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Fracture Mechanics Induced by Drilling and Blasting in Underground Openings: Prevailing Practices and Studies

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Abstract

This article presents the prevailing practices and literature review on the study of fracture mechanics of rock mass, during the construction of underground openings, by drill and blast method (DBM). The paper makes an introductory discussion on basic fracture and failure mechanics on the rock, followed by a review due to the blast-induced failure mechanism. The damage zone in drill and blast openings is discussed which is followed by the conclusions of the literature. DBM is a widely acceptable and broadly applicable approach in the underground construction method. The blasting damaged zones are classified as the borehole expansion zone, crushed zone, nonlinear fracture zone, and radial crack propagation zone. Blasting activities create stress waves in construction work, which fundamentally influence the fracture and failure mechanics of rock with pre-existing discontinuities, in-situ stress, and groundwater conditions. On the other hand, the cumulative action of subsequent explosion gas and stress waves defines the final damage limit and crack pattern map. Thus, it is of utmost importance to investigate the blast-induced fracture and failure mechanism to mitigate the instability in the underground openings.

Keywords: *Rocks, Drill and Blast Method, Failure Mechanics, In-situ Stress, Underground Opening*

1. Introduction

The Drill and Blast method (DBM) is a widely acceptable and broadly applicable underground construction method that is easy to mobilize, flexible with respect to equipment selection, minimum initial investment, and the possibility of construction of different openings shapes, and sizes. However, the surrounding rock mass experiences the formation of new cracks which may lead to short-term and long-term stability problems. If poorly blasted, the project cost may increase by 15% due to the blast-induced rock mass overbreak (Verma et al., 2018). From the study by Ramulu et al., 2009, it was found that the overbreak in compact basalt and amygdaloidal basalt varies from 5%-30% and which increases the project cost due to the installation of different support systems to reduce the instability in the tunnel. Rock mass damage due to the blasting is characterized by the propagation of geological discontinuities and new crack formations, which are mainly affected by controllable and uncontrollable parameters. The controlled parameters are rock mass overburden, stemming, spacing, sub-drilling, and delay time, while uncontrolled parameters are due to the geotechnical properties of surrounding rock masses. Controlled parameters can be optimized to reduce blast-induced unfavorable effects during the blast design phase. The uncontrolled parameters are complicated for theoretical modeling between rock mass and blast response (Zhao et al., 2016).

This paper presents a discussion of the prevailing literature reviews and practices in drill and blast-induced fracture mechanics. The article is prepared by referring to the various published articles on the relevant field of the topic. The study is primarily composed of experimental work and numerical modeling done by researchers listed in the references.

2. Fracture and Energy in rocks

2.1 Fracture and failure mechanics of rock

The laboratory experiment was conducted by Martin and Chandler (1994) under uniaxial compression and the measurement result classified in different deformations stages, these were found as crack closure, fracture initiation (elastic deformation), stable crack propagation (or critical energy release), unstable crack propagation (or material failure), and post-failure behavior (or structure failure), which are indicated as Stage II, Stage III, Stage IV, and Stage V, respectively as shown in Figure 1. In Stage III, it was found that the crack initiation stress (σ_{ci}) is about 40%–60% of the peak strength (σ_p) and crack coalescence stress is about 70%–90% of the peak strength (σ_p), and stable crack growth occurred between these zones (Martin and Chandler, 1994).

The main progressive failure stages in the rock mass are defined as the structural activation (initial crack compaction) stage, structural damage (linear elastic deformation) stage, structural adjustment (Stable fracture propagation) stage, and structural instability (unstable fracture propagation) stages, which are based on the characteristics of structural evolution and is shown in Figure 2 (Song and Zhang, 2022; Abioye, 2019).

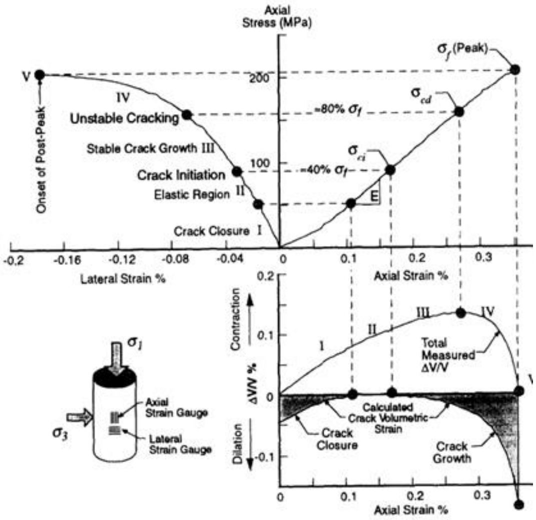

Figure1: Stress-strain Diagram showing Stages in the Failure Process (Martin and Chandler, 1994)

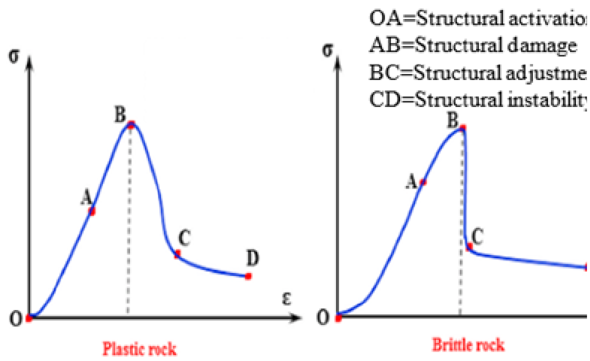

Figure 2: Progressive failure process based on Structural evolution perspective (Song and Zhang, 2022)

2.2 Energy transformations during rock fracture and failure

Failure and fracture mechanics of the rock are based on the energy prospective under compressive loading. Minimum driving energy in the rock transforms is the energy consumption from the high-state (shear resistance by cohesion i.e., zone C, see Figure 3) to the medium-state (friction zone A), and finally to the low-state (initial fracture zone B). With the initiation of existing microcracks in zone B, the area of zone C decreases (with reducing shear strength) and increases the area of zone A. In zone B short tensile cracks are formed gradually, and finally, the frictional resistance replaced the shear resistance of zone C as shown in Figure 3 (Song and Zhang, 2022).

In plastic rock (Class I, i.e., Peak modulus < 0), shear resistance due to cohesion gradually changed into the frictional resistance of the fracture surfaces. Consequently, greater energy is required for continuous fracture, propagation, and collapse of rock blocks. In brittle rocks (Class II, i.e., Peak modulus > 0), during the continuous rotation process in zone A, fan-shaped structures are formed by consuming less energy with minimum shear resistance. If the shear resistance is zero, the crack initiation and propagation are maintained only due to self-accumulated energy, which produces the most energy-efficient shear fracture mechanism. Eventually, it was concluded that the Class II type rock has a high energy dissipation rate (enough energy for failure development) than the Class I rock, where some energy loss during shear resistance occurs, i.e., the energy available from the material is insufficient (Song and Zhang, 2022; Tarasov and Potvin, 2013). Figure 4 shows the stress-strain curves of Class I and Class II rocks (Li, 2021).

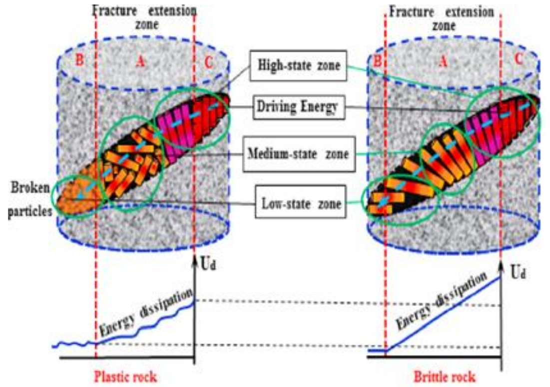

Figure 3: Post-peak progressive fracture mechanism (Song and Zhang, 2022)

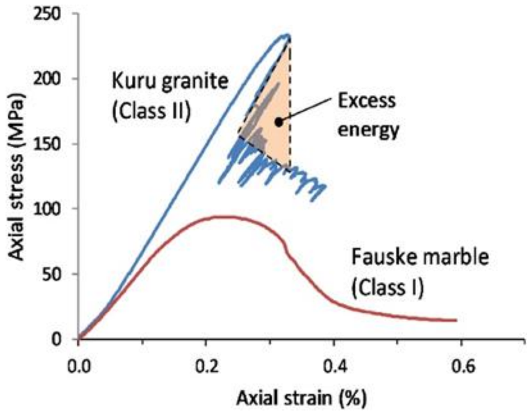

Figure 4: Stress-strain curves of Class I and Class II rocks (Li, 2021)

3 Blast-Induced Failure Mechanism

3.1 Failure mechanics in Single hole Blasting

Generally, shock wave (stress waves) and explosion gas pressure loading are ~~the~~ two basic forms of energy during rock blasting (Yilmaz and Unlu, 2013; Zhu et al., 2007). Blast-induced crack evolution mechanisms simulation under hydrostatic pressure and non-hydrostatic pressure were investigated by comparing specific decoupling coefficients. The blasting damaged zones were divided into the borehole expansion zone, crushed zone, nonlinear fracture zone, and radial crack propagation zone (Li et al., 2020).

3.1.1 Theoretical Background

After blasting in the initial borehole, it was enlarged as an uneven periphery and called an expanded borehole as shown in Figure 5. Likewise, adjacent to this expanded borehole zone, rock mass was found in broken and crushed nature, called the crushed zone. Furthermore, the network of circumferential and dense radial cracks propagation in the crushed zone is surrounded by a fractured zone with radial cracks (Chi et al., 2019). Likewise, it was observed that the crushed zone was formed due to the blast shock waves and radial cracks were initiated surrounding this crushed zone, while these radial cracks were further extended by the gas wave (Yilmaz and Unlu, 2013).

The decoupling ratio of the charge greatly influenced the expansion of the borehole and the size of the crushed zone, both were decreasing with increasing decoupling ratio. Moreover, it was found that the crack extension length in the water-coupling medium is 1.2 times longer than in sand-coupled and six times extended than the coupling in the air medium. Consequently, an air coupling medium is more appropriate to minimize rock mass damage due to blasting (Zhu et al., 2007). In a fractured zone on the other hand, the boundary condition of the granite rock greatly influenced the radial crack propagation. In Figure 6, (Chi et al., 2019) it was noticed that the fully confined boundary conditions sample S1 (5.61 g charge) and S2 (5.47 g charge) have one radial crack which was propagated in the boundary of the cylinder and its position is in the radial direction of drill-hole. Furthermore, in the gravelly-confined sample S3 (5.52 g charge) and S6 (2.00 g charge), the propagation of radial crack developed form drill hole to the periphery of the sample, eventually which divided the sample into pieces. By comparing of this radial crack propagation number and pattern, it was concluded that, few and short radial crack lengths were developed in fully confined boundary condition sample than in the gravel-confined conditions.

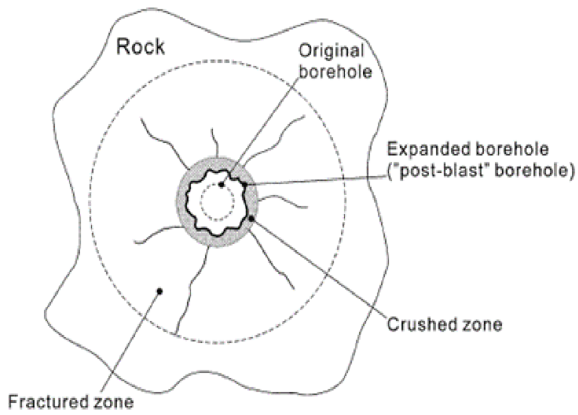

Figure 5: Definition of damage zones surrounding the borehole (Chi et al., 2019)

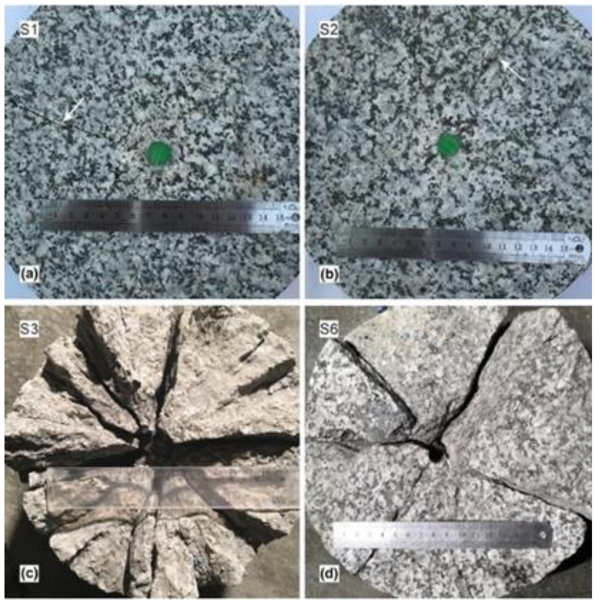

Figure 6: Radial cracks after blast: (a) S1 (b) S2 (c) S3 (d) S6 (Chi et al., 2019)

3.1.2 Numerical Modelling

The blasting fracture mechanism was investigated by Zhu et al., (2007) with numerical modeling process and the fractured zone in rock was classified as a crushed, fractured, and process. In the crushed zone, the rock mass is severely crushed near the borehole due to the high shear stress (from high radial compressive stresses). Additionally, fractured zones occur around the crushed zone, where the major tensile stress causes severe radial crack due to a very high crack density. Also, in the incipiently cracked zone, circumferential cracks are caused by reflected wave stress from the free boundary surface but not reflected in the transmitting boundary condition. Eventually, it was found that the shear failure condition occurs in the crushed zone and tensile failure in the severely fractured zone as shown in

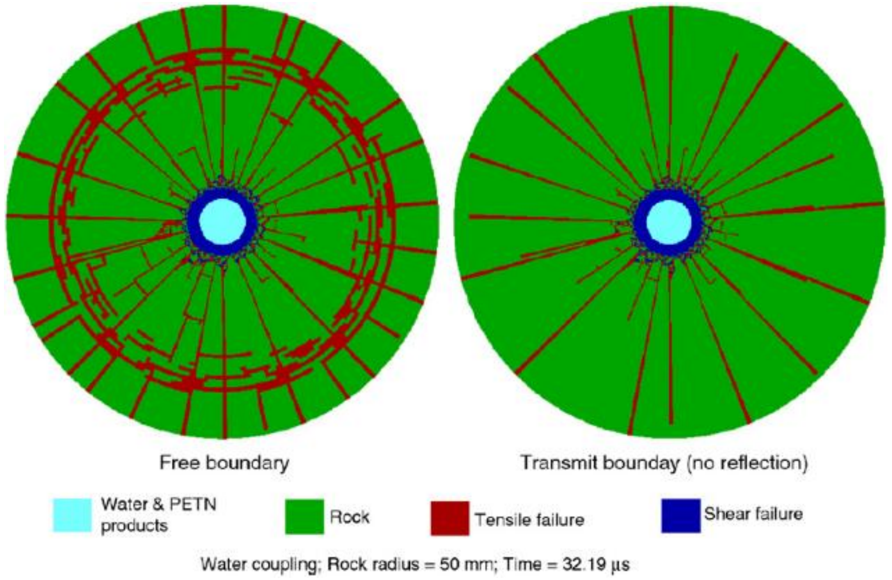

Figure 7 (Zhu et al., 2007). Also, the crack propagation direction in the radial zone was directed along the maximum principal stress direction and controlled by in-situ vertical stress. Finally, it was found that the increase in in situ stress decreases the blasting decoupling coefficient (Li et al, 2022).

Figure 7: Fracture mode in different boundaries (Zhu et al., 2007)

3.2 Effect of in-situ stress

The Rock fracturing is a process of crack initiation, propagation, and fracturing under external loading. The combined effect of high ground stress and blasting stress highly influences the process of crack propagation in the case of the blasting of deep rock (Yang and Ding, 2018). The blast-induced crack propagation and blasting design parameters in an underground opening are mainly influenced by the in-situ stress of the surrounding rock mass (Lu et al, 2012). High insitu stress highly suppressed the radial crack propagation in the radial zone but it has not much impact on crack propagation in the crushed as well as in the nonlinear fracture zone because during blasting, initial rock mass strength is much lesser than the pressure released due explosive during the blasting process. Also, the direction of radial crack and direction of maximum principal stress were directed in the same direction, where this propagation of crack was controlled and directed by the in-situ stress. Furthermore, it was observed that, the difficulty in the blasting increases due to increase in applied biaxial stress. This increased in-situ stress highly reduces the radial crack length of crack propagation in the rock mass. Also, this crack propagation in the rock mass becomes anisotropic. Eventually, it was observed that the length of the crack decreases with increases in the in-situ stress condition, which is shown in Figure 8 (Li et al, 2020).

Blast-induced fracture mechanisms in Laurentian Granite rock under confining in-situ stresses were investigated theoretically and verified by numerical modeling by Yi et al. (2017) where the result shows that the damage evolution, crack initiation, and developments were significantly affected by the level and state of in-situ stress. Likewise, evolution-induced dynamic stress highly dominated the crack propagation during the early phase however very quickly decayed in the crack vicinity (Yi et al., 2017).

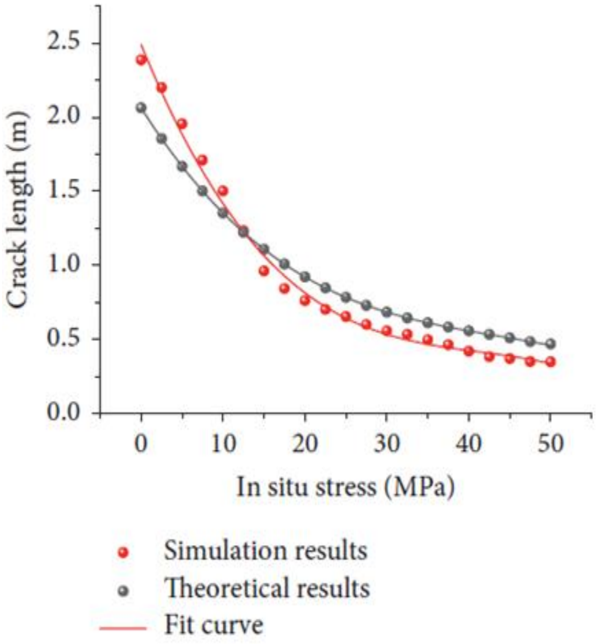

Figure 8: Crack lengths with various in situ stresses (Li et al, 2020)

Under uniaxial compressive static loading, it was observed that the maximum principal stress direction and the blast-induced crack propagation direction were aligned in the same direction (Yi et al., 2017; Kutter and Fairhust, 1971). This combined effect of initial static stress and blasting dynamic stress propagated the longest crack in the direction of initial static stress. The main crack propagation direction was deflected along the direction of principal stress due to an increase in the level of stress concentration. Also, it was found that with the increased stress, the propagation time of the main crack became shorter, and eventually, the fracture of the specimen was classified as a (shear fracture) failure mode II (Yang and Ding, 2018).

3.3 Effect of groundwater in the rock mass

In water-bearing rock mass with initial crack and discontinuities, a small diameter hole was blasted, and the effects of this single blast hole on the rock mass crack propagation and the fracture were analyzed by Wang et al. (2018). It was found that the presence of water in the rock mass accelerates the stress wave propagation during blasting and mode -I type (tensile fracture) crack propagation occurred. The amount of rock fracturing and vibration velocities during blasting decreased nonlinearly with increased blast source distance from the monitoring location

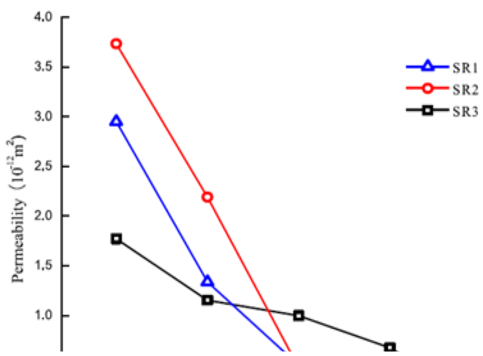

Figure 9: Permeability versus confining pressure (Zhao et al., 2016)

(Wang et al., 2018). Confining pressure in the rock fracture process is highly susceptible with the permeability of single fracture and fracture roughness effect in the rock mass. It was noticed that, primarily permeability decreased with tremendously due to increase in the confining pressure, after certain lowering the value of the confining pressure, the decreasing rate observed as slower as shown in Figure 9. Eventually, the fracture mechanism of rock mass was described by Darcy's law in the linear relationship between the flow velocity and rock fracture hydraulic gradient (Zhao et al., 2016).

3.4 Blasting in a rock mass with a pre-existing crack

The crack near the tunnel was highly disturbed by a high amplitude blasting load. Under blasting disturbance, initiation and propagation of this crack finally lead to catastrophic failure in the tunnel, which created risk and problems in the tunnel construction, eventually, delaying the construction progress and increasing project cost (Liu et al., 2016). External crack around a tunnel (ECT) was a Physical model built with a crack propagation gauge (CPG), which is used to record crack initiation and propagation time with a growth path in the green sandstone during the blasting load as shown in Figure 10. In this investigation, the failure mode due to the influence of blasting load, the tunnel symmetrical axis angle (α) was changed by measuring in the counterclockwise horizontal direction. Blasting in the underground opening with pre-existing joint was highly influenced the crack propagation in the existing tunnel. In this case, the failure zones were mainly separated into three regions like, surrounding area of blasthole, shadow and incident side of the tunnel (Li et al., 2022).

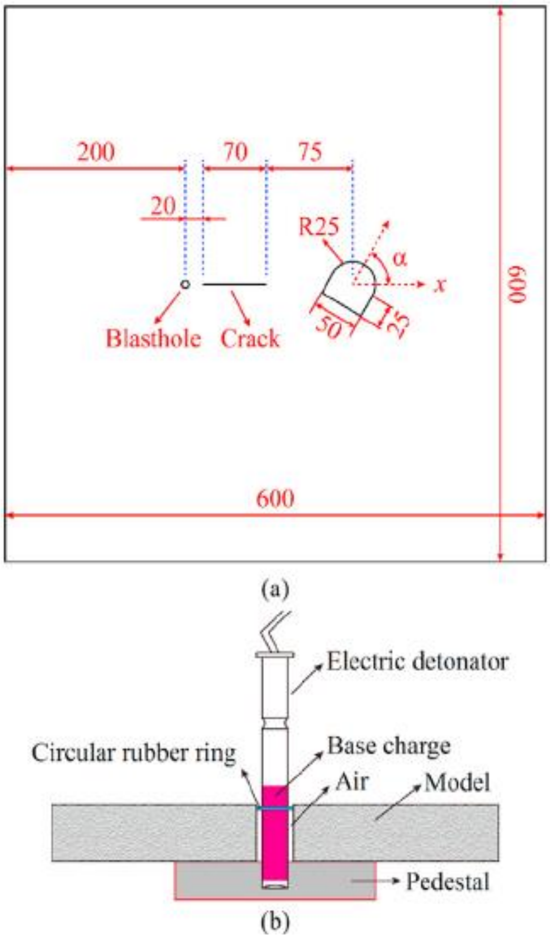

Figure 10: Sketches of (a) ECT model and (b) blast loading system in Green Sandstone Rock The unit for the dimensions in (a) is mm (Li et al., 2022)

In this study, it was observed that the blasting loading direction can influence the failure pattern in the incident and shadow sides of the tunnel with the pre-existing crack in the rock mass. When blasting direction α = 0^0, the existing crack propagated in the mid of the floor.

Also, with increasing the blasting angle α = 30^0, 45^0 and 60^0 it was noted that the propagation of crack in the tunnel corner and propagates in the spandrel of the tunnel when the blasting direction α = 90^0. Additionally, when the blasting direction α = 120^0,135^0,150^0, the pre-existing crack propagated at the collision point of the propagation of pre-existing crack and tunnel. These blasting direction and its crack pattern and position shown in Figure 11 (Li et al., 2022). At the initial stage of fracture mechanics of an existing crack, the incident stress wave predominantly influenced the initiation and propagation of the crack. Furthermore, it was observed that the crack initiated, and propagated crack failure was represented by a tensile failure mode (the mode I). After that, at the tunnel incident side, the reflected wave initiated the new crack and propagated towards the existing crack, and ultimately merged. Eventually, it was found that the crack propagation was characterized as a tensile failure, when α = 0^0, 45^0, 90^0, 120^0, 150^0, and 180^0. Also, when α = 30^0, the crack failure pattern was shear, whereas the mixed tensile-shear failure pattern exhibits when α = 60^0,135^0 and these results shown in Figure 12 (Li et al., 2022).

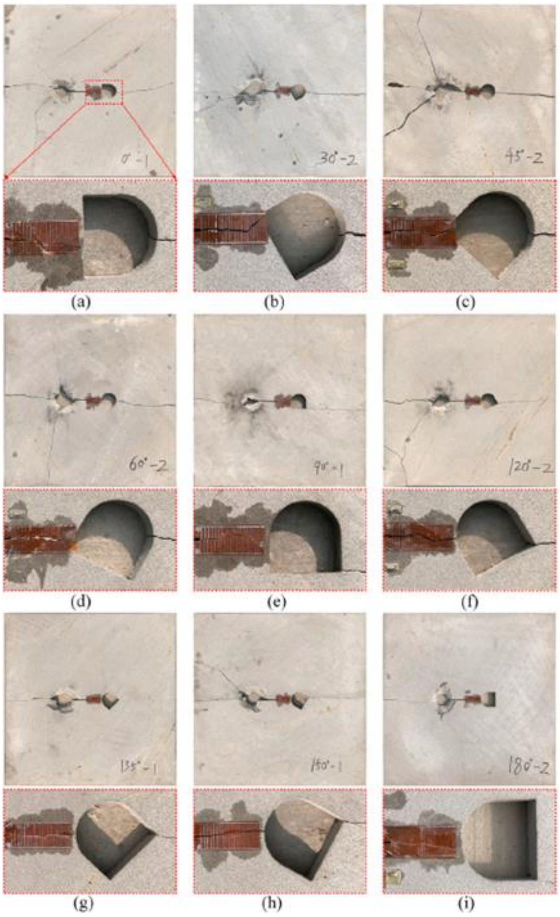

Figure 11: Failure modes of ECT models (a) α = 0^0 (b) α = 30^0 (c) α = 45^0 (d) α = 60^0 (e) α = 90^0 (f) α = 120^0 (g) α = 135^0 (h) α = 150^0 (i) α = 180^0 (Li et al., 2022)

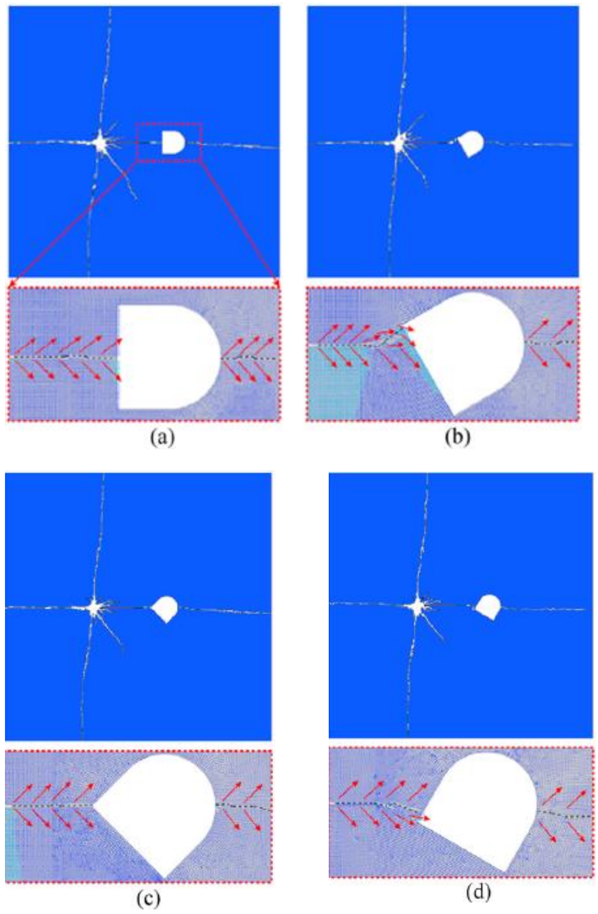

Figure 12: Failure modes (a) α = 0^0 (b) α = 30^0 (c) α = 45^0 (d) α = 60^0 (Li et al., 2022)

4 Damaged zone in Drill and Blast Opening

The three tunnel construction sites in the Himalayan region in India were studied by Verma et al. (2018) for in-situ stress and pre-existing fracturing of different rock formations (thinly bedded phyllite quartzite, quartz biotite schist, augen gneiss). Moreover, the level of the damaged zone was analyzed with a quality index Q rating and blast design parameter. During construction, peripheral rock mass damage is defined as overbreak zone, damaged zone, and disturbed zone as shown in Figure 13. Eventually, it was concluded that the rock mass quality index Q rating was reduced by the blasting operation.

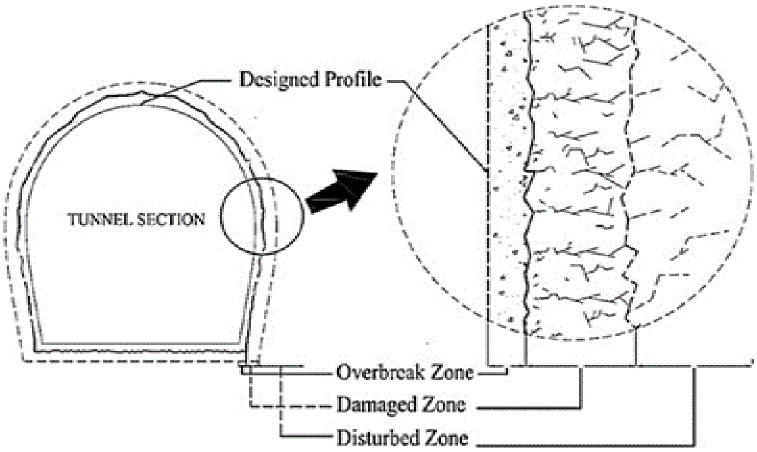

Figure 13: Blast-induced Rock Mass Damage Zone Tunnels (Verma et al., 2017)

The overbreak zone occurs on the periphery of the designed profile line, where rock is completely detached from the rock mass during underground excavation. Excavation of rock mass by blasting generates the network of micro-cracks and fractures and rock mass properties changed permanently in the damaged zone, which is lying around the overbreak zone (Saiang and Nordlund, 2009).

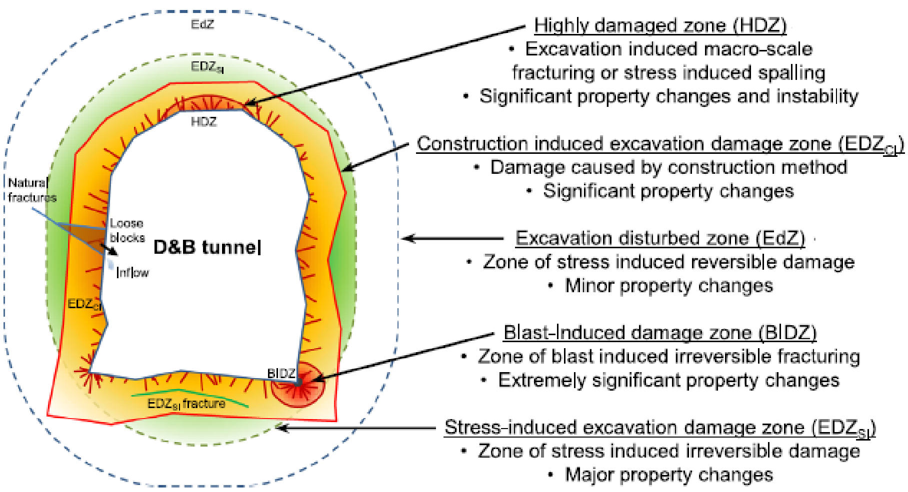

Figure 14: Different Excavation induced damage zones (Siren et al., 2015)

In situ stress and hydraulic permeability of rock mass are highly dominated in the disturbed zone which lies in the periphery of the damaged zone (Palmström and Singh, 2001). In previous research and findings, the damaged zone was distinguished as the stress-induced excavation damage zone (EDZ_{SI}) and the construction-induced-excavation damage zone (EDZ_{CI}) produced by mechanical excavation or by an explosion. The description of damages terminologies was collected as excavation damage zone and other excavation-induced damaged zones are described in Figure 14.

Furthermore, at the point of peak stress, the effects of the EDZSI were noteworthy but these EDZSI and EDZCI effects always occurred all around the tunnel perimeter (Siren et al., 2015; Malmgren et al., 2006).

The fracture mechanics model was used to investigate the fractured depth and crack pattern behind the excavated surface, where it was found that the construction-induced damage depth is greater than the stress-induced damage depth. Also, in the Drill and Blast tunnel, the tensile condition of fracture in the tunnel floor was observed. However, it was found that the stress-induced damage to the floor and roof but not to the wall (Malmgren et al., 2006).

5 Conclusions

The comprehensive review on the study made by the different researchers has been presented in this article. The article presents the introduction to DBM, theory of fracture mechanics of rocks and energy transformations. The DBM induced failure and fracture mechanisms are explained by theoretical, experimental, and numerical approaches. Also, the effect of in-situ stress, ground water, and pre-existing cracks in tunnels are presented. Moreover, it can be concluded that the stress-strain behavior of brittle and plastic rocks is different after post-peak failure. Further, the confining pressure influence the rock mass which is validated by experimental and numerical models. It was found that the crack length is inversely proportional to the in-situ stress. Similarly, permeability in rock is also inversely proportional to the confining pressure. Likewise, the cumulative action of subsequent explosion gas and stress waves defines the final damage limit and crack pattern map. Besides, it was also observed that the orientation of the blasting angles has significant effect on the DBM tunneling. Further, the effect of DBM in underground openings are classified into different excavated damaged zones, which are Overbreak, damaged & disturbed zones. Eventually, each zones have its high importance in the study of stability of the tunnel and cavern structures. Hence, the study presented in this article will be significantly applicable to the predictions of blast induced fracture instability in the rock mass and its mitigation during underground constructions work by DBM.

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