Estimation of Scale Effects of Intact Rock Using Dilatometer Tests Results

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ABSTRACT
Scale effects of mechanical properties have been observed on various types of rock. The reduction of peak strength and deformation modulus as the size of the sample tested increases is a significant factor for stability analyses. To estimate the large scale strength of intact rock, it is proposed to use in situ dilatometer tests results combined with laboratory tests on standard size samples. A correlation between the scaling of the deformation modulus and of the uniaxial compressive strength is established. This correlation is used to estimate the large scale strength of intact rock. This approach leads to more realistic results than the usual techniques used in geomechanics and ground control.

RÉSUMÉ
Des effets d'échelle des propriétés mécaniques ont été observées sur différents types de roche. La réduction de la résistance au pic et du module de déformation lorsque le volume de l'échantion testé augment est un facteur important lors des analyses de stabilité. Pour estimer la résistance de la roche intacte à grande échelle, il est proposé d'utiliser les résultats de mesures in situ avec un dilatomètre combinés à des essais en laboratoire sur des éprouvettes de dimension standard. Une corrélation entre les effets d'échelle du module de déformation et de la résistance en compression uniaxiale a été établie. Cette corrélation est utilisée pour estimer la résistance à grande échelle de la roche intacte. Cette approche mène à des valeurs plus réalistes que les techniques usuelles utilisées dans en géomécanique et contrôle de terrain.

1 INTRODUCTION

In underground hard rock mines, the size of openings can range from a few cm^3 (boreholes) to tens of thousands m^3 (for very large stopes). Some openings with a cross section smaller than 5 to 10 m^2 and can be used to move workers, equipment, ore, waste rock and backfill. Mine engineers must obviously asses the rock strength around these smaller openings to ensure their stability.

It has been shown by numerous studies that hard rocks exhibit scale effects (e.g. Hoek and Brown 1980, Bieniawski 1984, da Cunha 1993). Such scale effects affect the failure strength and deformability. Although the usual approaches to account for scale effects in rock masses tend to consider these effects globally, these effects can be divided in two components. The first component affects intact rock properties, while the second is related to the introduction of additional types of defects such as rock joints in the rock mass. The usual approaches for scale effects are based on geomechanical classifications and only account for the effect of rock joints, neglecting the influence of volume. In most cases in engineering, these approaches can be used, but for cases where rock joints are not a factor, the rock strength may be well overestimated.

In this paper, the authors present an approach based on dilatometer tests results to estimate the scale effect on the failure strength of intact hard rocks.

2 SCALE EFFECTS

2.1 Intact rock

The observed peak strength of hard rocks usually decreases with sample size. This phenomenon has been attributed mainly to statistical effects due to random bond strength and internal defect distribution (Weibull 1939, Jaeger and Cook 1979). Figure 1 illustrates the influence of size on the strength of hard rocks. The strength σ_n (and deformation modulus) usually decreases exponentially when the sample size increases. At a certain sample size d_L, the decrease practically stops until new types of defects are introduced (such as rock joints). When a few defects are present, the rock behaviour may be highly anisotropic (grey shade) depending on their orientation. When the defects are in sufficient numbers with different orientations (such as family joints), the behaviour can become isotropic. The scale effects are usually more pronounced for rock strength than for the deformation modulus (Fig. 2).

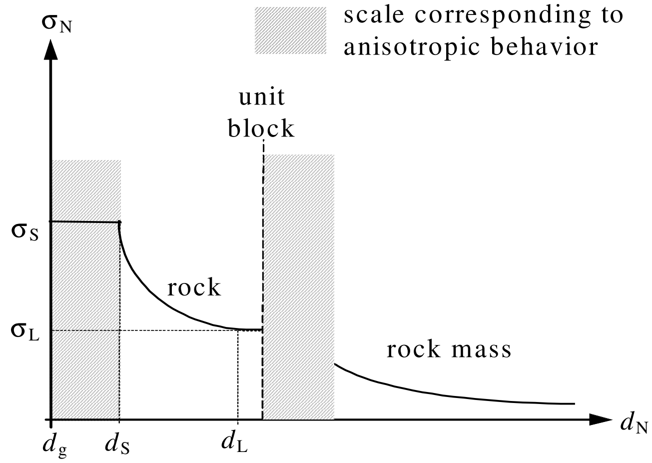


Figure 1. Schematic representation of scale effects on the strength of rock media; the shaded areas represent scales at which strength is not isotropic (adapted from Aubertin et al. 2000).

2.2 Rock masses

The scale effects in rock masses are rather similar to that observed for intact rock, and they are mostly due to the presence of rock joints. Like cracks at a smaller scale, joints are weak elements that reduce the failure strength and the deformation modulus. The usual approaches take into account the scale effects through the use of geomechanical classifications. Over the years, several authors have proposed empirical equations to estimate the scale effects on strength (e.g. Hoek and Brown 1980, 1988, 1997, 2002) or on the deformation modulus (e.g. Bieniawski 1978, Serafim and Pereira 1983, Nicholson and Bieniawski 1990, Mitri et al. 1994, Barton 2002). Table 1 shows some of these empirical equations. Figure 3 shows two of the most popular formulation proposed by Hoek and Brown (1988) and Nicholson and Bieniawski (1990) respectively for the strength and the deformation modulus.

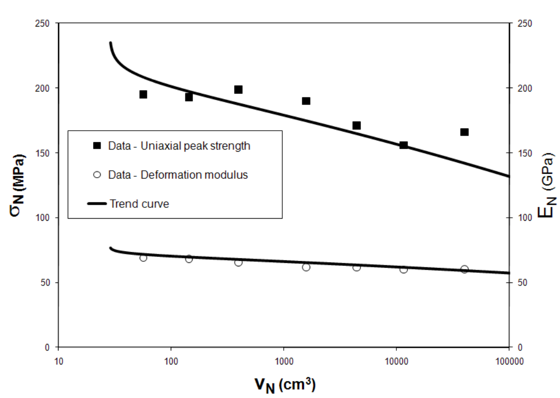


Figure 2. Variation of intact rock properties as a function of volume (data taken from Jackson and Lau 1990).

As it can be observed on Figure 3, the rock mass properties given by these equations in the absence of rock joints (i.e. with a rock mass rating RMR ≈ 100) are similar to the ones obtained in laboratory on a 50 mm diameter sample. This means that there is no effect of the volume tested with these equations. Neglecting this effect may lead to a gross overestimation of strength in the stability analysis of small openings (when RMR is high) that can be a hazard to personnel or equipments.

Table 1. Empirical equations to estimate the rock mass deformation modulus E_m.

Authors	Required parameters	Equation
Bieniawski 1978	RMR	$E_m = 2\ RMR - 100$
Serafim & Pereira 1983	RMR	$E_m = 10^{(RMR-10)/40}$
Nicholson & Bieniawski 1990	E_i & RMR	$E_m = E_i/100\ [0.0028RMR^2 + 0.9Exp\ (RMR/22.82)]$
Mitri et al. 1994	E_i & RMR	$E_m = E_i\ [0.5\ (1 - (\cos(\pi * RMR/100)))]$
Hoek & Brown 1997	GSI & σ_c	$E_m = (\sigma_c/100)^{0.5}\ 10^{(GSI-10)/40}$
Barton 2002	Q & σ_c	$E_m = 10Q_c^{1/3}$; $Q_c = Q * \sigma_c / 100$

RMR, GSI, & Q are geomechanical classifications, E_i : intact rock deformation modulus, σ_c : uniaxial compressive strength of intact rock.

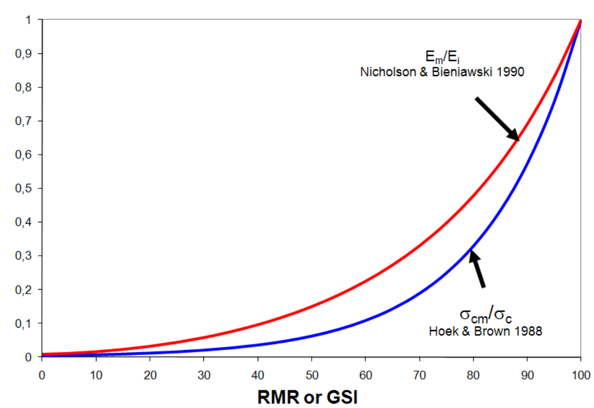


Figure 3. Evolution of rock mass properties as a function of the geomechanical classification (RMR or GSI). The equation from Nicholson & Bieniawski 1990 applies to the deformation modulus and the equation from Hoek & Brown 1988 applies to the peak uniaxial strength. σ_{cm} : rock mass uniaxial strength, σ_c : intact rock (50 mm diameter sample) uniaxial strength, E_m : rock mass deformation modulus, E_i : intact rock (50 mm diameter sample) deformation modulus.

To take into account the effects of size and rock joints on the rock mass strength, Aubertin et al. (2000) have proposed a two step reduction of the strength properties.

The correction for the rock mass strength is given by a continuity parameter Γ (Aubertin et al. 2000, 2001):

$$\sigma_{cm} = \Gamma\, \sigma_c \quad [1]$$

$$\Gamma = \Gamma_{100}\left[0.5\left(1-\cos\frac{\pi\,\mathrm{RMR}}{100}\right)\right]^p \quad [2]$$

with

$$\Gamma_{100} = \frac{\sigma_{cL}}{\sigma_c} \quad [3]$$

Here, σ_{cm} is the rock mass uniaxial compressive strength, σ_{cL} is the intact rock large scale uniaxial compressive strength, σ_c is the intact rock uniaxial compressive on a standard diameter sample (i.e. 50 mm), RMR is the value of the geomechanical classification (Bieniawski 1989), p is a material parameter that varies between 2 and 3 and Γ_{100} is the value of the continuity parameter for a RMR value of 100 (i.e. without rock joint). Figure 4 shows the comparison between the Hoek-Brown and the Aubertin et al approaches. It can be seen that these approaches are similar up to a RMR value of about 70. For RMR > 70, the Aubertin et al approach tends to a maximum value Γ_{100} defined by Eq. 3. However, the value of the large scale strength σ_{cL} cannot be easily obtained from typical site investigation. This value can either be obtained by laboratory testing different sample sizes, which can be very expensive and time consuming, or can be estimated by other means.

3 INDIRECT EVALUATION OF THE LARGE SCALE STRENGTH OF INTACT ROCKS

It has been observed in several studies (e.g. Jackson and Lau 1990, Tsoutrelis and Exadaktylos 1993, Singh et al. 1997) that there is a correlation between the scale effect on the deformation modulus and the uniaxial peak strength. To try to evaluate this correlation between strength and deformation modulus, a field and laboratory study has been carried out. Boreholes with different sizes have been drilled at the CANMET underground mine in Val-d'Or (Quebec). Boreholes 6 to 8 m long, ranging from size AQ, BQ, NQ and HQ, have been drilled in parallel at three different levels (40, 70 and 130 m). In situ dilatometer tests have been performed in the NQ holes by Labrie et al. (2006). The different rock core samples collected from the diamond drilling have then been tested in uniaxial compression with strain measurements at the École Polytechnique rock mechanics Laboratory. The results presented in this paper come from the 130 m test site (shown in Figure 5) but the three sites showed similar trends (Simon 2008).

3.1 Laboratory tests

For each size holes, ten samples were collected and tested in uniaxial compression with strain measurements. The different core sample diameters were respectively after preparation 30, 36, 48 and 63 mm. These tests have been conducted in conformity with ASTM standard. Figure 6 shows the results of the mean uniaxial compressive strength as a function the sample diameter (with the standard deviation range). It shows a reduction of the mean strength as the diameter of the sample with a very good correlation with a negative power law. Figure 7 shows the variation of the mean deformation modulus (taken as the tangent slope at 50% of the peak strength) as a function on the sample diameter. Here again, a decrease in the value of the modulus as the diameter increases is observed. Figure 8 shows the scale effect for the mean uniaxial compressive strength and mean deformation modulus normalized by the value of the standard diameter of 50 mm (actually, 48 mm). It can be seen that the scale effect is more pronounced for the peak strength than for the deformation modulus.

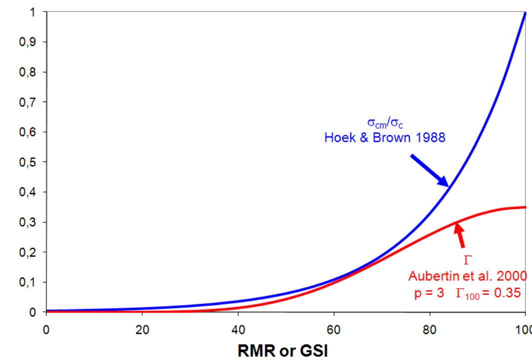


Figure 4. Comparison between the Hoek-Brown (1988) and the Aubertin et al. (2000) approach to estimate rock mass strength.

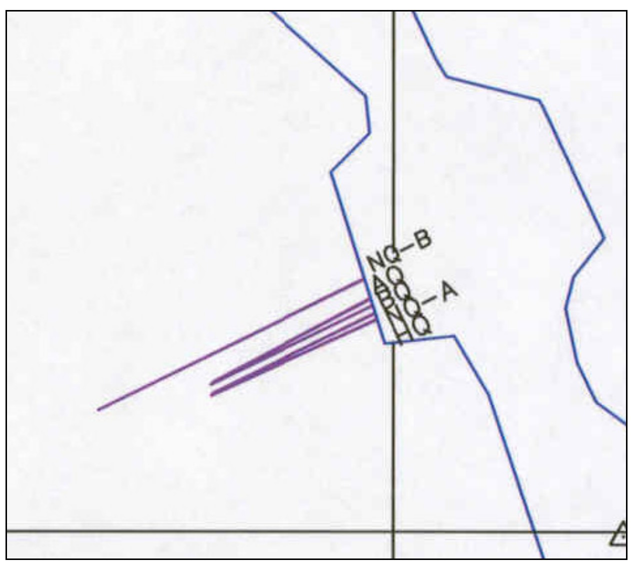


Figure 5. Test site for the 130 m level diamond drill boreholes (after Labrie et al. 2006).

3.2 In situ dilatometer tests

Dilatometer tests have been performed at the three different locations by Labrie et al. (2006). The dilatometer used is a ROCTEST, Probex-1 model. The dilatometer test performed inside drill holes consist in applying a continuous and constant pressure on the borehole walls and measuring the resulting deformation. The pressure is applied through a hydraulic radially expandable cylindrical probe that fit in N size boreholes. Five or six different pressure levels are enough to plot the curve of the pressure vs injected fluid volume. The deformation modulus can then be determined. Labrie et al. (2006) performed 14 dilatometer tests in the NQ (A & B) boreholes of the 130 m level site (Fig. 5). The mean deformation modulus obtained with the dilatometer tests is 28 GPa, comparatively with the mean value of 73 GPa obtained in laboratory tests performed on the core from these holes. This somehow proves that the volume involved in this test is much larger than that of the laboratory tests. It is also interesting to note that the values obtained with the dilatometer tests are lower than those obtained from most empirical equations proposed in the literature and shown in Table 1 (see Simon 2008 for more details).

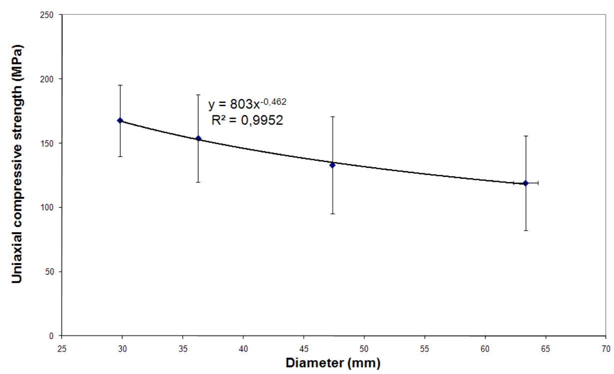


Figure 6. Mean uniaxial compressive strength as a function of the sample diameter for the 130 m level.

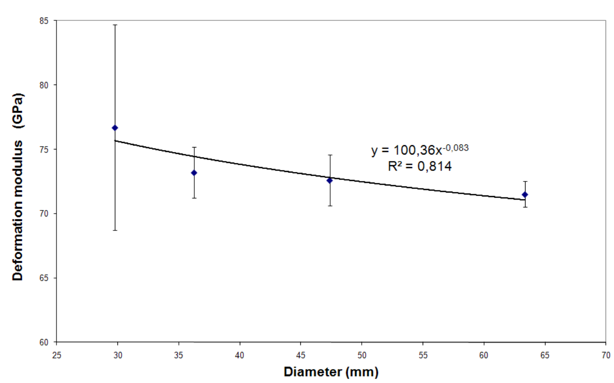


Figure 7. Mean deformation modulus as a function of the sample diameter for the 130 m level.

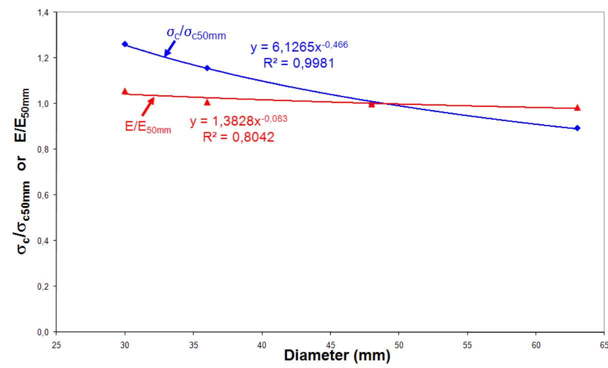


Figure 8. Variation of the mean uniaxial compressive strength and of the mean deformation modulus normalized by the mean value of the standard size (50 mm) as a function of the sample diameter for the 130 m level.

To estimate the volume implied in the dilatometer test, a numerical model was built using the FLAC software (Itasca 2002). Different analyses showed that the volume around the hole affected by the test is around 141 000 cm^3 (Fig. 9) or a equivalent diameter of around 2 m.

Figure 10 shows the variation of the deformation modulus as a function of volume including the dilatometer test results. It can be seen that the dilatometer test results does not really follow the trend of the laboratory results (even though the correlation factor is high). This could be explained by the influence of rock joints in the dilatometer test, which make the scale effect discontinuous (see Fig. 1). The deformation modulus value obtained with the dilatometer test cannot be considered to be a value of intact rock.

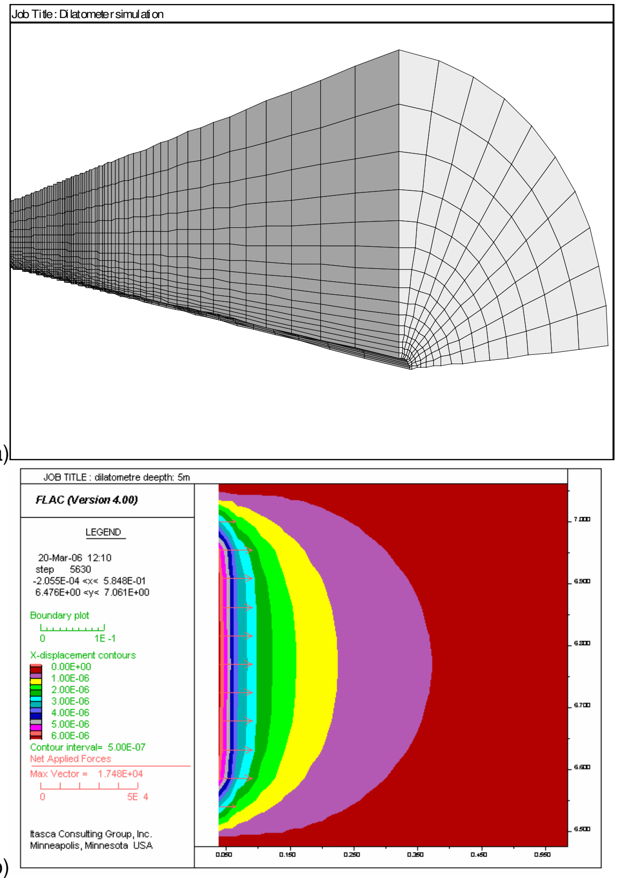
4 PROPOSED APPROACH TO ESTIMATE THE SCALE EFFECT OF INTACT ROCK

4.1 Estimation of the large scale deformation modulus

To be able to use the results obtained for the dilatometer tests, a correction must be applied to estimate the value of the deformation modulus of the rock mass without joints, or large scale modulus (E_L). To obtain the value of E_L, Eq. 2 can be transformed to:

$$E_L = \frac{E_D}{\left[0.5\left(1-\cos\frac{\pi\,RMR}{100}\right)\right]^p} \qquad [4]$$

Where E_L is the large scale deformation modulus and E_D is the deformation modulus obtained with the dilatometer test. Here, it is suggested to use a value of 1 for the p exponent. Note that with p = 1, the equation is similar than that proposed by Mitri et al. (1994). The corrected value (or large scale modulus) E_L obtained for the 130 m level is then 31.4 GPa (for a measured RMR value of 77.5).



a)

b)

Figure 9. Estimation of the volume influenced in the dilatometer test. a) Model built using the Flac software. b) displacements obtained from the simulation.

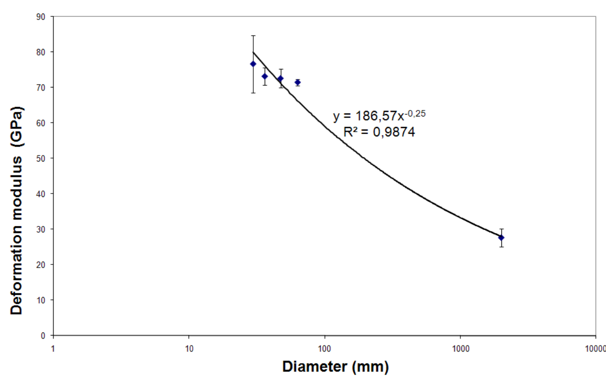


Figure 10. Variation of the deformation modulus) as a function of the diameter for the 130 m level (dilatometer test results taken from Labrie et al. 2006).

4.2 Scale effect of the uniaxial compressive strength

To define the scale effect evolution affecting the uniaxial compressive strength, Aubertin et al. (2002) have proposed the following equation:

$$\sigma_N = \sigma_L + (\sigma_S - \sigma_L)\left\langle \frac{d_L - d_N}{d_L - d_S} \right\rangle^{\lambda} \qquad [5]$$

Where σ_N is the strength of a d_N diameter sample, σ_S is the small scale (d_S) strength, σ_L is the large scale (d_L) strength, λ is an exponent that controls the non-linearity of the curve and $\langle\ \rangle$ is given by ($\langle x \rangle = (x + |x|)/2 \geq 0$). This last parameter means that if $d_N > d_L$, the value of $\langle\ \rangle$ is 0 and then $\sigma_N = \sigma_L$. Figure 11 shows the type of curves obtained from Eq. 5.

The study of scale effects of Li et al. (2007) has shown that the small and large scale diameter d_S and d_L are typically the same for several fine grained rock types. The values proposed by Li et al. (2007) are d_S = 5 mm and d_L = 2000 mm. When there is no value available for the exponent λ, Li et al. (2007) suggested using a value of 20.

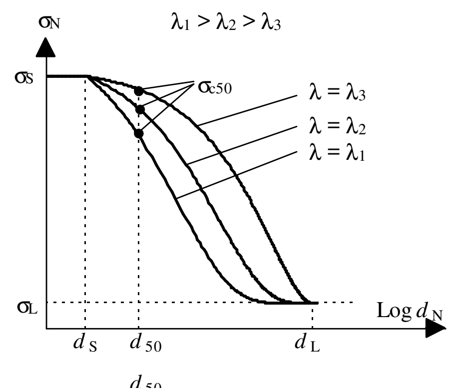


Figure 11. Schematic representation of the strength scale effect given by Eq. 5 where σ_{c50} is the uniaxial compressive strength of a 50 mm diameter sample (after Li et al. 2007).

To estimate the small scale strength σ_S, a simple statistical approach was developed by Li et al. (2007). This approach is based on the hypothesis that for a small enough probability (that depends of the number of tests performed), a statistical maximum limit would correspond to the small scale strength. This estimation can be formulated as follow (Li et al. 2007):

$$\sigma_{cS} = \overline{\sigma}_c + \frac{t_{\alpha/2,\nu}}{\sqrt{n-1}} S_0 \qquad [6]$$

Where σ_{cS} is the small scale uniaxial compressive strength, $\overline{\sigma}_c$ is the mean value of the uniaxial compressive strength of standard diameter samples, $t_{\alpha/2,\nu}$ is the Student law function with a confidence level of α and a ν (= n – 1) degree of liberty, n is the number of samples tested and S_0 is the standard deviation of the uniaxial strength measured values. Based on Eq. 6, a value of 211 MPa for σ_{cS} was obtained for the 130 m level.

To estimate the large scale strength σ_L, an approach based on the dilatometer test and standard laboratory test results is proposed. This approach was developed based on the results obtained for the three sites investigated (Simon 2008). The first step is to plot the curve of the corrected large scale deformation modulus (Eq. 4) with the standard deformation modulus (50 mm diameter sample) as shown in Figure 11. A power law curve can then be determined with the following general format:

$$\frac{E_L}{E_{50mm}} = m_1 * d^{-m_2} \qquad [7]$$

A similar curve can also be established for the strength:

$$\frac{\sigma_L}{C_{0\,(50mm)}} = c_1 * d^{-c_2} \qquad [8]$$

Based on the results obtained for the 3 sites, the following empirical equations are proposed to relate the deformation modulus and strength size functions:

$$c_1 = m_1{}^2 \qquad [9]$$

$$c_2 = \sqrt{m_2} \qquad [10]$$

For the 130 m level site, this leads to values of 5.7 and 0.47 for c_1 and c_2 respectively. The values obtained with Eq. 9 & 10 are relatively similar to the ones given by the strength trend curve in Fig. 11. For a large scale diameter of 2000 mm, Eq. 8 gives a large scale strength σ_L of 20.5 MPa.

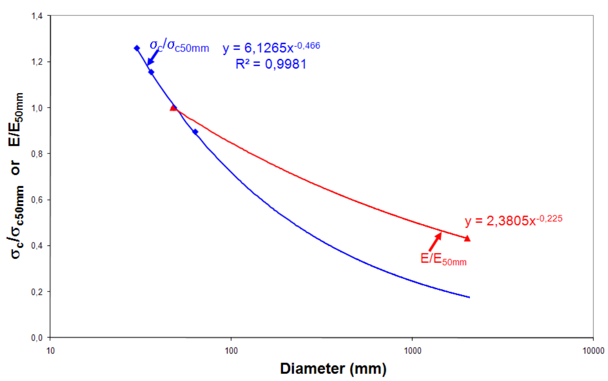


Figure 11. Variation of the normalized uniaxial compressive strength and normalized deformation modulus for the 130 m level.

Figure 12 shows the final scale effect curve based on Eq. 5. It can be seen that the proposed curve differs from the trend curve fitted on the strength data.

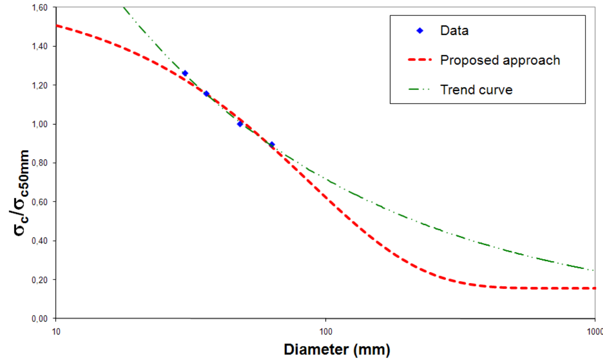


Figure 12. Variation of the uniaxial compressive strength of the 130 m level site obtained with Eq. 5. σ_S = 211 MPa, d_S = 5 mm, σ_L = 20.5 MPa, d_L = 2000 mm and λ = 23.

5 DISCUSSION AND CONCLUSIONS

The laboratory and field results obtained in this study indicate that there is a scale effect on both the uniaxial compressive strength and the deformation modulus as both properties tend to decrease as size increases. It also shows that the large scale strength of intact rock can be much smaller than the strength of standard size samples.

Dilatometer test results also showed smaller values than the values obtained from empirical equations proposed in literature. This illustrates how usual approaches can lead to overestimations by only considering the effect of rock joints. The deformation modulus measured in situ showed values lower by 15% to 170% than the deformation modulus obtained with the empirical equations shown in Table 1 (see Simon 2008 for more details). It appears that a two-step approach is more adequate to take into account scale effects for the rock and rock mass.

An approach to estimate the large scale strength of intact rock has been proposed. This approach is based of dilatometer test results and laboratory tests on standard size samples. This approach is based on the hypothesis that there is a link between the scale effect on strength and deformability of hard rocks. This hypothesis might not be true for all type of rocks. It is also assumed here that the volume involved with the dilatometer test is large enough to provide a good estimate of the large scale deformation modulus. The scale effect on the modulus observed in this study indicates that the ratio of E_d/E_{50mm} is between 0.37 to 0.43, which is lower than all the ratios E_m/E_{50mm} obtained with the equations of Table 1.

The determination of the empirical equations parameters (Eq. 9 & 10) proposed here are based on a very limited number of cases. However, the large scale strength obtained with this approach are very similar to the one obtained with a statistical approach proposed by Li et al. (2007; see Simon 2008 for more details).

If the popular Hoek and Brown (1988) criteria is used to estimate the rock mass uniaxial compressive strength σ_{cm} at the 130m level site, a value of 38.1 MPa is obtained (for a RMR value of 77.5). This value is much larger than the large scale value of 20.5 MPa obtained for intact rock with the proposed approach. Using Eq. 1-3 leads to a value of σ_{cm} between 14 and 16 MPa (depending on the value of p in Eq. 2). Then, the proposed approach can be considered more conservative than the usual approaches for estimating the rock mass strength.

The value estimated for d_L also has an impact on the large scale strength with the proposed approach. This value was estimated by the case studies performed by Li et al. (2007) and by numerical simulation. Figure 13 shows how this value influences the estimation of σ_L. It can be seen that a higher value of d_L reduce the value of the large scale strength with the proposed approach based on dilatometer test results. It could be argued that it would be more conservative to use a higher value than the value used in this study (2000 mm), but more case study are needed to evaluate this aspect.

The mean reduction of strength Γ_{100} for the intact rock estimated in this study with the proposed approach was 0.15. This is in concordance with other studies published in the literature (e.g. Jahns 1966, Bieniawski 1968, Pratt et al. 1972, Herget and Unrug 1974, Singh 1981, Swolfs 1983, Natau et al. 1995). More investigations are however needed to validate the proposed approach.

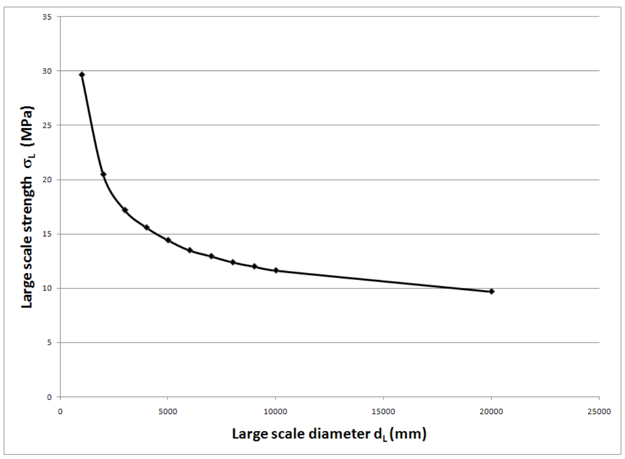


Figure 13. Variation of the estimated large scale strength of the 130 m level site depending on the selected value of the large scale diameter d_L.

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