INVESTIGATION OF SCALE EFFECTS IN HARD ROCKS

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ABSTRACT
Rock and rock masses present a variation of mechanical properties when the tested volume changes. Usual approaches for estimating rock mass properties do not really consider volume change. To investigate the scale effects on rock strength and the deformation modulus, core samples with different size were collected in an underground mine and tested in uniaxial compression. In situ dilatometer measurements were also performed to evaluate the deformation modulus. The results showed a decrease in strength and of the deformation modulus when the tested specimen size increased. The results also highlight that the usual approaches to estimate rock mass properties from those of intact rock samples can overestimate the strength and deformation modulus particularly for a rock masses with few or no joints.

RÉSUMÉ
Les roches et massifs rocheux présentent une variation des propriétés mécaniques lorsque le volume impliqué varie. Les approches usuelles pour extrapoler les propriétés du massif rocheux ne considèrent pas vraiment les variations de volume. Pour étudier les effets d'échelle sur la résistance et le module de déformation, différentes dimensions d'échantillons ont été forées dans une mine souterraine et utilisées pour préparer des éprouvettes testées en compression uniaxiale. Des mesures à l'aide d'un dilatomètre ont également été réalisées in situ pour évaluer le module de déformation. Les résultats ont montré une réduction de la résistance et du module de déformation avec l'augmentation du volume des éprouvettes. Les résultats indiquent également que les approches usuelles pour estimer les propriétés du massif rocheux à partir des propriétés des roches intactes peuvent surestimer la résistance et le module de déformation surtout pour les massifs présentant peu ou pas de discontinuités géologiques.

1. INTRODUCTION

Rock and rock masses have a complex mechanical behaviour. The design of excavation in rock media usually requires the evaluation of the rock mass strength and deformation properties. It is well known that rock properties are scale (or volume) dependant (e.g. Bieniawski 1968, Pratt et al. 1972, Jackson and Lau 1990, da Cunha 1993). Strength and deformation properties of rocks and rock masses are primarily controlled by defects in the matrix. These defects can range from microcracks and pores to shear bands in rocks, to joints and faults in rock masses. When the volume of the rock increases, a decrease in strength and an increase in deformability are usually observed. Because the volume involved around underground excavation is typically very large, it is, most of the time, neither economical nor feasible to determine the rock mass strength at this scale. Thus, to estimate the strength and deformation properties of rock masses, rock mechanics engineers must rely on some extrapolations, usually based on geomechanical classifications combined with laboratory tests on small scale samples (e.g. Hoek and Brown 1980, Gokceoglu et al. 2003).

Figure 1 shows a schematic representation of the scale effect for the strength of rock media. At the rock scale, it can be observed that the strength is reduced with an increasing volume until a limit value where the increase in the sample volume has no more effects. This volume is called large size unit block. At the rock mass scale, where new types of defects are present (such as rock joints), the strength decrease can be even more important. It can also be observed that the strength progression is not continuous. When a new type of defects appears (microcraks and pores when passing from the grain to the rock, and rock joints and faults when passing to the rock mass), it results in an important decrease in strength, mostly controlled by this new type of defect. Although usually less pronounced, the deformation modulus (usually associated to the elastic behaviour) shows a similar behaviour. As to the Poisson's ratio, its scale effects are usually considered negligibles, even if the passage from the large size unit block to the rock mass can also have an influence on this parameter.

In this paper, the authors present the results of an investigation on the scale effects of hard rock samples collected in a Canadian underground mine. Different sample sizes were tested in the laboratory to determine the evolution of peak strength and deformability with sample size. In situ dilatometer measurements were also performed to estimate the rock deformability at a larger scale. The objective of this investigation was to identify means to better estimate rock mass strength.

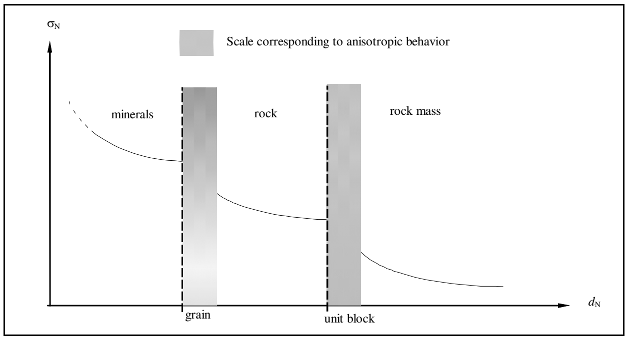

Figure 1 Schematic representation of scale effect on rock properties; the shaded areas represent scales at which strength is not isotropic. σ_N: nominal strength of sample with nominal size d_N (after Aubertin et al. 2000).

2. EVALUATION OF SCALE EFFECTS IN ROCKS

When performing a site characterization on a rock mass, the sample dimensions collected will have a strong influence on the results. Figure 2 shows on a logarithmic scale, the influence of the collected samples size obtained from diamond drilling. For laboratory testing, the American Society for Testing and Materials (ASTM) recommends a sample diameter of 50 mm (ASTM 1998), the equivalent of a NQ core.

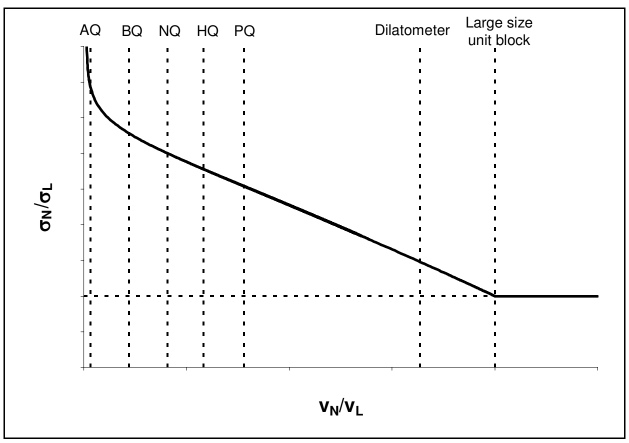

Figure 2. Schematic illustration of the rock strength in relation with the volume tested; σ_N, V_N: strength and sample volume; σ_L, V_L: strength and volume of the large size unit block.

The rock strength decreases progressively until the sample size reaches d_L, the large size unit block size. Afterwards, the scale effects disappear almost completely unless a new type of defects appears (such as joints for the rock mass). The rock strength at size d_L can be as low as 5 to 30% of σ_s, the strength at small scale d_s obtained in the laboratory (e.g. Jackson and Lau 1990, Sridevi and Sitharam 2000, Aubertin et al. 2001).

Scale effects are more pronounced in very brittle materials, and they progressively decrease when going from a brittle to a semi-brittle behaviour, altogether disappearing in the ductile (fully plastic) regime of inelastic flow. The influence of scale is also more pronounced in uniaxial tension than in uniaxial compression, and can be reduced substantially by applying a large confining pressure in triaxial compression tests.

To determine the scale effects, laboratory tests with different sample sizes can then be conducted. This can however be expensive and thus increase the cost of site investigations. The determination of properties at the rock mass scales usually bases on extrapolations using geomechanical classifications and standard laboratory tests on intact rock with a standard sample size (50 mm diameter).

2.1 Strength

One of the most popular approaches used to estimate the rock mass strength was proposed by Hoek and Brown (1980, 1988, 1997) with their failure criterion. They proposed a reduction of the strength based on the RMR or GSI geomechanical classification value by reducing strength parameters m and s accordingly. The values for undisturbed rock masses are then expressed by (Hoek et al. 1997):

$$\frac{m}{m_i} = \exp\left(\frac{GSI - 100}{28}\right) \qquad [1]$$

$$s = \exp\left(\frac{GSI - 100}{9}\right) \qquad [2]$$

where m and s are the strength parameters for the rock mass, m_i is the criterion parameter for intact rock, GSI is similar to the 1976 version of the RMR classification (Bieniawski 1984, 1989), or is equal to the RMR value less 5 for the 1989 version of the RMR classification.

2.2 Deformability

Many empirical relationships have been proposed over the years to estimate the deformation modulus at the rock mass scale. These are usually extrapolation either strictly based on geomechanical classifications, or based on a combination of these classifications and properties obtained in the laboratory on standard size samples. The most popular are the ones proposed by Bieniawski (1978), Serafim and Pereira (1983) and Nicholson and Bieniawski (1990) given in Table 1. Depending on the equation used, one can obtain very different values with the difference between the lower and higher values of more than 100% (Gokceoglu et al. 2003).

The major limitation of these approaches for strength and deformability is that they take into account the influence of geological discontinuities to reduce the rock mass properties, but they do not really consider the scale effects caused by an increasing volume. When the rock mass has a high RMR or GSI value, the extrapolated rock mass properties are close or equal to that obtained in laboratory on a 50 mm diameter sample. Figure 3 shows the evolution of properties according to the equations proposed by Nicholson and Bieniawski (1990) for the deformation modulus and Hoek and Brown (1980, 1988, 1997) for peak strength as a function of the RMR or GSI value. It can be seen that for a RMR value of 100 (corresponding to a rock mass without joints, i.e. identical to large size unit block), the mechanical properties are identical to that of a 50 mm diameter sample. This contradicts the findings of many studies on size effects that show an actual decrease of properties with an increasing volume (e.g. Hoek and Brown 1980, Bieniawski 1984, Martin and Read 1992, da Cunha 1993). The above mentioned extrapolation approaches can then lead to a non conservative estimation of the rock mass properties.

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

Authors	Limitations	Equation
Bieniawski (1978)	RMR > 50	E_m = 2 RMR –100
Serafim and Pereira (1983)	RMR ≤ 50	$E_m = 10^{(RMR-10)/40}$
Nicholson and Bieniawski (1990)		$E_m = \frac{E_i}{100}\left[0.0028\,RMR^2 + 0.9\exp\left(\frac{RMR}{22.82}\right)\right]$

E_m: rock mass deformation modulus (GPa); E_i: intact rock deformation modulus (GPa); RMR : geomechanical classification value.

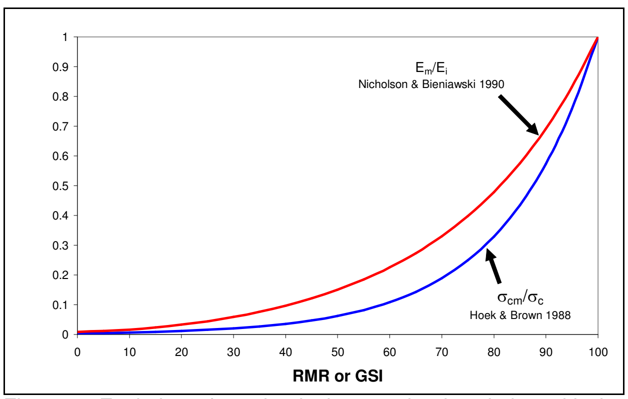

Figure 3. Evolution of mechanical properties in relation with the RMR or GSI value; σ_{cm}: rock mass uniaxial compressive strength, σ_c: uniaxial compressive strength of a standard rock sample (50 mm diameter), E_m: rock mass deformation modulus, E_i: deformation modulus of a standard rock sample.

To take into account both the volume effect and the presence of joints on the rock mass strength, Aubertin et al. (2000, 2001, 2002) have proposed the reduction of strength parameters Γ, given by:

$$\Gamma = \Gamma_{100}\left[0.5\left(1-\cos\frac{\pi RMR}{100}\right)\right]^p \quad [3]$$

with $$\Gamma_{100} = \sigma_{cL}/\sigma_c \quad [4]$$

Here Γ is a strength reduction factor called continuity (value between 0 and 1), Γ_{100} is the strength reduction factor for a RMR value of 100, σ_{cL} is taken as the uniaxial compressive strength of the rock at size d_L (the large size unit block), while σ_c is the standard size specimen strength of intact rock. Eq. (3) is based on the expression proposed by Mitri et al. (1994) for the deformation modulus where an exponent p was added to better relate strength and deformability parameters. A value of 3 for p was suggested by Aubertin et al. (2000). As it can be seen, there are two reductions for the rock mass strength. The first one given by Eq. (4) leads to the strength at the large size unit block, while the second reduction (Eq. 3) takes into account the influence of major defects such as rock joints and faults as represented by the RMR value. It is believed that such an approach leads to a better estimate of the rock mass strength. However, the determination of the value of σ_{cL} and the sample size d_L at which it is attained still poses some challenges.

3. IN SITU AND LABORATORY INVESTIGATIONS

The main objective of this experimental investigation was to estimate the properties of the large size unit block by a more practical and economical method than by testing samples with different sizes. To achieve this objective, the relation between the deformation modulus and the uniaxial compression strength of hard rocks was studied. Many researchers have found a connection between these properties (e.g. Gijòn and Gonzalez de Vallejo 1991, Pells 1993, Palchik 1999). Although a large deformation modulus does not always lead to a high strength, these two properties are largely controlled by the same phenomena. At the intact rock scale, the deformation modulus is influenced by the presence of pores and. Similarly, these defects can reduce the compressive strength. However, microcracks orientation and geometry will affect differently strength and deformation.

The deformation modulus shows similar scale effects as strength. Most empirical equations proposed in the literature for both properties are quite similar. A good example was shown in Figure 3 where the equation proposed by Nicholson and Bieniawski (1990) for the deformation modulus is compared with the one proposed by Hoek and Brown (1988) for strength. However, since most rock masses are not linear elastic media, it can be expected that the ratios of E_m/E_i (rock mass modulus / intact rock modulus) and of peak strength σ_{cm}/σ_c (rock mass uniaxial compressive strength / intact rock uniaxial compressive strength) are not identical. Nevertheless, if it can be shown that these properties follow a similar trend, it should be possible to estimate the large size unit block strength based on the evolution of the deformation modulus at a larger scale. The Γ_{100} parameter could then be estimated from the E_L/E_i ratio where E_L is the modulus at the large scale and E_i is the modulus of a standard size sample. To evaluate the deformation modulus at the large size unit block scale, the authors have used in situ dilatometer tests. It is recognised that this type of measurement is at an intermediate scale between the standard laboratory sample scale and the rock mass scale (Wittke 1990). Dilatometer measurements can be an economical alternative for a better estimation of large size unit block and rock mass mechanical properties.

To evaluate the possible correlation between deformation modulus and strength evolutions with respect to scale, several parallel holes of different calibre were drilled with a diamond drill and cores were retrieved. These diamond drill holes were performed in the spring and fall of 2005 at the CANMET underground Laboratory-Mine in Val-d'Or, Quebec. Four hole calibres were drilled (AQ, BQ, NQ and HQ) and were 6-9 m in length. The drilling was performed at three locations in the mine at levels 40 m, 70 m and 130 m. Dilatometer measurements were carried out in NQ holes by Labrie et al (2006). Rock cores were tested at the rock mechanics laboratory of the École Polytechnique de Montréal. An extensive uniaxial compression testing program was then completed for each hole drilled.

4. RESULTS

Partial results are presented here for two of the sites tested, the 40 m level and 130 m level. More results will be presented elsewhere.

4.1 Level 40 m

Two different geological units were identified along the hole: a granodiorite slightly sheared and silicified for the first 4 meters, and a granodiorite with fine to medium grain size in the last 3 meters (Labrie et al. 2006). Results are presented here for the first 4 meters.

Figure 4a presents the variation of peak strength with the sample volume. As it can be seen, the results show a reduction of strength with a larger sample volume tested. The mean strength of HQ samples (larger volume) is only 44% of the mean strength of AQ samples (smaller volume). Moreover, the mean strength of HQ samples is only 77% of the mean strength of the standard size sample (≅ 200 cm^3) obtained from NQ cores. These results seem to indicate that in this case, the Hoek and Brown approach would lead to an overestimation of the strength for the large size unit block. The value of the uniaxial compressive strength given by the Hoek and Brown approach for a GSI of 100 (i.e. large scale with no joints) is equal to that obtained from the NQ core samples (77 MPa) but the mean value obtained from HQ core samples is only 59 MPa.

Figure 4b shows the variation of the mean deformation modulus taken at 50% of peak strength with the sample volume. A decrease in the modulus value with an increasing volume can be observed. The mean modulus value of HQ samples (≅ 425 cm^3) is only 77% of the mean modulus value of AQ samples (smaller volume) and 88% of the mean modulus of the standard sample size (≅ 200 cm^3). It can also be observed that the reduction is less pronounced than that of the uniaxial compressive strength. A dilatometer measurement performed by Labrie et al. (2006) is also shown in Figure 4b.

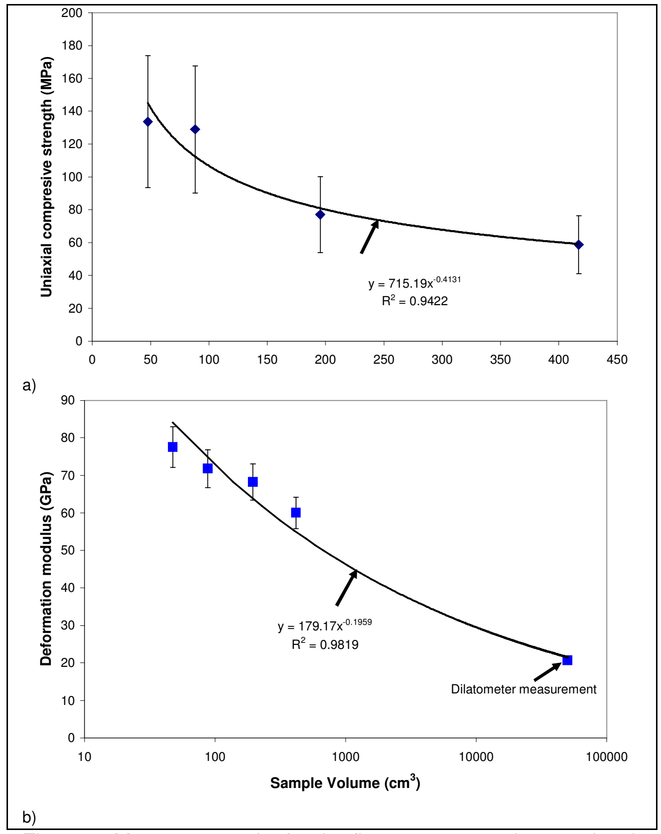

Figure 4. Mean test results for the first 4 meters at the 40m level. Rock type: granodiorite slightly sheared and silicified. Each data point represents 4 test results. The dilatometer measurement is taken from Labrie et al. (2006). Dilatometer volume implied estimated at 50 000 cm^3.

To estimate the volume implied in the dilatometer measurements, the authors used the equation proposed by Wittke (1990):

$$V_d = 6 \pi l d^2 \quad [5]$$

where V_d is the volume associated to the dilatometer measurement, l and d are the length and diameter of the dilatometer. With the dilatometer used (Roctest's Probex-1), this lead to a volume of 50 000 cm^3. However, much more studies are needed to confirm the actual material volume that contributes to this type of measurement.

From Fig. 4b, it can be seen that the deformation modulus obtained with the dilatometer is very low compared to the laboratory test results. Moreover, if one was to use the equation proposed by Nicholson and Bieniaswki (1990) to estimate the rock mass modulus (a RMR value of 75 was determined by Labrie et al. 2006), a rock mass modulus value of 27 GPa would be obtained. This value is 30% larger than the dilatometer measurement obtained (21 GPa). In addition, the equation proposed by Bieniawski (1978) leads to a value of 50 GPa for the rock mass modulus, which is more than twice as high than the dilatometer's result.

4.2 Level 130 m

Two different geological units were also identified along the hole: a granodiorite with fine to medium grain size for the first 5.5 meters, and a granodiorite slightly sheared and silicified in the last 4 meters (Labrie et al. 2006). Results are presented here for the first 5 meters.

Figure 5a presents the variation of peak strength with the sample volume. Here again, the results show a reduction of strength with a larger sample volume tested. The effect is less pronounced than the first site. The mean strength of HQ samples equals 70% of the mean strength of AQ samples and 89% of the mean strength of the standard sample size.

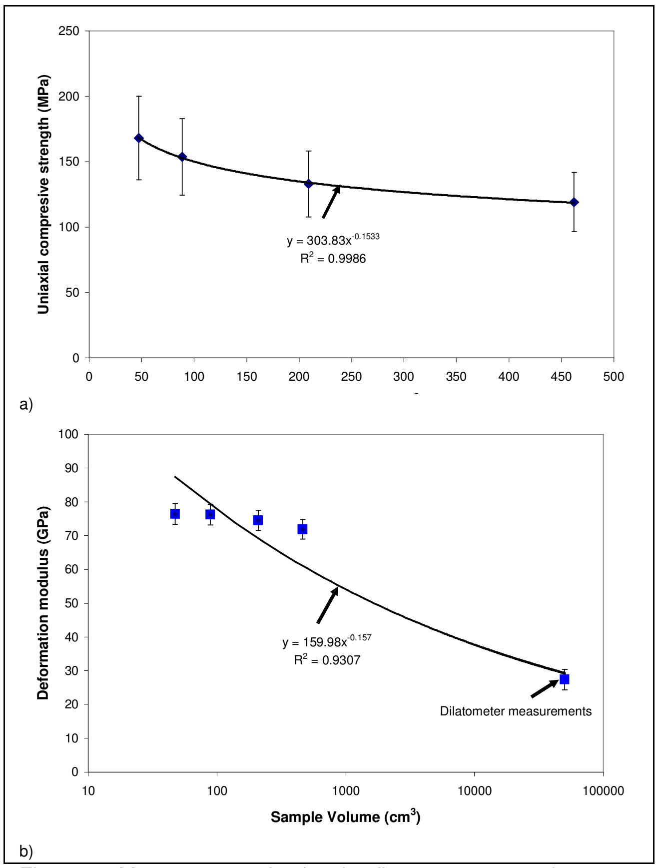

Figure 5. Mean test results for the first 5 meters at the 130m level. Rock type: granodiorite with fine to medium grains. Each data point represents 6 test results. Dilatometer measurements (8 results) taken from Labrie et al. (2006). Dilatometer volume implied estimated at 50 000 cm^3.

Figure 5b shows the variation of the mean deformation modulus taken at 50% of peak strength with the sample volume. A small decrease in the modulus value with an increasing volume can be observed. The mean modulus value of HQ samples equals 94% of the mean modulus value of AQ samples and 96% of the mean modulus of the standard sample size. However, the reduction is less pronounced than that of the uniaxial compressive strength. Dilatometer measurements performed by Labrie et al. (2006) shown in Figure 5b are again much lower than that obtained in laboratory. It can be seen that the modulus obtained with the dilatometer is very low compared with the laboratory test results. If the rock mass modulus is estimated with the equation proposed by Nicholson and Bieniaswki (1990) with a RMR value of 77 (determined at the site by Labrie et al. 2006), a modulus value of 32 GPa is obtained which slightly overestimate the mean dilatometer measurements results (27 GPa) but the results are much closer.

5. DISCUSSION

The results obtained from this study indicate that the usual approaches used to extrapolate rock mass properties may lead to important overestimations of the rock mass strength and deformation modulus. A more systematic approach should be used to estimate these properties, such as the one proposed by Aubertin et al. (2000, 2001, 2002) which takes into account the volume effect and the presence of defects such as rock joints.

Dilatometer measurements also showed much lower modulus values than that obtained in laboratory. It should be recalled however that this kind of instrument leads to a modulus measurement that is perpendicular to the axis of the core sample tested in laboratory. If the rock tested has an anisotropic behaviour, then the dilatometer's results should not be compared with laboratory results performed on the same core samples.

The results also show that the decrease in strength is non linear and that a lower bound value would be obtained for the large size unit block. For the deformation modulus, since rock joints can affect the dilatometer measurements and the value obtained, it is not clear if the large size unit block would be smaller or larger than the volume implied with the dilatometer test. Figures 4b and especially 5b seems to indicate that this volume should be smaller than that of the dilatometer test because the dilatometer's results does not seem to follow the decrease trend obtained form the laboratory samples. However, since the real volume implied in the dilatometer test is unknown, it would be unwise to draw conclusions at this point.

Finally, further analyses of these results are needed to determine the actual volume of the large size unit block and how the dilatometer measurements can be used to estimate the its uniaxial compressive strength.

6. CONCLUSIONS

In this paper, the authors have presented an investigation on the scale effects on hard rocks properties. Different size core samples were collected in an underground mine and tested in uniaxial compression. Dilatometer measurements were also performed on site to evaluate the deformation modulus. The results showed a decrease in strength and with the deformation modulus when the sample size tested increases.

7. ACKNOWLEDGEMENTS

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