Review: Geotechnical Design in Highly Stressed Brittle Rock

by Erik Eberhardt, Mar. 2016

Brittle failure, both in the form of major spalling and potentially strain bursting, can be expected where massive to blocky rock mass conditions (Fig. 1, Cai et al. 2004) coincide with exceptionally high in-situ stresses. (Note that vertical in-situ stresses are generally projected from tunnel overburdens, but the potential exists for horizontal in-situ stresses to be even higher due to locked-in tectonic stresses.)

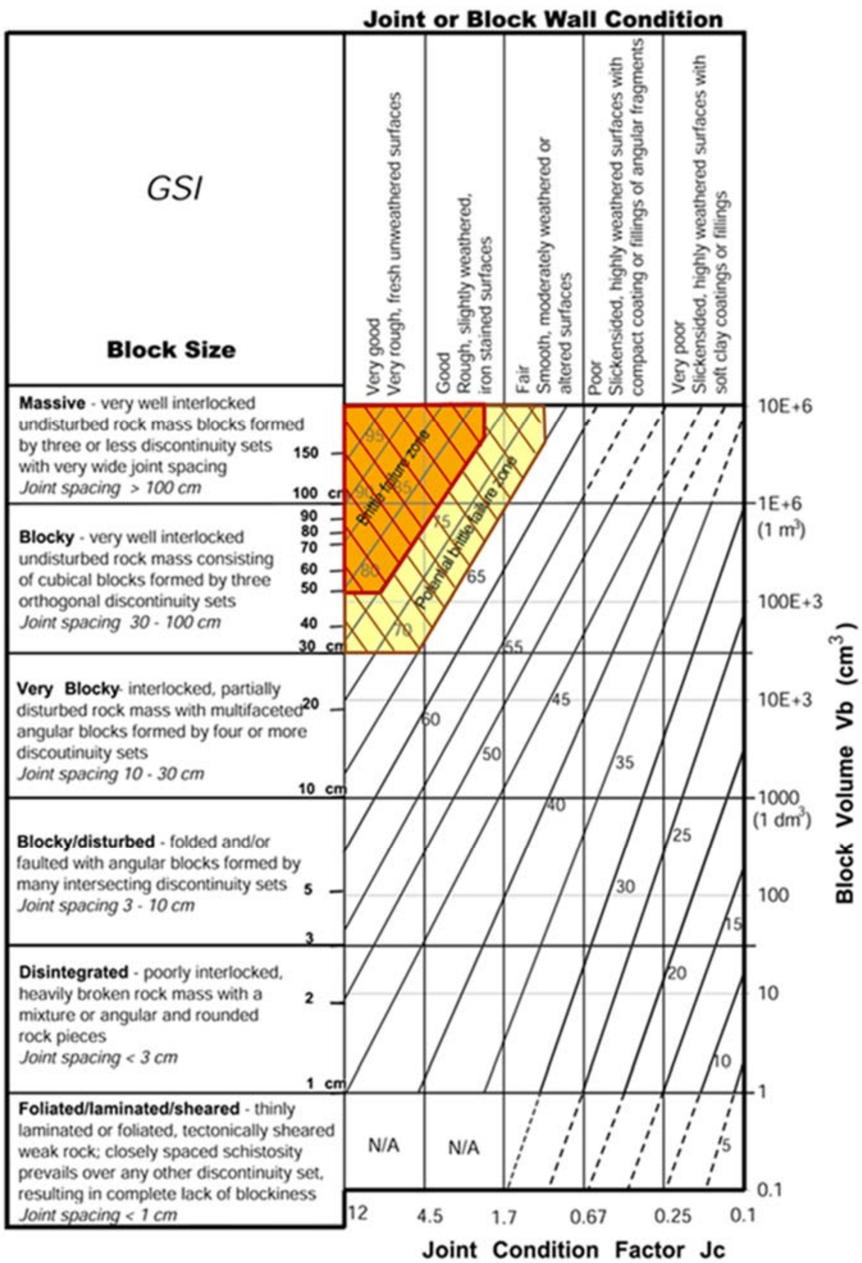

Figure 1. Geological Strength Index (GSI) lookup chart after Marinos & Hoek 2000 with suggested ranges for potential brittle failure (hatched areas) specified by Cai et al. (2004) and Diederichs (2007).

Tunnelling in brittle rock under high in-situ stress conditions poses a number of unique challenges. The rock becomes less forgiving than that encountered under lower stresses, and the application of conventional design principles (e.g. stand-up time) and constitutive laws (e.g. Mohr-Coulomb elasto-plastic) can seriously mislead designers. Stress-induced fractures that develop subsequent to tunnel excavation have in the past been misinterpreted as natural joints, resulting in difficulties in developing effective ground support strategies.

Recent experiences, however, in linking practices in deep mining with high overburden tunnels (e.g. Gotthard Base Tunnel in Switzerland) have led to the development of a framework for designing deep tunnels in brittle rock (e.g. Martin et al. 1999, Kaiser 2006, Diederichs 2007). Underpinning this framework is the understanding that stress driven failure involves a tensile fracturing process in which the fractures form parallel to the excavation boundary (i.e. major principal stress σ_1). Summarizing Kaiser (2006), when $\sigma_{max}/\sigma_{ci} > 0.8$ intact rock fracturing starts to overlap structurally-controlled failure processes, where σ_{max} is the maximum tangential stress at the boundary of a circular opening in elastic ground ($\sigma_{max} = 3\sigma_1\text{-}\sigma_3$), and σ_{ci} is the laboratory uniaxial compressive strength of the intact rock. From a constructability perspective, while stress-driven fracturing may not add much load to the support system, it may increase overbreak. In addition, strain-bursting potential in massive to blocky rock may lead to elevated risk levels (worker safety concerns). At high stresses, typically $\sigma_{max}/\sigma_{ci} > 1.15$, in strong hard rock, deep-seated stress-driven failure dominates. The rock mass surrounding the tunnel becomes heavily fractured, tends to increase in volume (due to dilation), and deformation control measures are required to contend with rock mass bulking.

Understanding that brittle failure involves a tensile fracturing process, Diederichs (2007) proposed a criterion for susceptibility to brittle spalling (as opposed to plastic shear) based on the Hoek-Brown parameter m_i. Strain burst potential is dictated by σ_c as stronger rocks allow for a larger build-up of strain energy (Fig. 2). It should be emphasized that this criterion does not assess or predict whether spalling or strain bursting will occur, as the stress conditions to be encountered aren't accounted for. It simply relates the rheology of the rock based on its m_i value to the potential for brittle behaviour versus a more ductile shear behaviour.

Stress-induced spalling can lead to insufficient stand-up time, excessive overbreak, loosening of the rock, difficult rock containment issues and requirement for additional grouting behind the tunnel lining. To estimate the depth of brittle failure around tunnels in high stress environments, Martin et al. (1999) proposed an empirical criterion, where:

1) The initiation of stress-induced failure occurs when the ratio of the maximum tangential boundary stress (σ_{max}) to the laboratory uniaxial compressive strength (σ_{ci}) exceeds 0.4 (see Fig. 3). In other words, the stress-induced failure process begins at stress levels well below the rock's unconfined compressive strength.

2) When this condition occurs, the depth of stress-induced brittle failure around a tunnel in massive to moderately fractured rock can be estimated by using an elastic stress analysis and a Hoek-Brown failure criterion with the associated parameters m=0 and s=0.11 (Fig. 3). The fundamental assumption here is that the stress-controlled failure process around the tunnel is dominated by cohesion loss. Hence the m_b parameter, which can be equated to frictional strength, is set to zero. It should be emphasized that this treatment (i.e. m=0) differs from that which would be used for an elasto-plastic yielding failure mechanism where the frictional strength component mobilizes and dominates the behaviour of the rock mass, requiring the m value to be set to a typical value for the rock type in question.

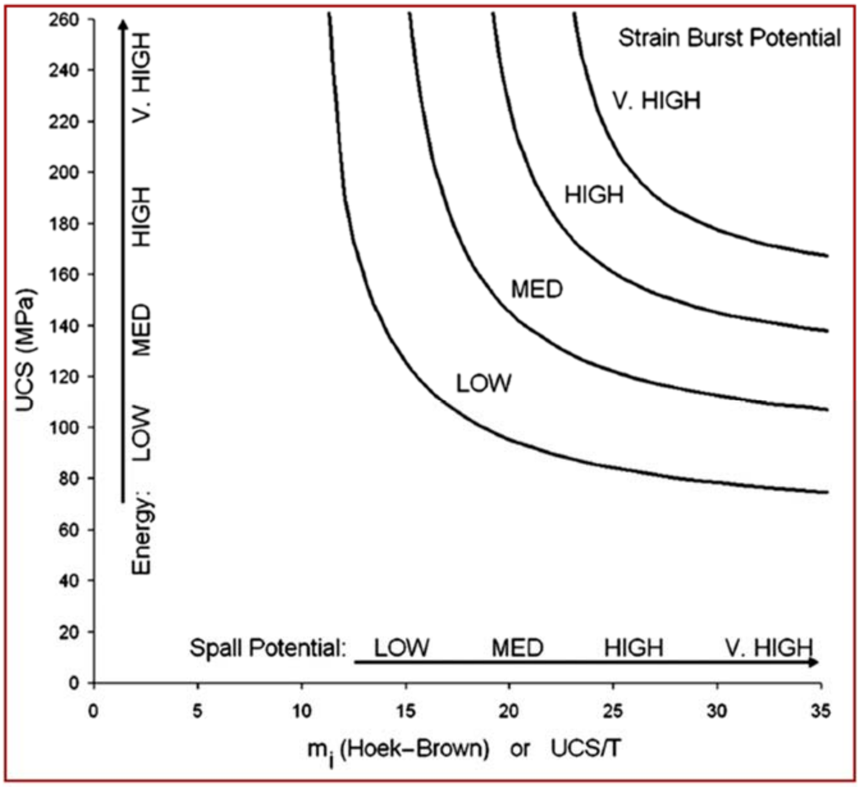

Figure 2. Potential for spalling and strain bursting after Diederichs (2007).

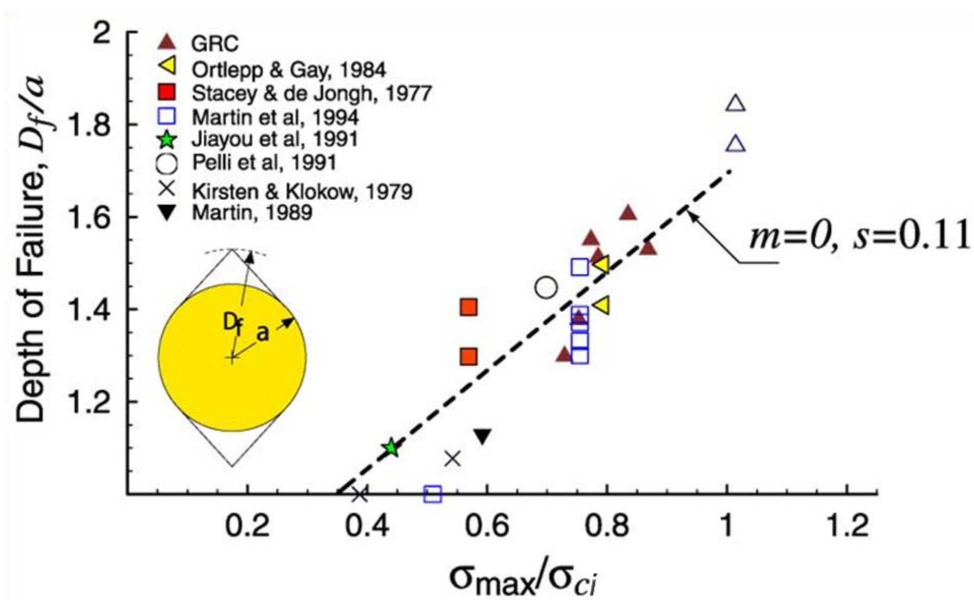

Figure 3. Empirical prediction of depth of stress-induced failure using the Hoek–Brown brittle parameters m_b=0 and s=0.11 (after Martin et al. 1999).

These findings and the empirical relationship suggested by Martin et al. (1999) have since been repeated and confirmed in other studies on tunnel stability in highly stressed ground (e.g. Kaiser et al. 2000, Diederichs et al. 2004, etc.). The data suggests a linear relationship for the depth of failure, given as:

$$\frac{R_f}{a} = 0.49(\pm 0.1) + 1.25\frac{\sigma_{max}}{\sigma_{ci}} \tag{1}$$

where R_f is the depth of failure, *a* is the tunnel radius, σ_{max} is the maximum tangential boundary stress, which can be approximated as $\sigma_{max} = 3\sigma_1\text{-}\sigma_3$, and σ_{ci} is the laboratory uniaxial compressive strength of the intact rock.

A second, and perhaps more rigorous, methodology for predicting depth of spalling overbreak for deep tunnels in blocky to massive rock (GSI > 65) is provided by Diederichs (2007). The procedure introduces a bi-linear failure criterion that accounts for the different stress thresholds under which brittle fractures initiate and propagate during spalling (Fig. 4). The criterion captures the dependence of fracture propagation on confinement for materials that are prone to spalling, and can be incorporated into a non-elastic numerical model using modified Hoek-Brown parameters. The inclusion of the role the confining stress plays is an important consideration as the spalling process will self-stabilize at some distance into the rock mass due to confinement. This is not fully accounted for in the empirical relationship by Martin et al. (1999).

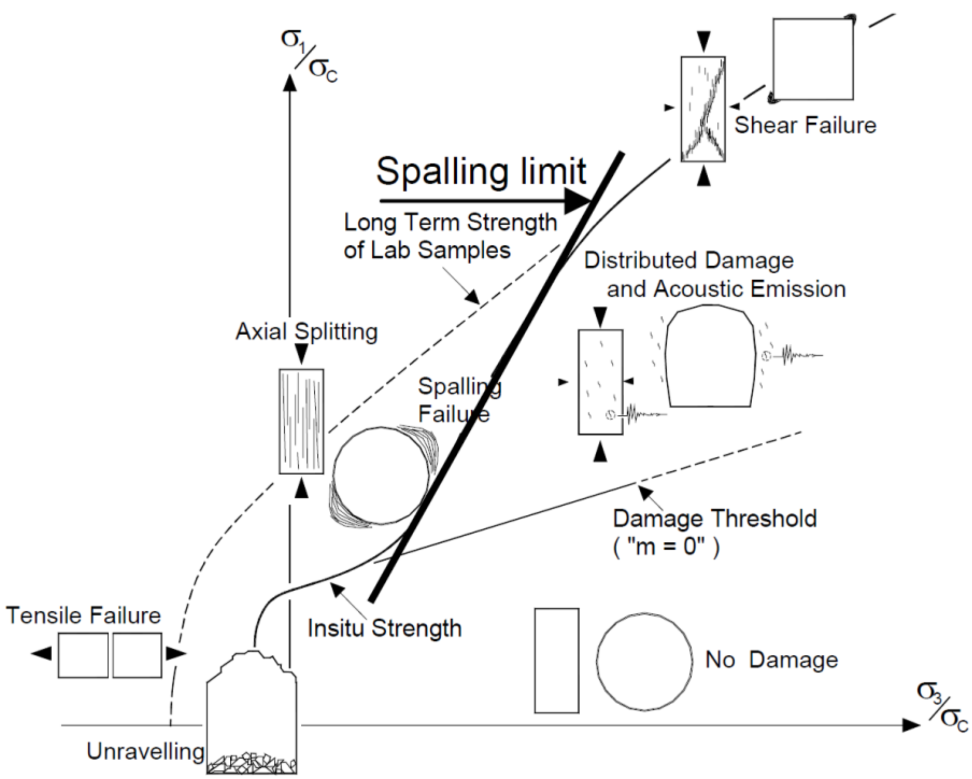

Figure 4. Composite failure envelope for modelling damage initiation and spalling limit (after Diederichs 2007).

The general methodology suggested by Diederichs (2007) involves:

1) For the lower bound threshold for spalling initiation, determine a damage initiation threshold σ_{di}. Although this value is best established using acoustic emission data from laboratory testing (Eberhardt et al. 1998), a value of $0.4\sigma_{ci}$ is generally suitable for crystalline rock and coincides with the ratio for spalling initiation on the tunnel boundary provided by Eqn. (1), i.e. where R_f/a = 1.
2) For systematic damage, determine σ_{sd}. Again, in the absence of acoustic emission and laboratory test data, previous experience with crystalline rock suggests a value of $0.6\sigma_{ci}$ (Diederichs et al. 2004) should be suitable for scoping calculations.
3) Set the Hoek-Brown exponent to a_{di} = 0.2 to determine the maximum depth of damage (worst possible case), and a_{sd} = 0.25 to provide a more realistic prediction of actual visible spalling (worst probable case).
4) Obtain a reliable measurement (or estimate) of tensile strength.
5) For damage initiation and systematic damage, calculate the modified s and m_b values from:

$$s_{di} = \left(\frac{\sigma_{di}}{\sigma_{ci}}\right)^{1/a_{di}} \quad \text{and} \quad s_{sd} = \left(\frac{\sigma_{sd}}{\sigma_{ci}}\right)^{1/a_{sd}} \tag{2}$$

$$m_{di} = s_{di}\left(\frac{\sigma_{ci}}{|\sigma_t|}\right) \quad \text{and} \quad m_{sd} = s_{sd}\left(\frac{\sigma_{ci}}{|\sigma_t|}\right) \tag{3}$$

These values can then applied within an elasto-plastic finite element program (e.g. Phase2 by Rocscience 2009) using a modified Hoek-Brown constitutive model where the damage initiation values are used for peak strength and the systematic damage values are used for the residual strength.

Other key points of consideration with brittle rock failure during tunnelling include:

- By observing the failure processes, it is possible to make predictions regarding the ground response to tunnelling and to anticipate situations that may lead to construction difficulties. Recognize the actual conditions early and utilize these throughout the project execution. Respect the unexpected by ensuring sufficient flexibility in design and construction techniques. As emphasized by Kaiser (2006), the behaviour of highly stressed, brittle failing rock is often not fully anticipated and/or integrated into tunnel designs, leading to costly mistakes.
- Construction difficulties resulting from stress-induced fracturing and spalling of the rock include drilling and grouting bolts in fractured rock, setting ribs on irregular surfaces due to overbreak, floor heave due to bulking, and slowing the construction progress. The level of construction difficulty increases rapidly, in a non-linear manner with height of overburden, and rock support systems that can effectively control the related unravelling process must be adopted.
- The role of rock support for stress-driven failure problems is not to prevent it but to control it, with different support elements (bolts, mesh, shotcrete, etc.) each fulfilling a different function. These functions are to retain broken rock near the excavation, and to control the bulking process related to stress-induced fracturing, fragmentation and dilation.
- While the depth of spalling is essentially independent of the support pressure, dilation of the broken rock is very sensitive to confinement. Anticipated bulking is best managed with a tight retaining system (e.g. early shotcrete).
- The required bolt length is equal to the maximum depth of failure (D_f above) plus a safe anchor length. For split set bolts, the pick-up length (D_f) must be deducted from the bolt length to assess

the holding capacity of the remaining anchor length. Other failure modes such as structurally controlled failures where overburdens and in-situ stresses are lower must be assessed separately.

- Salamon (1984) shows that the sudden release of energy in the form of strain bursting is largest when rock is excavated in a single stage but reduced when excavated in smaller sequenced steps. Although burst-resistant support systems help to mitigate the hazard, constructive means should be used to induce incremental spalling through sequential excavation and/or destress blasting techniques when strain bursting is anticipated or encountered.
- Similar to that observed around the tunnel boundary, stress-induced fracturing and spalling also occurs at the tunnel face resulting in an effective strength decrease and unstable tunnel face. These processes can be exacerbated by significant stress rotations as the tunnel face is advanced (Eberhardt 2001). Kaiser (2006) recommends that highly stressed tunnel faces should be shaped in a convex shape to remove potentially unstable rock that poses a safety hazard to workers near the tunnel face.

References

Cai, M., Kaiser, P.K., Uno, H., Tasaka, Y. & Minami, M. (2004). Estimation of rock mass deformation modulus and strength of jointed hard rock masses using the GSI system. *International Journal of Rock Mechanics & Mining Sciences* 41(1): 3-19.

Diederichs, M.S. (2007). Mechanistic interpretation and practical application of damage and spalling prediction criteria for deep tunnelling. *Canadian Geotechnical Journal* 44(9): 1082-1116.

Diederichs, M.S., Kaiser, P.K. & Eberhardt E. (2004). Damage initiation and propagation in hard rock tunnelling and the influence of near-face stress rotation. *International Journal of Rock Mechanics and Mining Sciences* 41(5): 785-812.

Eberhardt, E. (2001). Numerical modeling of three-dimensional stress rotation ahead of an advancing tunnel face. *International Journal of Rock Mechanics and Mining Sciences*: 38(4), 499-518.

Eberhardt, E., Stead, D., Stimpson, B. & Read, R.S. (1998). Identifying crack initiation and propagation thresholds in brittle rock. *Canadian Geotechnical Journal* 35(2): 222-233.

Kaiser, P.K. (2006). Rock mechanics considerations for construction of deep tunnels in brittle rock. In Leung & Zhou (eds.), *Rock Mechanics in Underground Construction: 4th Asian Rock Mechanics Symposium, Singapore*. Singapore: World Scientific Publishing, pp. 47-58.

Kaiser, P.K., Diederichs, M.S., Martin, D., Sharpe, J. & Steiner, W. (2000). Underground works in hard rock tunnelling and mining. In *GeoEng2000, Melbourne*. Technomic Publishing Company: Lancaster, pp. 841-926.

Martin, C.D., Kaiser, P.K. & McCreath, D.R. (1999). Hoek-Brown parameters for predicting the depth of brittle failure around tunnels. *Canadian Geotechnical Journal* 36(1): 136-151.

Salamon, M.D.G. (1984). Energy considerations in Rock Mechanics: Fundamental results. *Journal of the Southern African Institute of Mining and Metallurgy* 84: 233-246.