Practical Rock Spall Prediction in Tunnels

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1 Introduction

Spalling is a mode of damage and overbreak in tunnels at depth in hard rocks (low porosity). Spalling is defined as the development of visible extension fractures under compressive loading near the boundary of an excavation. Varying levels of spall damage are illustrated in Figure 1.

Figure 1: Clockwise from upper left: spall initiation, surficial spalling, low and moderate spall overbreak in a TBM tunnel wall (modified after [1]).

It is important to note that spalling associated with hard rock excavations, while brittle in nature, need not be violent. This process dominates rock damage and failure processes in crystalline rocks near excavation boundaries under high stress. Spalling can be violent or non-violent and in
some cases can be time dependent. In unsupported conditions and under an anisotropic in situ stress field, the process of spalling can form notch geometries often confused with wedge fallout.

Strain bursting is often confused with spalling. Strain bursting is the violent rupture of a volume of wall rock under high stress. In spalling rocks, the spalling damage (extension fractures) can happen before the actual rockburst or strain burst. It is the instability created (example: buckling) by the formation of parallel and thin spall slabs that provide the kinematics for the sudden energy release. While even weak rocks can spall, the ability to store energy, typical of strong rocks, is normally required for strain bursting. Typical examples are shown in Figure 2.

Figure 2 : (Left) Moderate wall spalling and breakout in a TBM tunnel (after [2]; (Right) Strain Burst in a Mining Tunnel.

Extensive research over the last few decades (See [1] for a complete history and references) has demonstrated that brittle non-porous rocks exhibit an maximum long term tunnel wall strength of no more than half of the laboratory test value for unconfined compressive strength (UCS). Damage monitoring [3] indicates that this lower bound strength threshold is less sensitive to confining stress than the peak lab envelope such that the limit for major principal stress is approximately

\[ \sigma_{1\text{max}} = C_l + (1\text{ to } 2)\sigma_3 \]  \[1\]

Where \( C_l \) is the Crack Initiation Threshold (typically 35-50% of UCS) obtained through laboratory testing as discussed presently in this paper. Back analysis of observed tunnel breakout [4,5] indicates that this threshold only applies a low confinements. At higher confining stress the strength envelope makes a transition up to the envelope defined by the yield or Crack Damage Threshold, \( CD \), obtained from testing. Uniaxial (unconfined) \( CD \) is typically 70-90% of UCS.

The envelope for insitu damage of massive rocks is limited in tension by the true tensile strength of the rock, \( T \). The full spalling envelope and the zonal significance is illustrated in Figure 3. Below the Lower bound field strength envelope (CI) no damage occurs within the rock. Above this envelope, micro-cracks initiate at the grain scale. Above CI but at high confinements these initiating micro-cracks quickly stabilize as they propagate away from the nucleation site. Upper bound strength (CD envelope) is controlled by shear fractures formed by microcrack coalescence. Acoustic emissions can occur in this range above the CI treshold and even minor joint slip events as small asperities locking up structure become weakened. At low confinement, such as that near an excavation wall, rock that is stressed above CI will incur spalling damage as new extension cracks and old cracks are allowed to propagate in an unstable fashion. In situ strength in this spalling zone falls significantly short of that predicted by lab testing.
Before applying this fundamental model of rock strength and behaviour it is necessary to establish a number of stress limits in a consistent and standardized fashion. These limits include the conventional strength index UCS (unconfined compressive strength), CD (critical crack damage or shear yield strength) and CI (crack initiation).

2 Uniaxial Compressive Strength, UCS

Uniaxial Compressive Strength, UCS, is defined as the ultimate breaking strength of the rock core. This is simply expressed as the maximum load at failure divided by the cross sectional area of a cylindrical sample. The ISRM Suggested Methods [7] recommend a standard diameter of 54mm and a length of between 2.5 and 3 times the diameter (12.5-15cm). The aspect ratio recommendation has been shown by Mogi [8] to ensure kinematic freedom for the development of a shear fracture beyond a ratio of 2.5. Mogi showed that UCS increases dramatically for aspect ratios greater than 2.5 while staying relatively constant beyond a ratio of 3.

It has become common practice to scale the UCS according to sample size. This practice is based on a relationship proposed by Hoek and Brown:

\[
\frac{UCS}{UCS_{D50}} = \left( \frac{50}{D} \right)^{0.18} \quad \text{or} \quad UCS_{D50} = 0.5 \cdot UCS \cdot D^{0.18}
\]

where D is in mm and UCS is in MPa. Note that Hoek and Brown (Figure 4a) recommended 50mm as a standard sample while 54mm as become the de facto standard as it represents the core diameter recovered in "N" size core. The relationship above is useful when upsampling UCS obtained from larger core. It is highly problematic, however, when used to downscale UCS from smaller core. Data from Hawkins (Figure 4b) shows that for a number of rock types (sedimentary in this case) the relationship proposed by Hoek and Brown does not hold true. This is likely due to the difficulties in providing perfect axial loading for a smaller core and the increased influence of single grain rupture in small sample. Figure 4c) and other studies by Martin [3] suggest that the scale effect is in fact a reduction to an asymptotic strength (about 80% of UCS) representing true
yield in compression. This would be consistent with other materials such as steel or concrete in which ultimate strength is preceded by a behavioural transition (non-linearity) indicative of yield.

In short, it is not recommended to apply any compensation for core smaller than 40mm as the scatter and uncertainties in the material and the testing equipment outweigh the statistical significance of scaling relationships. Smaller core should only be used for fine grained samples.

In addition, UCS should not be downgraded according to these relationships to account for tunnel scale strength. UCS should be viewed as a standard index strength and such downgrading will result in highly conservative analyses. Use of the Crack Damage strength, CD, to be described presently, is recommended for estimation of an upper bound for field strength. The asymptotic decay of strength for larger samples, illustrated in Figure 4, is simply the transition between a higher geometry controlled and dilation-dominated failure strength (UCS) to the true yield strength (CD). At the grain scale, there is more kinematic freedom for overall failure in a large sample, once local yielding takes place.

3 Crack Damage Threshold, CD

Like most materials, rock undergoes a transition between load-parallel elastic response and non-linear behaviour. Historically, this yield strength has not been used in rock mechanics and rock engineering. This threshold, however, marks the last truly material-specific strength threshold. Failure, beyond this stress level in uniaxial loading, is controlled by sample scale, loading rate and machine configuration resulting in the large typical scatter in UCS results. This threshold has had numerous symbols and acronyms in previous iterations. Martin [3] had previously defined this threshold as \( \sigma_{cd} \), for example. The new ISRM Commission on Rock Spall Prediction has established CD as the appropriate and standard acronym.

Martin [3] suggested using the point of reversal in volumetric strain (contraction to expansion):

\[
\varepsilon_{VOL} = \varepsilon_{Axial} + 2\varepsilon_{Lateral} \quad \text{(Note: lateral strain = circumferential strain)}
\]

Diederichs [6,11] showed that this volumetric strain reversal is appropriate for uniaxial compression but overpredicts CD for confined loading and suggested the point of axial strain non-linearity as the universal indicator of CD. Under uniaxial conditions, both volumetric strain
reversal and axial strain non-linearity are coincident. Both approaches are shown in Figure 5b. This would also correspond to a significant decrease in the incremental tangential modulus, $E_T$, measured over a moving but constant stress-strain increment as loading increases.

The crack damage threshold is also apparent from acoustic monitoring recorded during sample testing. The total hit count dramatically increases at CD. This is preceded by a constant rate increase during the preceding test phase (normally reflected in a straight line response in a log(AE count) vs log(Axial Stress) plot as shown in Figure 5a).

![Figure 5: Damage detection using a) acoustic emissions [11]; b) strain measurements [12].](image)

### 4 Crack Initiation Threshold, CI

Perhaps the most important threshold for design predictions, CI represents the stress level at which grain scale cracks begin to nucleate within the sample. Prior to this point, there is no new damage induced and the long term strength of the rock remains unchanged. After new cracks begin to nucleate, they can propagate under low confinement conditions. This process may be time, temperature and humidity dependant. Eventually rocks stressed above this threshold at low confining pressures will eventually incur spalling damage.

This threshold has also had a number of conflicting symbolic representations including $\sigma_{ci}$ [3] and UCS$^*$ [1]. The ISRM Commission on Rock Spall Prediction has established CI as the standard nomenclature. This threshold can be detected using acoustic emission monitoring of lab samples as demonstrated in Figure 5a. Note that sensor gain is critical here as there may be noise and low amplitude random cracking prior to the CI threshold. CI is the first point at which a systematic increase in crack emissions follows an increase in applied stress. The recorded AE counts should involve a at least half of the sensors in the sample (a minimum of 4 sensors spaced about the cylinder at least 1/3 of the sample length away from the platens is recommended [12]).

In strain based monitoring, CI is the first point of lateral strain non-linearity. Care should be taken as crack closure strain anomalies may overlap damage initiation strain readings, especially for damaged samples. A robust approach to CI detection is based on a reversal (or onset of expansion) of the “crack damage strain”:

$$\varepsilon_{CV} = \varepsilon_{VOL} - \varepsilon_{EV}$$

[4]
Where elastic volumetric strain, $\varepsilon_{VOL}$, is obtained from equation 3 assuming elastic response based on Young’s Modulus, $E$, and Poisson’s Ratio, $\nu$.

CI can also be detected as an increase in the “Instantaneous Poisson’s Ratio”:

$$\nu_I = \frac{\Delta \varepsilon_{Lateral}}{\Delta \varepsilon_{Axial}}$$  \hspace{1cm} [5]

monitored over a constant interval throughout the strain history. This value will settle in to the elastic constant after crack closure and will begin to increase again after CI.

4 Elastic Constants, $E$ and $\nu$

The ISRM Suggested Methods [7] recommends that Young’s Modulus, $E$, and Poisson’s ratio, $\nu$, be calculated as tangential values at single point on the stress-strain curves at 50% of the ultimate load. Crack Initiation, CI, typically occurs at 40-45% of UCS for most rocks. It is therefore recommended that the elastic constants be taken at 35-40% of UCS although overlap with crack closure strain non-linearities (Figure 5b) should be avoided.

5 Empirical Spall Prediction

Once CI has been determined, it is possible to predict the onset of spalling from Figure 6.

![Figure 6: Empirical prediction of spall related overbreak depth.](image)

Where CI is known, a very preliminary approximation of overbreak extent measured as a radial distance from the tunnel center, $r$, is given (for a tunnel of planned radius, $a$):

$$\frac{r}{a} = 0.5 \left( \frac{\sigma_{max}}{CI} + 1 \right) \hspace{1cm} for \hspace{0.5cm} \sigma_{max} > CI$$  \hspace{1cm} [6]

In addition, the locations of the maximum overbreak can easily be predicted as those tunnel boundary surfaces parallel to the direction of maximum insitu stress.
6 Numerical Analysis for Spall Prediction

These are a number of complex discrete fracture models under development for applications in spall prediction [14,15 and others]. At present, however, none are in common usage and most require significant verification and improvements in accessibility for the front line engineer. While these developments show great promise for the future, it is possible to utilize industry standard numerical programs to predict the onset and extent of spalling.

The complex yield curve in Figure 3, incorporating both the initiation limit at low confinements as well as the transition to shear behaviour at high confinements, can be incorporated in continuum models through an initial yield function corresponding to crack initiation and a subsequent "residual" threshold reflecting the transition to shear or the limit for spalling. The initial yield function that has low confinement sensitivity (low friction) and a cohesive term that provides an unconfined stress limit equivalent to CI. A typical friction angle for this threshold corresponds to typical mineral cleavage surface values (friction coefficient of 0.3-0.4). It is also important to provide for a transition or a cutoff at the true tensile strength. The "residual" envelope has a low cohesive intercept and a higher frictional term reflecting the steepness of the transitional spalling limit in Figure 3. This two-envelope approach can be incorporated through the appropriate specification of the "peak" and "residual" parameters [1] as in PHASE2 (Fig 7a) or as strain dependent conversion of cohesion and friction parameters [as in 5] in FLAC or UDEC (Fig 7b). Both approaches are illustrated in Fig.7 with results presented in Figure 8 for a typical tunnel example. The derivation of generalize Hoek-Brown parameters is highlighted in Fig 7a with equivalent Mohr-Colomb parameters used in Fig. 7b. Using the latter approach it is also possible to represent the upper high confinement limit (CD) in Figure 3 (not shown in Fig. 7).

![Figure 7: Modelling damage initiation and the spalling limit: a) using “peak and residual” parameters (PHASE2) and b) using “strain-weakening” model (FLAC) (modified after [1]).](image)

The following general methodology is suggested for Generalized Hoek-Brown [16] parameters:

For initial (peak) threshold:

\[ a = 0.25 \quad s = (CI/UCS)^{1/a} \quad m = s(UCS/T) \]

For residual threshold:

\[ a = 0.75 \quad s = 0.0001 \quad m = 7 \text{ to } 10 \]

Mohr-Coulomb parameters can then be determined by inspection or best fit. The plastic strain required to affect the change in cohesion and friction in Figure 7b (FLAC strain-weakening model) should be approximately equal to 2\(CI/E\). Note that in this model the response at low confining stress is brittle while at higher confining stress the response is strain hardening after initial damage. Damage is indicated by model "yield" while breakout may be smaller, indicated by a high shear strain concentration as illustrated in the example tunnel analysis in Fig. 8. Spall extent is accurately modelled although the displacements in continuum modelling require validation.
Figure 8: Comparison of spall damage and overbreak prediction for a large TBM tunnel. Contour lines indicate significant (order of magnitude) increase in shear strain beyond elastic background.

References


