic event and the potential energy induced with the bulked rock mass moving under gravity. Hence, the capacity of support should absorb two kinds of energy above and the equation of energy absorption requirement is shown below:

$$\boldsymbol{E_a = \frac{1}{2} \times m \times v^2 + q \times m \times g \times d}$$

Where; E_a is energy absorption requirement (KJ/m^2), M is Mass of potentially ejected rock (kg), V is ejection velocity (m/s), Q is direction factor (a constant equal to 1 for a rockburst from the backs, 0 from the wall and -1 from the floor), G is acceleration due to gravity (m/s^2), and D is distance the failed material travel (m), commonly assumed 0.2m which is an allowable displacement of the excavation walls during a rockburst. To achieve stability, the support system must be able to absorb the energy released by ground. The energy absorption capacity of different reinforcement systems (rock bolts) is shown in Figure 15. The detail calculation avoided here, more information could be found in (Kaiser et al.,1996, and Louchniokv & Sandy, 2017).

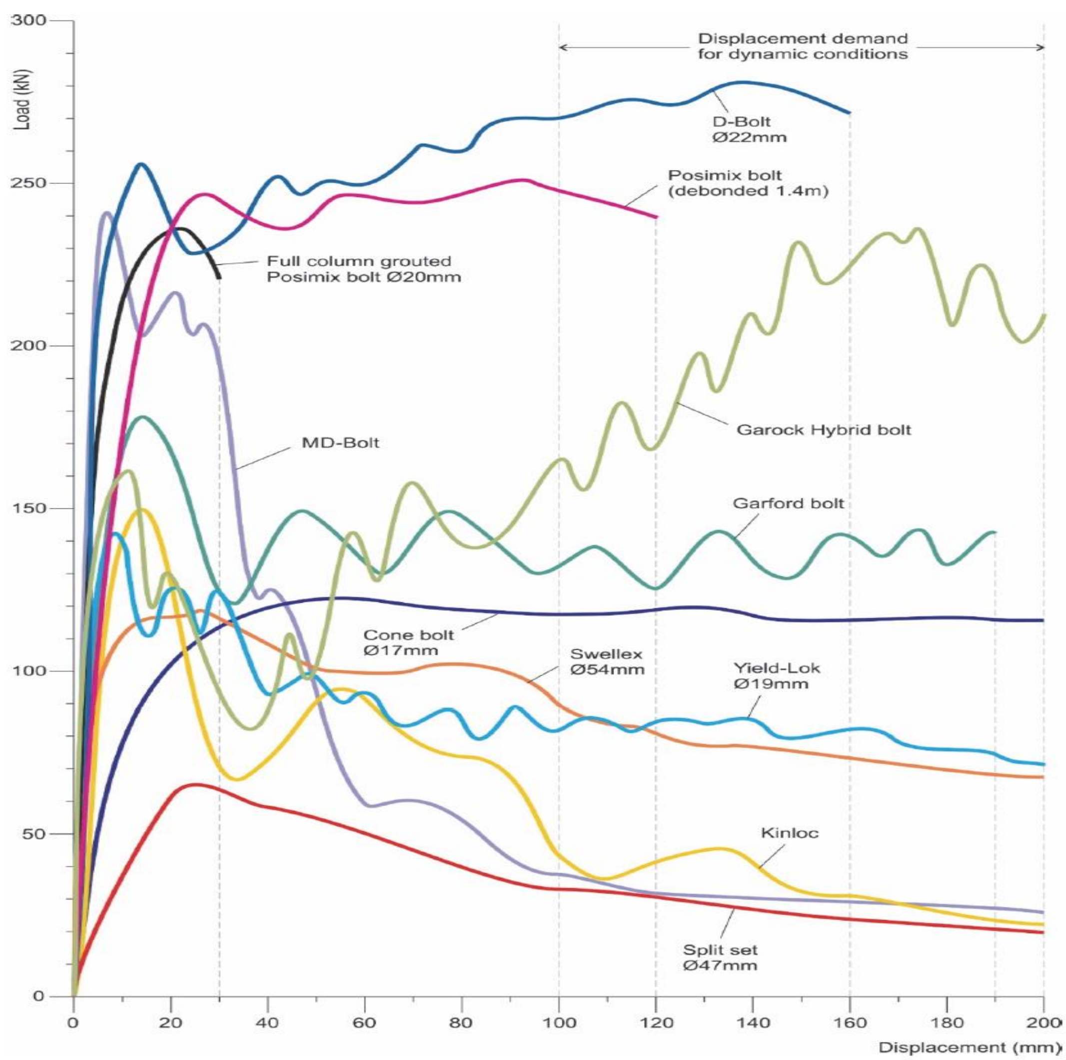


Figure 15: Typical load-displacement characteristics of commercially available rockbolts determined by dynamic testing (Louchnikov and Sandy, 2017).

Rock support system design methods in sudden failure (rockburst) prone zones could evaluated using criteria illustrated in table 1.(Kaiser and Cai, 2012).

Table 1. design criteria for tunnel support in sudden failure prone zones.

	Criterion	Formula	Remarks
1	Force criterion	$FS_{load} = \frac{support\ load\ capacity}{load\ demand}$	FS_{load}= The load safety factor, Where load consists of both static and dynamic.
2	Displacement criterion	$FS_{dis} = \frac{support\ displacement\ capacity}{displacement\ demand}$	FS_{dis}= The displacement safety factor
3	Energy criterion	$FS_{enerfy} = \frac{support\ energy\ capacity}{energy\ demand}$	FS_{energy}= The energy safety factor
	System compatibility criterion	This criterion is established to guarantee the three mentioned criteria above are compatible.	

Kaiser and Cai (2012) pointed out that rockbursting is a complicated phenomenon which can be induced by mining and can be observed in underground openings at great depth, and it is difficult to design a confident supporting system under rockburst conditions, owing to lack of knowledge of rockburst mechanism and exact damage procedures.

Seven simple principles applied in prone burst ground were summarized, which are a. avoid rockburst; b. flexibility / yielding support; c. address the weakest link; d. intergrated system support; e. simplicity; f. cost-effectiveness; g. observational construction (Cai & Champaigne, 2009).

However ground control design methods have evolved over recent decades, but in such a complex rock mass, static and dynamic stresses, and excavation characteristics, fully understanding of the system is almost impossible. On one hand, this is due to lack of capability to capture proper design data reliable data such as rock mass characteristics, ground stresses and its variation over time. On the other hand, however the simple rock behaviour mechanisms such as block sliding, shearing understanding are comprehensible, but understanding the rock mass behaviour mechanisms and mechanism transition in complex geological conditions such as initial behaviour or precursor, secondary or tertiary behaviour, and final residual behaviour are extremely difficult. In such complex situation, sound judgement from a team of relevant disciplines could lead to efficient design outcomes, where they considers most probable scenarios according to previous experiences. It is also noteworthy that, considering the limitations of computer (Analytical or numerical) simulation methods, cautions must be taken in application of them. The reliability on simulating results closely dependent on input rock mass geometry, geological material properties, and loading conditions, in which, as discussed earlier, huge uncertainty associated for determining those in put data.

6- CONCLUSION

As a consequence of rapid growing trend of resource extraction in world, depth of excavations for resource exploitation increases. Eventually excavations faces with transition from low stress to high stress condition. In this paper, comprehensive aspects on rock behaviour at deep underground excavation were investigated. The state of art of rock behaviour at micro- meso- and macro-scale were discussed and relevant challenges along with achieved knowledge, experiences, and research results were presented.

At micro-scale, research results revealed that:

- In the atomic scale rocks with covalent bonding shows brittle behaviour.
- Rock micro-structure closely depends on bonding dimensions, rocks with two dimensional bonding leads to foliation and consequently anisotropic rocks behaviour, while uniform rocks with three-dimensional bonding structure shows higher degree of anisotropy.
- Apart from chemical bonding, rock behaviour significantly influenced by deficiencies such as; particle-crystal boundaries, heterogeneity, pores and micro-cracks, which reduces the rock strength 2-3 order of magnitude.
- Granite SEM images proves the deficiencies between crystals, micro-cracks and pores at each crystal, and weakness and foliation of mica components in granite.
- When stresses applied on specimen, new tensile cracks nucleated and initiated from the edge of existing micro-cracks, and rate of crack propagation depends on the differential stress level.

At meso-scale, research results proves that:

- True triaxial test compared to conventional triaxial tests represents ground stress condition in more realistic way.

- True triaxial testing makes it possible to apply different stress paths in the ranges of ground in situ stresses, concentrated stresses and even dynamic loads (resembling seismicity). There are also capability to study the rapid unloading one of principle stresses on rock specimen to simulate the effect of excavation on rock behaviour.
- Careful assessment of the full stress–strain curves of the true triaxial test results of granite and conventional triaxial test results of Marble shows that rock mechanical properties such as magnitude of linear elasticity, ductility domain, peak strength value, ranges of brittleness, and residual strength level significantly differs with changing confining stresses. The rock stress – strain behaviour variation were categorised to four distinct stages consisting; 1) Elastic-stable micro-cracking, 2) Stable - unstable micro-cracking, 3) Unstable micro-cracking-brittle failure, and 4) Brittle failure-residual strength. The ranges of rock behaviour at each stage with different confining stresses were illustrated, which could be used as input for mechanical parameters in design analysis.
- However the existing cohesion and friction models were introduced, but using the abovementioned rock behaviour categorisation, more conceptual models could be developed.

At macro-scale, investigation results verifies that:

- Counteraction between ‘Rock Mass Composition (RMC)’, ‘Active Stress Condition (ASC)’, and ‘Excavation Method, Size and Orientation (EMSO)’ to estimate the ‘Rock Mass Behaviour (RMB)’ were discussed and presented as a verbal equation. Clear understanding of these factors and their associated sub-factors and their proportional effect along with sound engineering judgement could results appropriate counter measure to achieve safe and economically cost effective excavation design.
- Using the presented verbal equation and experiences, rock mass behaviour (RMB) were predicted in wide range rock masses quality and stress conditions.
- To reduce the sudden failure risk and consequent fatalities, a micro-seismic monitoring system were designed and implemented for perdition and warning of failure and evacuation in timely manner.
- To verify the presented approaches, rock mass behaviour and failure mechanisms were illustrated in a deep gold mine in Western Australia.

To manage the ground behaviour (so called ground control); considering the static and dynamic loading and interlocked nature of rock masses at deep underground excavations, the ratio of “Ground energy demand” to “support energy absorption capacity” is mostly used for stability evaluation.

To sum up, it should be noted that the geomechanics at general and deep underground geomechanics specifically is a developing field due to lack of capability to achieve proper ground characteristics, huge number of variables and their coupled interactions, and lack of capability to analysis them properly. Therefore, the results from abovementioned analysis should not be taken as granted and always solid engineering judgement must involve in interpretation and design. It is also hoped that future development in sophisticated ground exploration technologies along with advances in computation science will assist geomechanics engineers to mature their understanding of rock behaviour in engineering activities.

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