Estimate of support and reinforcement cost increase associated to poor blasting practices in drifting

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ABSTRACT: Starting from realistic information, it is analyzed how poor blasting techniques produce an increase in the needs of support and reinforcement in drifting for mining purposes. A basic approach has been developed and applied, based on the fact that the lowering in the geomechanical quality of the rock mass can be quantified according to the D parameter as proposed by Hoek et al. (2002) and then, the support and reinforcement is calculated according to the convergence confinement technique. A direct estimate of the lowering of the quality of the rock mass, according to Q system can also be applied. The increase of support and reinforcement is quantified in economic terms, following Peruvian mining actual data. Results obtained indicate that an improvement in the quality of blasting can reduce support and reinforcement costs in mines, but this should be analyzed case by case.

1 INTRODUCTION

Drift advance rates are often not as good as expected. Quick drift development permits rapid access to ore bodies, which may highly improve their value. This tends to be critical for particular mining methods as block caving. Some mining companies are planning block caving because of its long-term low production cost. Some others afraid for a possible drop in metal prices prefer to use top-down approaches, such as sub-level caving.

A significant part of drift costs stems from support and reinforcement. Should any of this support approach or reach failure or should the drift face collapse, then the drift and support need to be rehabilitated. This often increases costs much more than the initial installation, due to loss of access, down-time of the area, potential for injuries and lost revenues. Some types of collapse, severe deformation and squeezing in mining drifts are shown in Figure 1.

As it can be observed when applying empirical methods such as Barton's Q, when underground excavations are designed to last a limited time (some months to years) as it is usually the case of mining, the stability problems are expected more often than in civil engineering field, where tunnels are designed to last longer and to suffer very small strains.

Another important issue in some underground mines, where drift advance is performed by a contractor, and support and reinforcement are installed by mine personnel, is that whereas the main aim of the contractor is to advance very quickly, this may result in the application of non-careful blasting techniques which produce damage in the rock mass surrounding the drift and, ultimately, increase the cost of support and reinforcement.

One of the long recognised limitations of the rock mass classification is the effect of drifting practice and construction methods on the excavation wall rock quality as a dominant factor increasing excavation support demand. The idea of this study started from this back-draw of empirical methods. Peruvian metal underground mining costs are used to quantify the topic in two cases and to show that, eventually, care must be taken in the whole drifting process, especially in the drilling and blasting operations.

Figure 1. Drifts with stability problems of different types such as collapses and high deformation.

2 SOME COST ESTIMATES OF MINE DRIFTING IN PERUVIAN MINING

The development of a drift by means of drill and blast is a sequential operation in four steps: 1) drilling a round, 2) loading and blasting the round, 3) mucking the broken rock and scaling if needed and 4) support and reinforce the round. The first three steps can be joined in a single stage called drift advance.

We have recovered actual cost estimates of drift advance, support and reinforcement in the metallic Peruvian mining. The data correspond to six different mines and to 3 to 4 m diameter drifts excavated in rock masses from very bad to good quality. In two of the mines the costs were calculated according to rock mass quality. The average, standard deviation and percentage data recovered are shown in table 1, where the detailed costs are estimated.

Table 1. Average, standard deviation and percentage drifting costs from six Peruvian mines.

	Mean	St. Desv.	Mean
	$/m.	$/m.	%
Drilling	205	63	22
Blasting	84	18	9
Mucking	104	38	11
Drift advance	393	97	42
Support	437	235	47
Reinforcement	106	64	11
Support + Reinf.	543	253	58
TOTAL	935	272	100

It should be pointed out that since the drifts were excavated in different quality rock masses, the costs of support and reinforcement are particularly variable, as the high values of their standard deviation suggest. However, and in average terms it can be observed from table 1 that the average cost of support and reinforcement is 1.5 times that of drift advance, 3 times that of drilling and 6 times that of blasting. As a consequence and from these figures it does not seem logical to try to save a small amount in blasting (10 % of the total cost of drifting) if this can increase (sometimes a lot) the cost of support and reinforcement (60 % of the total cost of drifting).

Due to the cyclic nature of drill-and-blast, each activity time must be optimised to obtain an increase in advance rate. The drift advance varies according to the local conditions, equipment and construction practices. According to Suorineni et al. (2008), uncontrolled blasting can result in overbreak and increased mucking and support installation times.

In one of the mines the drifting costs have been obtained for decreasing rock mass quality and for increasing support demand. Results are shown in Figure 2, which can be representative in general terms of the mines in the region. Some of the solutions budgeted in Figure 2 are shown in Figure 3.

SUPPORT & REINFORCEMENT	
	H4x4
1 m advance:	$515,0/m
Shotcrete 2":	$280,0/m
	$795,0/m
1 m advance:	$510,0/m
Mesh+Bolting (st.):	$320,0/m
	$830,0/m
1 m advance:	$505,0/m
Shotcrete 2"+Bolting (st.):	$450,0/m
	$955,0/m
1 m advance:	$500,0/m
Shotcrete 2"+Bolting (st.)+Mesh:	$590,0/m
	$1.090,0/m
1 m advance:	$495,0/m
Shotcrete 4"+Bolting (st.)+Mesh:	$880,0/m
	$1.375,0/m
1 m advance:	$490,0/m
Shotcrete 2"+Steel arch (per m.)+Layer:	$970,0/m
	$1.460,0/m
1 m advance:	$485,0/m
Shotcrete 2"+Steel arch (per m.)+Layer+concr. wall:	$1.050,0/m
	$1.535,0/m

Figure 2. Drifting costs from one of the mines for decreasing rock mass quality and increasing support and reinforcement demand. Bolting refers to one bolt per 4 m^2.

Figure 3. Different support and reinforcement solution for drifts in Peruvian mines.

According to these data, it can be stated that whereas the cost of advance is higher and sometimes even double than that of support and reinforcement in average to good quality rock masses, the opposite occurs for drifts excavated in very bad to bad quality rock masses where the advance cost remains sensibly equal, but support and reinforcement costs can be double or even more than those of the advance.

As derived also from this drifting cost structure it is important to highlight that savings in drilling and blasting (20 % of the average cost) are useless if they produce increasing needs of support and reinforcement (60 % of average cost), not to mention the case when collapses or unbearable deformations occur (Figure 1).

3 CONSIDERATION AND QUANTIFICATION OF BLAST-INDUCED DAMAGE

The use of high breaking efficiency explosives in the last third of the XIXth century meant an increase on productivity as well as a larger damage to the rock mass. This was due to the strong explosive features of the recently appeared dynamites, which produced stresses on the drill-hole a hundred times more powerful than those occurred due to black powder. For various decades there was not a special awareness about the damage on the rock mass occurred due to the use of explosive (Figure 4); and it was accepted as a slight inconvenience inherent to the use of high fragmentation capacity explosives.

However, in the fifties some Scandinavian researchers (see for instance Langefors & Kihlström, 1987; Lundborg, Holmberg & Persson, 1980...), with a excellent engineering judgement and scope, realized that it was possible to control up to a certain extent the alteration induced by the explosive use on the rock; for in particular conditions, and specially in large open pit mines, where enormous bench blastings were performed, this would mean a considerable save in costs together with an improvement in safety conditions.

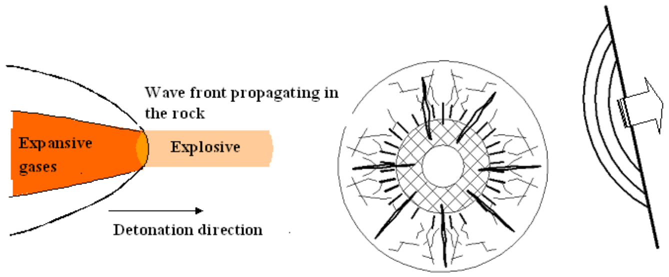

Figure 4. Damage on the rock mass occurred due to the use of explosive.

The consequences of blast damage on the rock mass surrounding the tunnel have been traditionally assessed in terms of overbreak, instead of quantifying the rock mass strength loss. As pointed out by Saiang & Nordlund (2009), even if the strength and stiffness are difficult to measure, they are also some of the most significant parameters in order to study excavation behaviour. Some researches have focused on correlating the extent of the damage zone with the explosive charge concentration.

Suorineni et al. (2008) show in their study that blast damage was meant to cause overbreak of up to 40 % in some drifts. According to the Q (Barton et al., 1974) support system chart, increasing excavation size by 40% can be equivalent to reducing the rock mass quality by one class. As a consequence the final output of reducing rock mass quality is an increase in support demand. These authors proposed a construction damage factor (C_f) (see Table 2) to account for this type of damage to correct the rock mass quality ($Q^*=Q^{C_f}$) in such a way that the appropriate temporal support can be used.

Table 2. Guidelines for selection of construction damage factor (C_f) according to Suorineni et al. (2008).

Description	C_f
Excellent perimeter blasting. HCF >75%. No overbreak	1
Controlled blasting. HCF:30-75%. Less than 10 % overbreak. Regular drift profile.	0.9
Good conventional blasting. HCF:10-30%. Fair overbreak (10-20%). Fair drift profile.	0.8
Poor conventional blasting. HCF<10%. Moderate overbreak (20-30 %). Irregular drift profile.	0.7
Very poor blasting. Major rockfalls. No HCF. High overbreak (>30 %). Irregular drift profile.	0.6

This approach only works for average or better quality rock masses and it is not valid when $Q < 1$.

This C_f can also be related with the so-called disturbance factor (D) proposed by Hoek et al. (2002) to correct the indicated blast excavation damage effect on rock mass behaviour. According to these authors a large number of factors can influence the degree of disturbance in the rock mass surrounding an excavation and it may never be possible to quantify them precisely. Nevertheless, and based on their experience they have established a set of guidelines for estimating this factor as it appears in Table 3.

Table 3. Suggested guidelines for selection of disturbance factor (D) to quantify damage factor to the rock mass according to Hoek et al. (2002).

Description	D
Excellent quality controlled rock blasting or excavation by TBM results in minimal disturbance to the confined rock mass surrounding the tunnel.	0
Mechanical or hand excavation in poor quality rock or where squeezing problems result in significant floor heave unless temporary invert is placed.	0.5
Very poor quality in a hard rock tunnel results in severe local damage, extending 2 to 3 m, in the surrounding rock mass.	0.8

In a first approach C_f =0.6 corresponds to D=0.8 and C_f =1 corresponds to D=0. This influence can be large in particular cases (specially for average class rock masses) as it is observed in what follows. All the indications given are guidelines and should be carefully used within the frame of wider design methodology as suggested by authors such as Starfield & Cundall (1988).

In their approach to the topic, Saiang & Nordlung (2009) used a numerical method to study tunnel behaviour, modelling a zone around the tunnel with decreased strength and stiffness properties. They used the classical elastic-perfectly plastic Mohr-Coulomb behaviour mode, due to the lack of data on post-failure behaviour characteristics. They focused hard rocks in the form of good and very good quality rock masses.

In our approach we model all the rock mass surrounding the tunnel as damaged rock. Obviously this is not true and only a zone around the tunnel is damaged. In our case we have to think that we usually deal with not very hard rock and not very good quality rock masses, where the damaged zone is wider than that occurring in the Scandinavian tunnels. So we put the stress in a more realistic modelling of post-failure behaviour following the guidelines by different authors (Hoek et al., 2002, Cai et al., 2007) and ourselves (Alejano et al., 2009).

The fact of considering all the material around the drift as damaged and not a limited extent zone, simplified this initial analysis and in this way, the Convergence Confinement Method (CCM) approach can be applied. This is obviously not accurate, but on the one hand, it is a conservative approach and on the other hand, we consider this as an initial approach to obtain preliminary results, which can be later extended to numerical modelling.

4 POST-FAILURE BEHAVIOUR MODELS

In regard to possible post-failure behavior models, Hoek & Brown (1997) suggested from their experience in the numerical analysis of a wide variety of actual cases, three basic types of post-failure behaviour (Figure 5).

1) Elastic perfectly plastic behaviour for rock masses with GSI < 25.
2) Elastic brittle behaviour, for rock masses with GSI > 75.
3) Strain softening behaviour, for rock masses with GSI from 25 to 75.

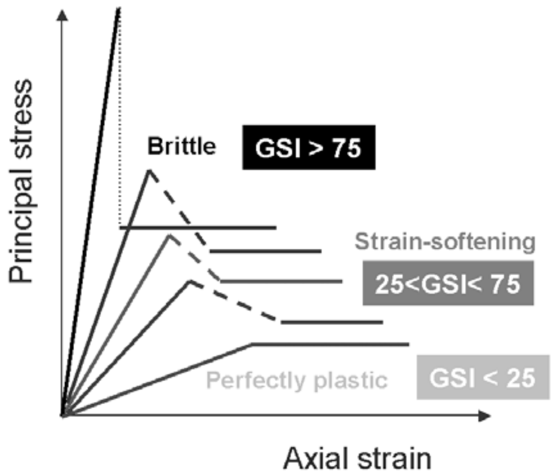

Figure 5. Stress-strain graphs representing expected post-failure behaviour of different quality (GSI) rock masses.

4.1. Strain-softening

Strain-softening behaviour is founded in the incremental theory of plasticity (Hill, 1950), developed in order to model the process of plastic deformation. According to this theory, a material is characterized by a failure criterion *f*, and a plastic potential, *g*.

One of the main features of the strain-softening behaviour model is that the failure criterion and the plastic potential do not only depend on the stress tensor σ_{ij}, but also on the so-called plastic or softening parameter η. Then, the behaviour model is plastic strain dependent.

The failure criterion is defined:

$$f(\sigma_r, \sigma_\theta, \eta) = 0 \tag{1}$$

The strain-softening behaviour is characterized by a gradual transition from a peak failure criterion to a residual one. This transition is governed by the softening parameter η. In this model, the transition is defined in such a way that the elastic regime exists while the softening parameter is null, the softening regime occurs whenever $0 < \eta < \eta^*$, and the residual state takes place when $\eta > \eta^*$, being defined η^* as the value of the softening parameter controlling the transition between the softening and residual stages. Figure 6 illustrates this type of strength-weakening behaviour for an unconfined compressive test.

The slope of the softening stage or drop modulus is denoted by *M*. If this drop modulus tends to infinity, the perfectly brittle behaviour appears, and if it tends to zero, the perfectly plastic behaviour is obtained. Perfectly brittle or elastic-brittle-plastic and the perfectly plastic behaviour models are limiting cases of the strain-softening model, which is considered as the most general case.

The constitutive equation of a strain-softening material can be obtained according to the incremental theory of plasticity. The plastic strain increments can be obtained from the plastic potential:

$$g(\sigma_r, \sigma_\theta, \eta) \tag{2}$$

according to:

$$\dot{\varepsilon}_r^p = \dot{\lambda}\frac{\partial g}{\partial \sigma_r} \text{ and } \dot{\varepsilon}_\theta^p = \dot{\lambda}\frac{\partial g}{\partial \sigma_\theta} \tag{3}$$

where $\dot{\lambda}$ is a plastic multiplier and it is unknown.

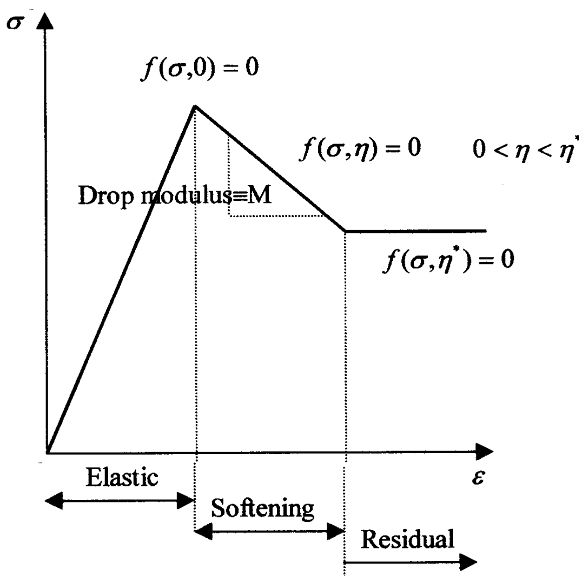

Figure 6. Stress-strain curve of an unconfined test performed on a sample of a strain-softening material.

Equation (3) is the constitutive equation of the plastic regime and it is usually called flow rule. If the plastic potential coincides with the failure criterion, then it is called associated flow rule, and if not, it is called non-associated flow rule.

Incremental plasticity involves the consideration of a fictitious ‘time’ variable. This ‘time’ variable called τ, controls the plastic strain increments in such a way that

$$\dot{\varepsilon}_r^p = \frac{\partial \varepsilon_r^p}{\partial \tau} \text{ and } \dot{\varepsilon}_\theta^p = \frac{\partial \varepsilon_\theta^p}{\partial \tau} \tag{4}$$

4.2. Mohr-Coulomb strain softening

If we consider a Mohr-Coulomb yield criterion:

$$f(\sigma_\theta, \sigma_r, \eta) = \sigma_\theta - K_\phi(\eta)\sigma_r - 2C(\eta)\sqrt{K_\phi(\eta)} \tag{5}$$

a plastic potential in the form:

$$g(\sigma_\theta, \sigma_r, \eta) = \sigma_\theta - K_\psi \sigma_r \tag{6}$$

where K_ψ is known as dilation coefficient or dilatancy relationship:

$$K_\psi = \frac{1+\sin\psi}{1-\sin\psi} \tag{7}$$

and piecewise lineal functions of plastic parameter for cohesion $c(\eta)$ and friction angle $\phi(\eta)$ being ϕ^p and c^p the peak parameters and ϕ^r and c^r the residual ones (Figure 7). The elastic regime is characterized by shear modulus G and Poisson’s ratio ν.

The plastic parameter taken is the plastic shear strain:

$$\eta = \varepsilon_1^p - \varepsilon_3^p = \varepsilon_\theta^p - \varepsilon_r^p = \gamma^p \tag{8}$$

4.3. Implementation of strain softening models

In previous works we have analysed rock masses with strain softening behaviour, and for those kind of rock masses, increasingly complex models have been proposed by Alejano et al. (2009):

1) Strain-softening with constant drop modulus and dilatancy (Figure 8).
2) Strain softening with a variable drop modulus and constant dilatancy.
3) Strain softening with a variable drop modulus and variable dilatancy.

But for the sake of simplicity and for this analysis we have chosen the simpler strain-softening model (constant drop modulus and constant dilatancy) since there is no big difference between the three models from an engineering point of view.

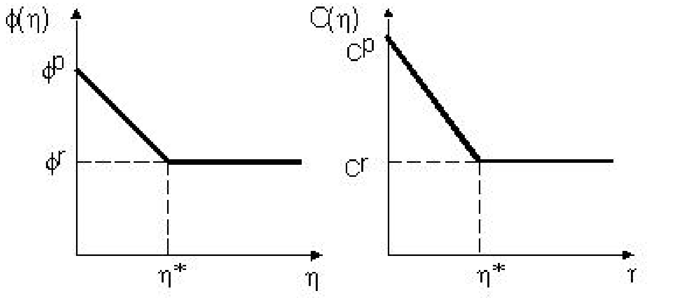

Figure 7. Cohesion and friction angle functions of plastic parameter.

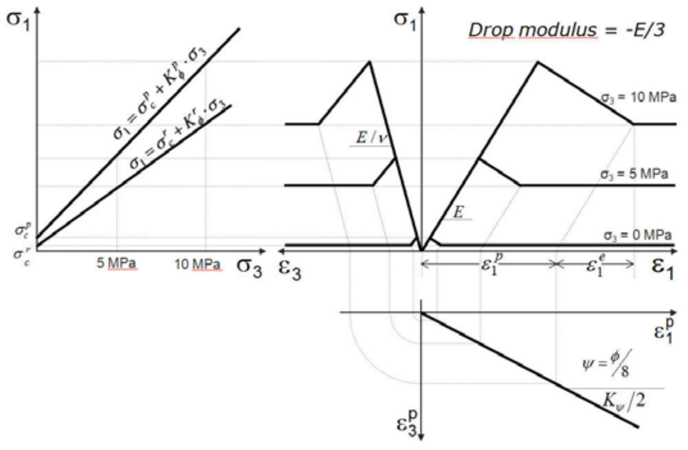

Figure 8. Strain softening model with a constant drop modulus and constant dilatancy for an average quality rock mass.

This model represents a first simple approach to modelling strength softening behaviour. In this step forward towards real rock mass behaviour in terms of model complexity, a strain softening approach makes the sudden stress drop associated with brittleness happens in a controlled manner.

5 THE CONVERGENCE CONFINEMENT METHOD

The convergence confinement method (CCM) is a 2D simplified approach for resolving the 3D rock-support interaction problem associated with installation of support near a tunnel face in underground excavations in rock, introduced in the thirties and later developed by different authors (see for instance Hoek & Brown, 1980; Brown et al. 1983; Brady & Brown, 1993 or Panet, 1995), has been comprehensive reviewed by Carranza-Torres & Fairhurst (2000).

The three basic components of the CCM consist of three graphs: the longitudinal deformation profile (LDP) -which relates tunnel deformation vs. distance to the tunnel face-, the support characteristic curve (SCC) –representing the stress-strain relationship of the support system- and the ground reaction or response curve (GRC) (Figure 9).

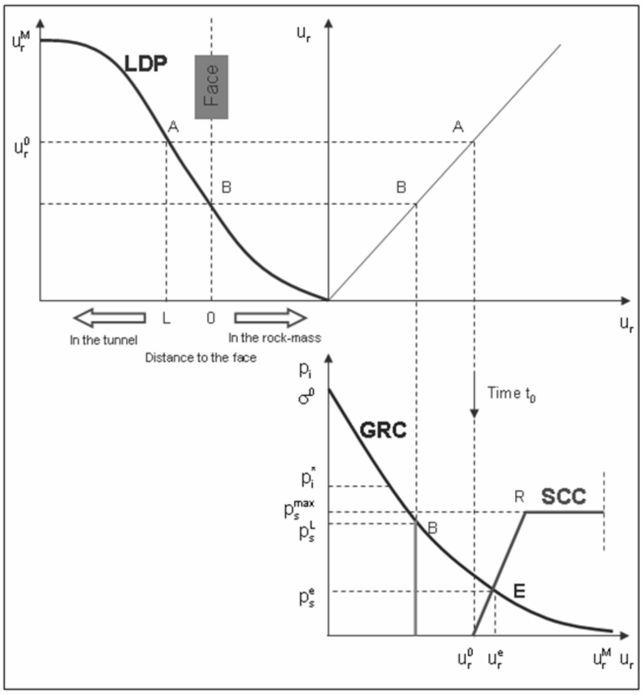

Figure 9. Main ingredients of the convergence-confinement method (CCM): the longitudinal deformation profile (LDP), the ground reaction curve (GRC) and the support characteristic curve (SCC). Based on Carranza-Torres & Fairhurst (2000).

Panet (1995), Chern et al. (1998) and Vlachopoulos & Diederichs (2009) have studied the LDP. The last mentioned reference provides a technique to obtain the LDP for different quality rock masses, which is used in this work. The way to obtain the SCC has been proposed for different types of support and reinforcement by Hoek & Brown (1980), and later other authors have dealt with this subject (Hoek, 1999; Carranza-Torres & Fairhurst, 2000 and Oreste, 2003). Finally the GRC describes the relationship between the decreasing of inner pressure and the increasing of radial displacement of tunnel wall, and it is evaluated on the basis of rock mass behaviour.

GRC often use elastic perfectly plastic models in practice (Carranza-Torres & Fairhurst, 2000; ROCSCIENCE, 2003). If failure is allowed to occur, these simple models do not represent well the actual stress-strain behaviour of the rock mass, except for bad quality rock masses. In all other cases, strain softening or brittle models are convenient to simulate ground behaviour correctly. Hoek and Brown (1997) rejected the elastic-perfectly plastic assumption as inappropriate for rock masses with average or high geotechnical quality (GSI>30). On the basis of this argument, the elastic perfectly-plastic approaches can only be applied to low quality rock masses (GSI<30). For average quality rock masses, a strain-softening behaviour model is needed. Other authors (Kaiser et al., 2000; Diederichs, 2003) have also indicated that for high quality rock masses (GSI>75) a type of brittle behaviour could be expected that does not fit the Hoek-Brown failure criterion approach.

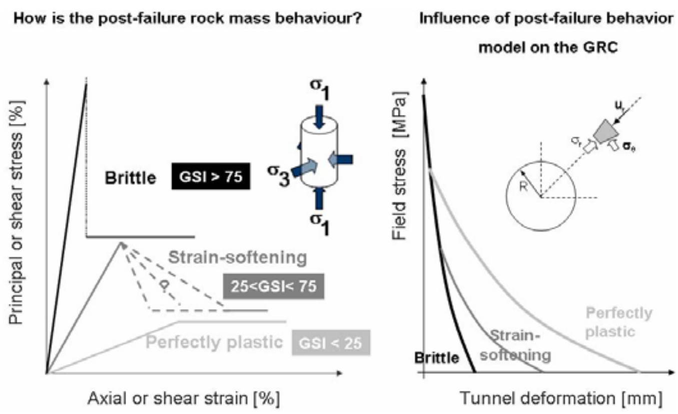

Figure 10. Different post-failure rock mass behaviour models with the corresponding ground reaction curve (GRC) for rock masses with different geological strength indices (GSI).

Strain-softening and brittle models are complex to characterize, since they need not only a peak failure criterion but also a residual one. They also need some other post-failure parameters. To estimate the residual criterion is at least not easy task. Some researchers are considering this important topic in order to deepen our knowledge on rock mass stress-strain behavior. In this way, Cai et al. (2007) have recently proposed to extend the *GSI* system for the estimation of a rock mass's residual strength. It is proposed to adjust the peak *GSI* to the residual GSI_r value based on the two major controlling factors in the *GSI* system. The proposed method was validated using in-situ block shear test data..

Even once peak and residual failure criteria have been defined, the behaviour of the rock still remains incompletely known. The Young's modulus and the Poisson's ratio, together with the drop modulus (usually confinement-stress-dependent) and the dilatancy angle need to be determined to fully represent the complete stress-strain behaviour necessary to be able to understand and model excavation behaviour.

The authors of this paper have been working for the last years in the development of techniques in order to obtain GRC for tunnels excavated in rock masses of different quality, as well as in the post failure dilatant behaviour of rock masses (Carranza-Torres et al., 2002; Alonso et al., 2003; Alejano & Alonso, 2005).

The SCC can be calculated depending of the type of support and reinforcement utilized according to Oreste (2003). The intersection of the GRC and SCC would yield the equilibrium point, which can in turn be used to estimate the safety factor of the support systems. This can be done, as Figure 11 illustrates, in terms of a stress factor of safety ($F.S._{\sigma}$), as proposed by Hoek (1999) or in terms of strain safety factors ($F.S._{\varepsilon}$), as proposed by Oreste (2003). We use both in our approach, being the safety criteria ($F.S._{\sigma}>1.1$), ($F.S._{\varepsilon}>1.3$), ($F.S._{\sigma} \times F.S._{\varepsilon}>1.5$), as included in table 5.

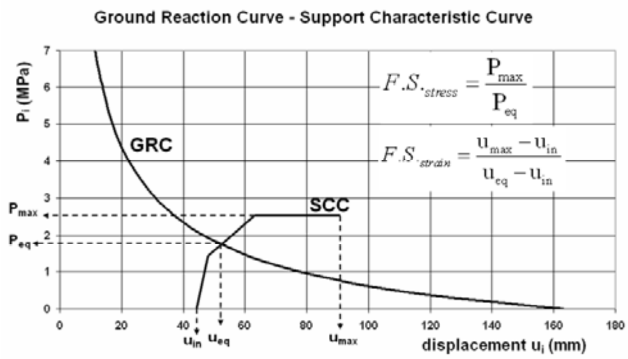

Figure 11. Definition of different safety factors for ground reaction curves (GRC) and support characteristic curves (SCC).

6 STUDY OF TWO DRIFTS

Two drifts have been selected in this study. One is 4 m diameter drift excavated in an average quality basaltic rock mass (*GSI* =55) and the other a 4 m diameter drift in a good quality rock mass (*GSI* =65).

6.1 *A drift in a basaltic average quality rock mass*

A mine drift 4 m x 4 m is excavated 400 m deep, in a rock mass with *GSI* = 55, σ_{ci} =23 MPa and m_i =10. An isotropic stress field σ_0 =12.07 MPa, as required for the standard application of the CCM, is assumed.

With these initial values the elastic and peak stress failure values have been estimated according to RocLab for the cases of excellent quality blasting (*D*=0), low quality blasting (*D*=0.5) and very poor blasting (*D*=0.8). The residual stress failure has been estimated according to Cai et al. (2007), considering reasonably that, after failure, the deterioration of the rock mass can be quantified as having a GSI_r =33. In this way, the peak and residual values of cohesion and friction are obtained, with the help of RocLab (ROCSCIENCE, 2002) and following the Hoek et al. (2002) approach, in this case with residual values. Dilatancy was obtained from the following equation proposed by Alejano et al. (2009) based on Hoek & Brown (1997):

$$\psi = \frac{5 \cdot GSI - 125}{1\,000} \cdot \phi_p \qquad \text{for } 75>\text{GSI}>25 \qquad (9)$$

The value of the plastic parameter marking the achievement of the residual strength values has been estimated according to Alejano et al. (2009). In this way the set of parameters needed to model the basaltic rock mass (*GSI* = 55) for the cases of *D*=0, *D*=0.5 and *D*=0.8 are shown in Table 4, and triaxial tests on rock mass specimens are presented in Figure 12.

For this differently damaged rock masses we have obtained the corresponding ground reaction curves following the technique by Alonso et al. (2003), which are presented in Figure 13. This result is a clear indication of the fact that the damage to the rock due to blasting, may produce a high increase in displacements. In this case the extent of the plastic zone or relation between the plastic radius and the tunnel radius play an important role in the differences of final deformation observed, The obtained plastic radii for the unsupported drift are included in Table 5 and illustrated in Figure 14.

Table 4. Geomechanical parameters of the basaltic rock mass for different damage levels.

	No damage	Little damage	High damage
	D = 0	*D* = 0.5	*D* = 0.8
γ (kN/m^3)	26.7	26.7	26.7
E (MPa)	3837.8	1901.1	1179.2
ν	0.25	0.25	0.25
c_{peak} (MPa)	1.183	0.96	0.781
ϕ_{peak} (°)	32.62	28.32	24.22
$c_{res.}$ (MPa)	0.801	0.582	0.417
$\phi_{res.}$ (°)	26.3	20.35	15.25
ψ (°)	4.9	4.2	3.6
η*	0.0049	0.0073	0.093

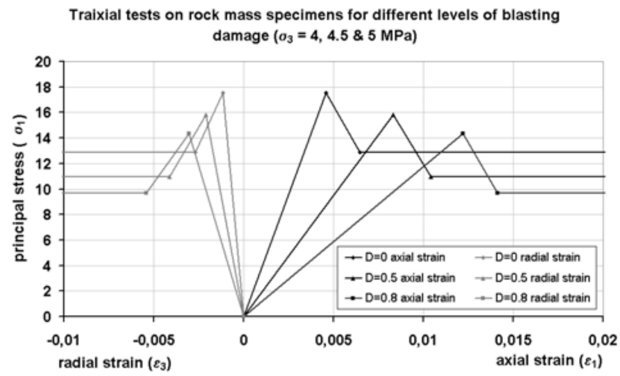

Figure 12. Stress-strain results for compressive triaxial tests on rock mass specimens.

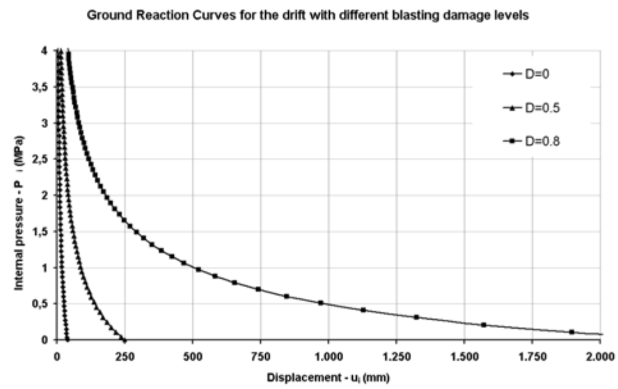

Figure 13. Ground reaction curves for tunnels excavated in the basaltic rock mass with different damage levels.

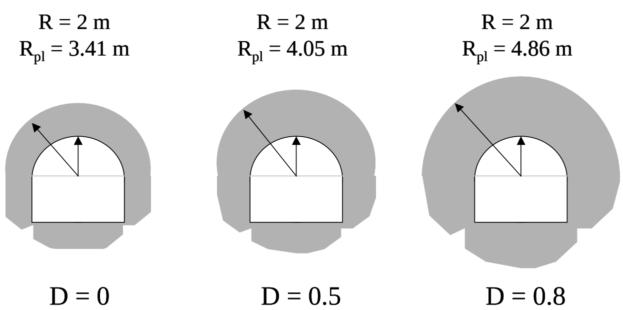

Figure 14. Plastic radii of the drift for different damage levels.

The standard application of support and reinforcement of 5 cm of shotcrete and rock bolting spaced 2 m x 2m is considered for the case of the undamaged rock mass. In this way the GRC together with SCC (following Oreste, 2003) are obtained and presented in Figure 15. For the little damage case (*D*=0.5), this support system produced stress factors of safety just one, and therefore, it was decided to propose an increase in the shotcrete width to achieve 7.5 cm (Figure 16).

The corresponding GRC and SCC presented in Figures 15 and 16 mark that for the case *D*=0, 5 cm of shotcrete and standard 2 m long bolting (10 tons, spaced 2 m x 2 m) was sufficient to ensure stability. For *D*=0.5, to ensure stability 7.5 cm of shotcrete are given to obtain reliable values of the safety factors as shown in table 5.

It can therefore be stated that whereas final displacements may achieve values around 2.5 cm for an undamaged rock, this value attains around 12 cm for slightly damaged rock, needing in this case an increase in the shotcrete layer width.

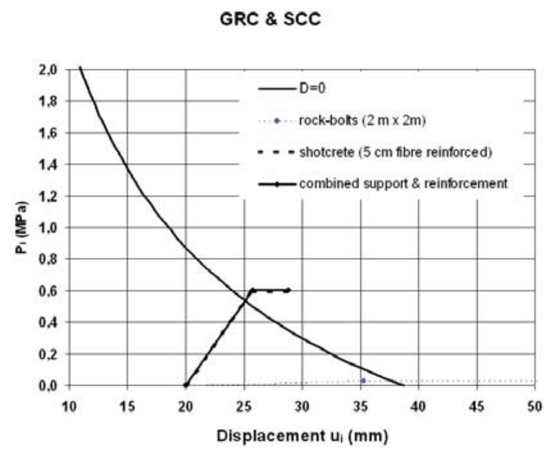

Figure 15. Ground reaction curve and support and reinforcement characteristic curve as designed for the drift excavated in the undamaged rock mass (*D*=0).

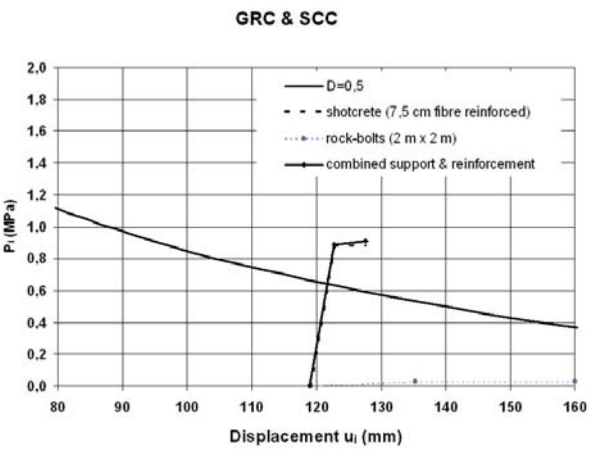

Figure 16. Ground reaction curve and support and reinforcement characteristic curve as designed for the drift excavated in the somewhat damaged rock mass (*D*=0.5).

For the highly damage rock (*D*=0.8), the LDP approach by Vlachopoulos & Diederichs (2009) reflected in Hoek et al. (2008) indicates 1 m of closure in the face, which means that the full collapse of the unsupported drift can be expected, or high demand of support and problems. Also the plastic radius achieve in this case around 4.9 m and well over the standard bolt length. Even if CCM considers all the rock mass damaged instead of an annulus around the excavation, so the displacement results may be overestimated, therefore this type of situation should be avoided at all cases.

Table 5. Results of the application of CCM to the indicated drift in basaltic rock for the support and reinforcement indicated in the main text.

	No damage	Little damage
	D=0	D=0.5
R_{pl} / R	1.7	2.05
u_{eq} (mm)	25.15	127.7
P_{eq} (MPa)	0.54	0.64
u_{max} (mm)	28.8	121.7
P_{max} (MPa)	0.607	0.908
F.S.(Hoek, 99-stress)	1.13	1.42
F.S.(Oreste,03-strain)	1.70	3.21
$S.F._{\sigma}$ x $S.F._{\varepsilon}$	1.93	4.57

Following the prices given in Figure 2, it turns out that an increase in support demand of 2.5 cm of shotcrete means around 150 $/m (10-15 %) cost increase for *D*=0.5. For *D*=0.8, obviously the needs of support would increase much more and serious problems would arise. However, the application of careful perimeter blasting would increase drilling cost in around 30 %, representing a total drifting cost increase around 6 % (80 $/m). So, for this case if the calculation process correctly represents actual behaviour, this strategy would be suggested. If perimeter blasting is applied and carefully performed, it would be much easier and cheap to support and reinforce the drift. If not, problems can be expected.

Another approach to the topic is to consider the rock mass quality according to Barton et al. (1974) and the approach by Suorineni et al (2008), according to which, to estimate the needed support, the rock damage can be quantified with C_f,. The estimate of *Q* for the rock mass is around 2.5 and for the case of C_f 0.8 and 0.6 it diminishes to 2 and 1.7, which means a little increase in the demand of support according to tables of support application in underground excavations (Grimstad & Barton, 1993). Not being the result equal, the same trend is detected as in the CCM approach previously presented. This approach by Suorineni et al. (2008) is specially designed and it works better, however, for better quality rock masses.

6.2 *A drift in a sandstone good quality rock mass*

A drift excavated in a good quality rock mass is now considered. The rock mass characterization was taken from Cai et al. (2007). For the purpose of this study a mine drift 4 m x 4 m is excavated 1,000 m deep, in a rock mass with GSI = 65, σ_{ci} =162 MPa and m_i =19. An isotropic stress field σ_0 =26 MPa is assumed. The residual stress failure has been taken from Cai et al. (2007), considering a residual value of $GSIr$ =28.

With these initial values, the peak stress failure values have been estimated according to RocLab for the cases of excellent quality blasting (D=0), low quality blasting (D=0.5) and very poor blasting (D=0.8) and presented in Table 6. The values of dilatancy and the plastic parameter for the residual case have been quantified as in the previous case.

Table 6. Geomechanical parameters of the sandstone rock mass for different damage levels.

	No damage	Little damage	High damage
	D=0	D=0.5	D=0.8
γ (kN/m^3)	26	26	26
E (MPa)	25584	14279	9219
ν	0.2	0.2	0.2
c_{peak} (MPa)	5.967	5.014	4.304
ϕ_{peak} (º)	49.88	46.70	43.35
$c_{res.}$ (MPa)	3.001	2.199	1.585
$\phi_{res.}$ (º)	39.01	31.55	24.51
ψ (º)	9.98	9.34	8.67
η^*	0.021	0.024	0.025

For these differently damaged rock masses we have obtained the GRC following the technique by Alonso et al. (2003). Other similar approaches, such as that by Guan et al. (2007), can be used for the same purpose. The curves are shown in Figure 17.

The results obtained are an indication of the fact that the damage to the rock by means of blasting in this case does not produce a high variation in rock mass performance, since final displacements are in the range from 3 to 12 mm. Also, the calculated plastic radii for the unsupported drift varies from R_{pl} = 2.2 m for the undamaged case (D=0), to R_{pl} = 2.35 m for the slightly damage case (D=0.5) and R_{pl} = 2.55 m for the high damage situation (D=0.8). In all these cases a little application of support in the form of 2.5 cm layer of shotcrete would ensure stability, with safety factor well over 3 in all cases.

These results from the continuous approach should be carefully taken. We have to bear in mind that discontinuity-controlled instability problems are neither considered, nor spalling behaviour, typical of very hard rocks as it is the case.

Nevertheless, according to the CCM approach, perimeter blasting would not be a better option in this case. This is a clear evidence that proves that the application of perimeter or contour blasting techniques should be analysed in a case by case basis.

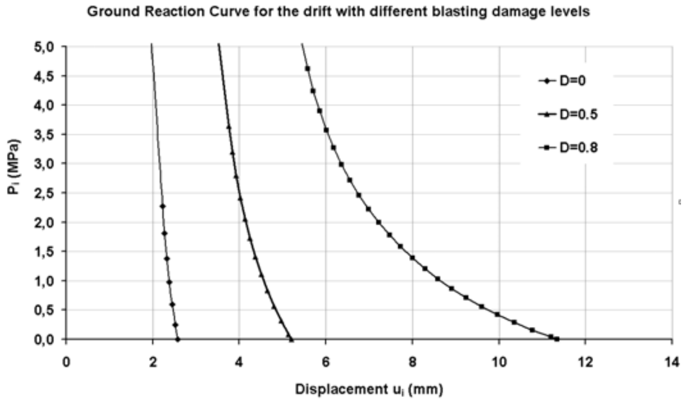

Figure 17. Ground reaction curves for tunnels excavated in the sandstone rock mass with different damage levels.

The estimate of Q for the rock mass in this case is around 10 and for the case of C_f = 0.8 and C_f =0.6 it diminishes from this figure to 6 and 4 respectively, which means that in all cases support is not needed for the 2 m diameter drift according to tables of support application in mining drifts (Grimstad & Barton, 1993) and for ESR = 1. Remark that this approach does not tend to be directly applied in mining as suggested by Suorineni et al. (2008).

7 CONCLUSIONS

Three ingredients are put together in this study, namely costs of support and reinforcement derived from average computation in various Peruvian metallic mines, techniques of quantification of rock mass damage permitting to incorporate this topic in the parameters of classical rock mass behaviour models and the convergence confinement technique to propose adequate support and reinforcement systems and to asses the effectiveness of the stabilization.

In what concerns rock mass characterization, not only the blasting damage (following the approach by Hoek et al., 2002) has been accounted for, but special attention has been devoted to the adequate characterization of the post failure behaviour including estimate of the residual failure envelope (Cai et al., 2007) and the estimate of post-failure deformability characteristics in terms of drop modulus or values of plastic parameters for the residual stage. The GRC have been calculated in their rigorous form according to the approach by Alonso et al. (2003) capable of coping with this type of rock mass behaviour.

The applications developed consisted in estimating the support and reinforcement in two drifts: one in a low strength basaltic average quality rock mass and the other in a very strong good quality sandstone rock mass. In the first case, it turns out that according to the CCM approach, important differences in support demand are revealed. This would indicate that carefully contour blasting would be a correct strategy for the excavation of this drift. In the second case and, probably, due to the fact that the strength of the rock avoids a significant extent of the plastic aureole, no differences in support demand are foreseen by the method. These results are not equal, but reasonably consistent, with the conclusions derived form rock mass classification systems accounting for blasting damage as developed by Suorineni et al. (2008).

All these facts can also be an indication that rock damage affects more dramatically to drifts suffering a great deal of failure around.

Also, it has been observed that in Peruvian mining either they contour blast all drifts, either they never do so. It seems that the presented approach could be an effective tool to establish some simple rules, which can help to decide when to apply or not to apply contour blasting, according to the cases. Remark that spalling and brittle failure are considered to be out of the scope of this work.

Further work consists of fine-tuning post-failure rock behaviour, implementing the approach in continuous numerical models delimiting the extent of the damage zone and, finally, the application to well documented case studies.

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