EVALUATION OF THE DEFORMATION MODULUS OF ROCK MASSES: COMPARISON OF PRESSUREMETER AND DILATOMETER TESTS WITH RMR PREDICTION

ÉVALUATION DU MODULE DE DÉFORMATION DES ROCHES: COMPARAISON ENTRE LES ESSAIS AU DILATOMÈTRE ET AU PRESSIOMÈTRE AVEC ESTIMATION DU RMR

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ABSTRACT

This paper presents the result of comparisons between the modulus of deformation obtained from dilatometer tests and the geomechanical quality of the rock mass using the RMR classification and the basic intact rock properties such as the uniaxial compressive strength and Young's modulus.

As result of several civil works constructed in the last ten years and also using data from bibliography, the data used in this study includes more than 700 results.

The first objective was to compare the dilatometer modulus with RQD and RMD including ISRM and Goodman lithological classifications of rock masses. Subsequently, it has been decided to use the RMR without considering the lithology, as the differences where found insignificant.

The second step was to scrutinize the data, excluding those with the following limitations:

- Weathering grade $\geq$ IV
- Dilatometer modulus $\leq$ 0,5 GPa

Also in those cases in which $E_m \leq 10$ GPa, 15 points were added to the value of RMR because an undrained modulus was being considered.

The third goal was to perform a sensitivity analysis of the data, using the following criteria:

- Comparison E^i *vs*. σ^i_c
- Comparison E^i *vs*. E_m, and
- E_m/E^i *vs*. RMR

Excluding any data with anomalous ratios, the final database consists of 436 cases in which known values of E_m, RMR, σ^i_c and E' are considered reliable.

With this database several correlations were investigated to estimate rock mass deformability improving on the existing criteria of Bieniawski (1978), Serafin-Pereira (1983), Nicholson-Bieniawski (1990) and Hoek (1995).

1. Introduction

The main purpose of this Key Note Address is to present a state of the art evaluation of the rock mass deformability and to present the results of comparisons between the modulus of deformation obtained from dilatometer and pressuremeter tests and the geomechanical quality of the rock mass, using the RMR classification. In addition, intact rock properties such as uniaxial compressive strength and Young's modulus are discussed.

The importance of the knowledge of the deformation modulus of the ground is well known for estimations by means of stress-strain analysis and has been studied in the past three decades (since Bieniawski, 1978).

In **Figure 1** an example of stress-strain calculations solved with FLAC code is shown.

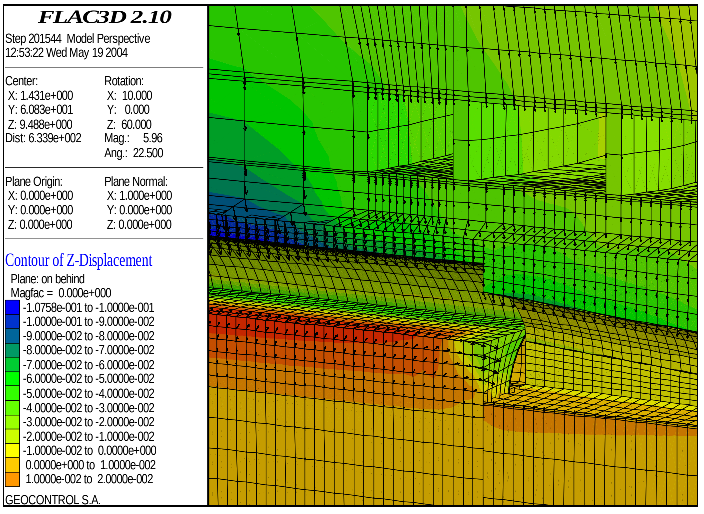


Figure 1. Example of a stress-strain numerical analysis (Bocabarteille *et al.*, 2000)

It depicts estimation of the convergence of a tunnel, constructed in several phases, and settlements induced in an existing structure above the tunnel.

In this case, an assumption of the stiffness of the ground is of paramount importance for the evaluation of these deformations.

As such data cannot be determined directly by laboratory tests, several approaches have been tried in order to estimate the properties of the rock mass in situ.

2. Approach to the problem: The concept of an equivalent elastic continuum

Deformability of a jointed rock mass is the result of the stiffness of the rock itself and the stiffness of the joints; the equivalent elastic continuum has the same deformation characteristics as the jointed rock mass. The **Figure 2** shows this concept.

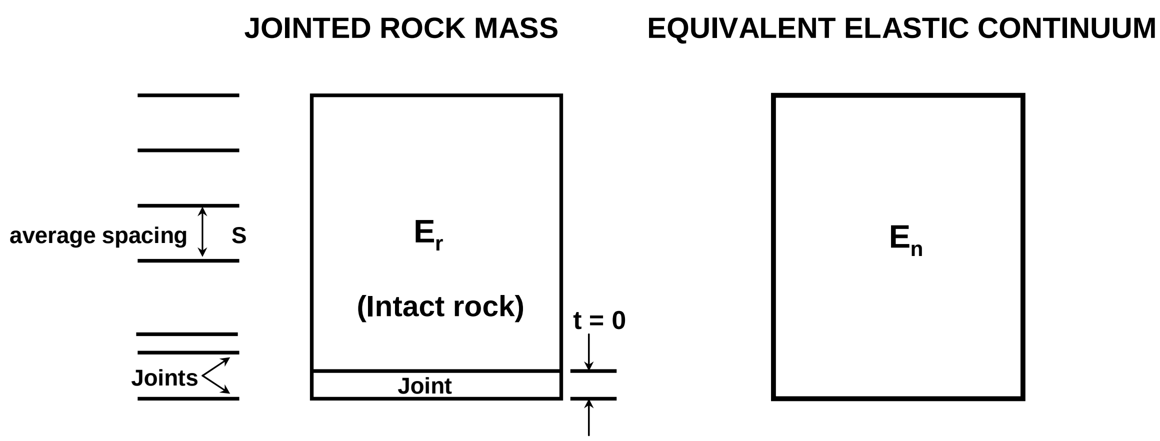


Figure 2. The concept of an equivalent elastic continuum versus a jointed rock mass.

Therefore,

$$u_{\text{jointed rock mass}} = u_{\text{equivalent elastic continuum}}$$

or

$$\frac{1}{Er} + \frac{1}{k_n s} = \frac{1}{En} \tag{1}$$

where E_r is the rock deformation modulus

k_n is the joint normal stiffness

s is the average joint spacing

E_n is the equivalent deformation modulus

In this line of reasoning an equivalent elastic continuum, several authors presented their findings such as Kulhawy (1978), shown in **Figure 3**, and Kulhawy and Goodman (1980). They proposed

$$E_m = j \cdot E_i \tag{2}$$

where E_m is the rock mass deformation modulus, j is the average joint spacing and E_i is the intact rock modulus.

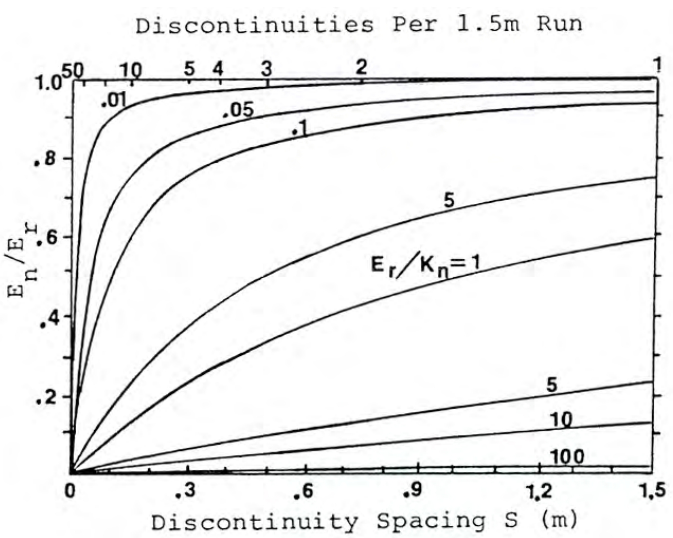


Figure 3. Modulus Reduction *vs.* Discontinuity Spacing (Kulhawy, 1978).

Also Bandis *et al.* (1983) discussed evaluation of the joint normal and shear stiffnesses.

A second approach was initiated by Bieniawski (1978), in which the evaluation of the rock mass deformation modulus was empirically estimated. This approach will be discussed later.

3. Some considerations of the scale effects in rock masses

One of the considerations of scale effects in rock masses was by Hoek and Brown (1980) where the strength of the rock mass was estimated by means of the value of the RMR.

Also, the ISRM organized a work group for the investigation of the scale effects in rock masses concerning strength, deformability, joint properties, permeability, and even in situ stresses.

Figure 4 shows the concept of this effect. The results of these studies were presented in two Workshops, at Loen (1990) and Lisbon (1993).

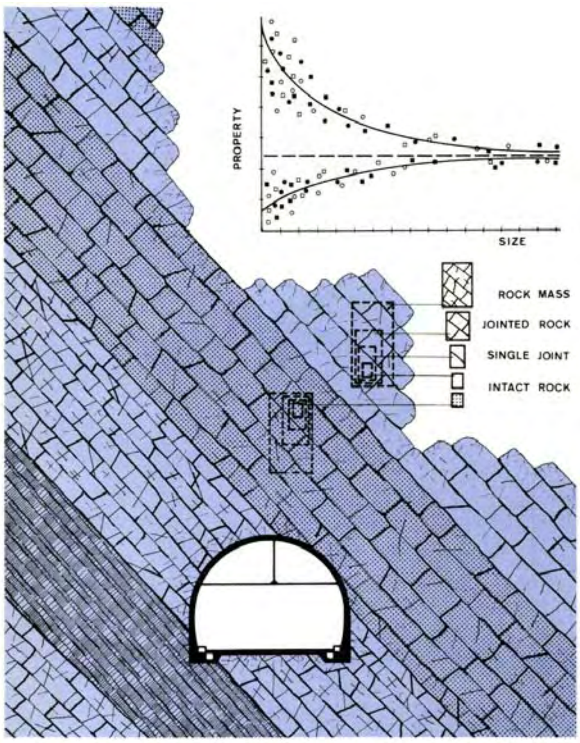


Figure 4. The concept of a rock mass and its scale effect (Pinto da Cunha, 1993).

In particular, concerning the scale effect in rock mass deformability, Pinto de Cunha and Muralha (1990) showed the effect of the volume involved in the test of the deformation modulus measured.

Figure 5 shows this phenomenon, where LAB are laboratory tests, BHD are Borehole Dilatometer Tests and LFJ are Large Flat Jack Tests. Two different ideas can be derived from this figure. One is that the bigger the volume involved in the test, the lower the modulus. The second is that the bigger the volume, the smaller the variability of the results.

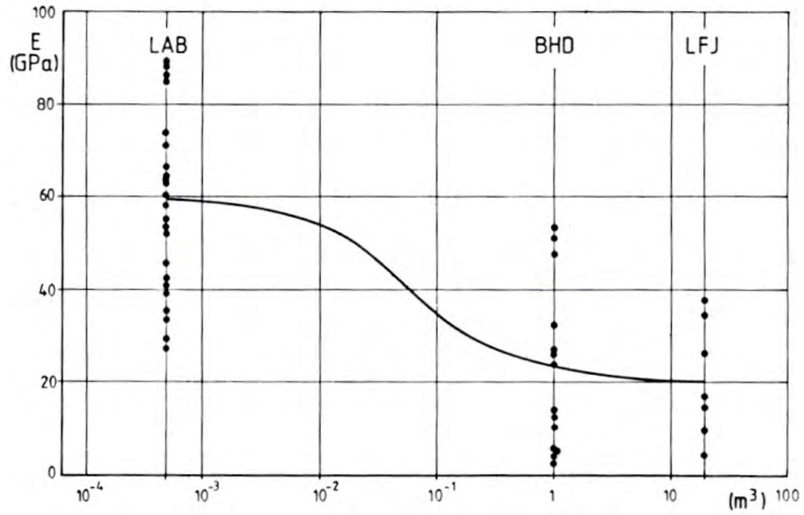


Figure 5. Deformability modulus *vs.* tested volume (Pinto da Cunha & Muralha, 1990).

Furthermore, He (1993) studied the influence of the test volume on deformation modulus. In **Table I and II** this scale effect can be seen using several in situ methods for its determination. These results are consistent with the earlier studies by Bieniawski (1979) of the influence of the various in situ deformability methods.

Table I. Scale of different in-situ methods (He, 1993).

Method	Range of test dimension (m)	Equation	Test volume (m^3)
Dilatometer	~0.07	$8.75\pi D^2 l$	0.13 (0.1)
Flat jack	0.2x0.3 – 1.0x1.0	$2[\pi(2D)^3]/3$	0.71-3.1 (1.0)
Plate loading	0.2 – 1.0	$4[\pi(2D)^3]/3$	0.27-33.5 (10)
Pressure Tunnel	2x5 – 5x30	$8.75\pi D^2 l$	550-20600 (1000)
Tunnel relaxation	--	--	(> 1000)

Table II. Deformation modulus by different methods (He, 1993).

Method	No.	Mean (GPa)	St.dev. (GPa)
Dilatometer	30	13.80	10.01
Flat jack	31	25.13	17.42
Plate loading	96	15.65	15.70
Pressure Tunnel	45	11.30	8.90
Seismic method	11	19.68	6.80
Tunnel relaxation	16	22.71	12.40

Pinto da Cunha and Muralha (1990) also made a comparison between the deformation modulus measured at laboratory and in situ by a large flat jack, versus joint intensity. The results are shown at **Figure 6**.

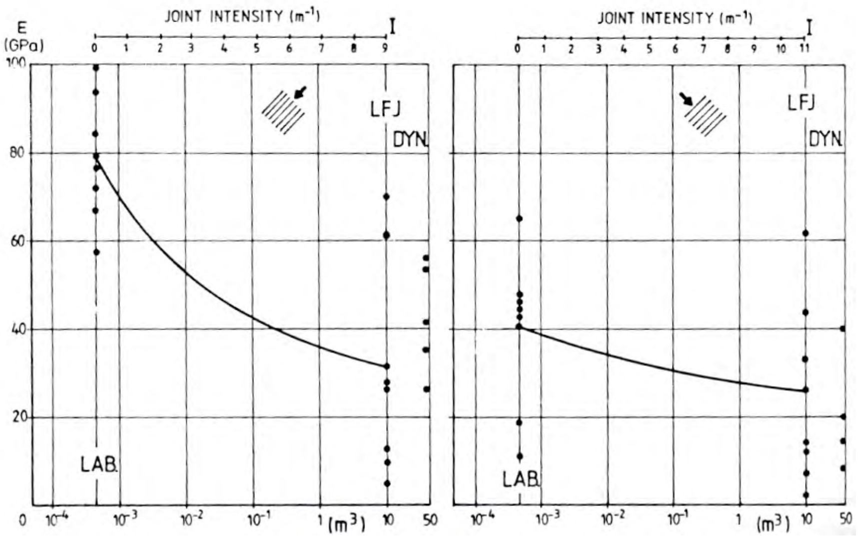


Figure 6. Deformability modulus *vs.* joint intensity and tested volumes

(Pinto da Cunha & Muralha, 1990).

4. Principal methods of in situ deformability determination

Several methods are available for in situ determination of ground deformability. These methods can be classified in three main groups:

- Borehole expansion tests.
- Geophysical methods.
- Plate load tests.

Borehole expansion tests are widely used with several devices:

- Ménard pressurementer.
- Flexible dilatometer.
- Self-boring pressuremeter.
- Full displacement pressuremeter.

All of the above four methods are under standardization by CEN (CEN, 2005).

According with these standards, for conventional rock masses, the flexible dilatometer is the best tool, while the other should be used for soft rocks only.

The **geophysical methods** for the in situ measurement of rock mass deformability are based on the measurement of the compressional and shear wave velocities, V_p and V_s, and constitution therefore the evaluation of dynamic elastic properties.

These moduli must be correlated with the elastic modulus. Several studies can be found in the bibliography about this correlation (Starzec, 1999 or Galera *et al.*, 2001).

Figure 7 shows the correlation between the modulus obtained by flexible dilatometer tests and the dynamic modulus obtained with a full wave sonic logging. A correlation can be seen:

$$G_d(\mathrm{MPa}) = 10.95 \cdot G_{stat}(\mathrm{MPa}) + 1449 \quad (3)$$

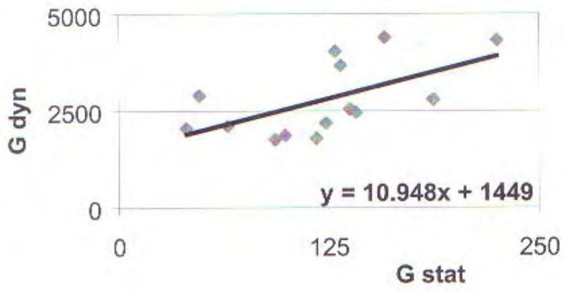


Figure 7. Correlation between dynamic and static shear modulus

(Galera, Peral, Rodríguez, 2001)

Plate loading tests also provide the evaluation of the reduction in modulus due to the scale effect in rock masses. Its use is restricted mainly due to high costs. Nevertheless it is used for some rock foundations as well as for dam site investigations.

Figure 8 shows an example of a plate loading test done at experimental gallery of Le Boulou (France) for the Perthus Tunnel (Celada *et al.*, 1997).

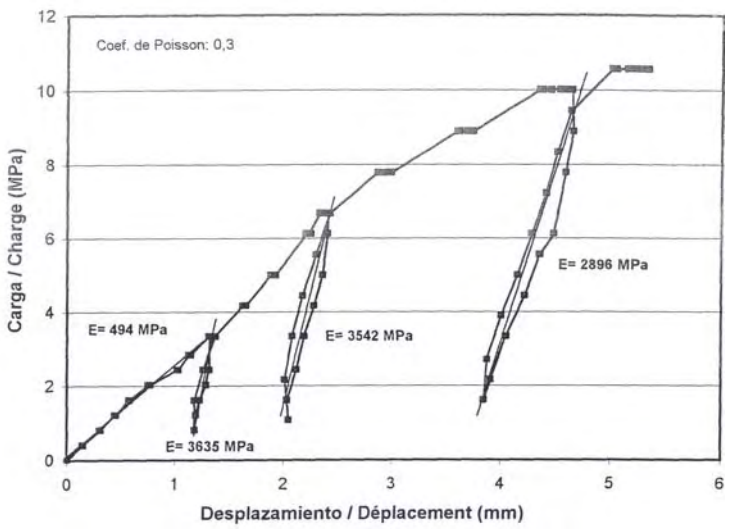


Figura 8. Plate load test in Montesquieu schist (Celada *et al.*, 1997).

It may be concluded that the dilatometer and pressuremeter tests are more preferable methods for the in situ determination of rock mass deformability.

5. Empirical evaluation from geomechanical classifications

It is clear that in situ methods are the best approach to predict deformability of rock masses. However, in situ tests are relatively expensive and not always provide reliable results due to several reasons.

Rock mass deformation modulus estimation by correlations with geomechanical classifications appeared as a traditional tool in rock mechanics since Bieniawski (1978) and his RMR index.

Subsequent correlations have included RQD (Gardner, 1987; Kayabasi *et al.*, 2003; and Zhang and Einstein, 2004), Q system (Barton, 1983; Grimstad and Barton, 1993), and RMR (Serafim and Pereira, 1983; Nicholson and Bieniawski, 1990; and more recently, Hoek *et al.*, 1995).

Currently, three different correlations using Q, RQD and RMR are used:

a) Q and rock mass deformation modulus

The literature about the correlation Q-E_m is not so abundant as with RMR. Nevertheless, Barton (1983) and Grimstad and Barton (1993) provided a study with several geophysical borehole measurements obtaining the following relations:

$Q = 10(V_p - 3.5)$ with Vp in km/s, and concluding

$$E(GPa) = 25\,LogQ \tag{4}$$

although in other projects $E = 10\,LogQ$ was found more suitable.

b) RQD and rock mass deformation modulus

Note that RQD has the limitation of being a rock **core** quality index and not a rock **mass** index like RMR or Q.

Gardner (1987) proposed the following expression,

$$E_m = \alpha_E \cdot E_i \tag{5}$$

where α_E = 0.0231·RQD-1.32 (≥0.15). This method was used by the AASHTO (American Association of State Highway and Transportation Officials).

More recently Zhang and Einstein (2004) recommended the following relations:

$E_m/E_i = 0.2 \cdot 10^{0.0186RQD-1.91}$ (Lower bound)

$E_m/E_i = 1.8 \cdot 10^{0.0186RQD-1.91}$ (Upper bound)

$E_m/E_i = 10^{0.0186RQD-1.91}$ (Mean)

These expressions are shown in **Figure 9**. Note the large scatter.

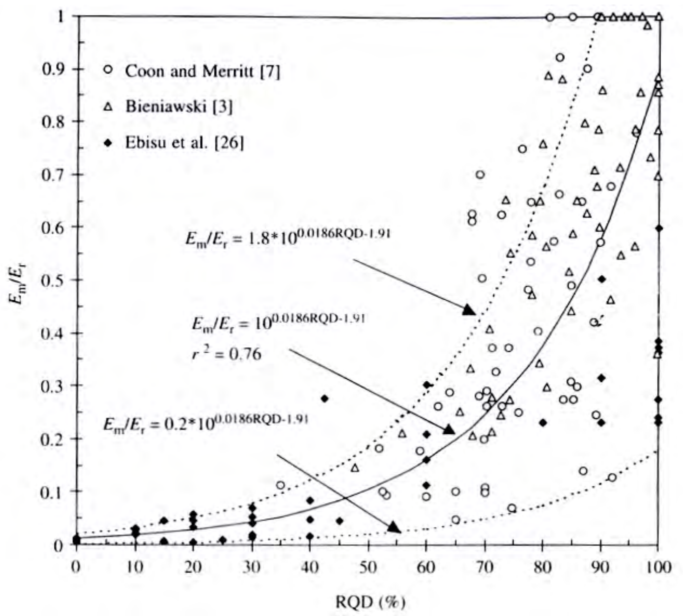


Figure 9. Relationships between RQD and E_m/E_r (Zhang and Einstein; 2004)

c) RMR and rock mass deformation modulus

The first correlation between RMR and rock mass deformation modulus was proposed by Bieniawski (1978), as

$$E_m(\text{GPa}) = 2 \cdot \text{RMR} - 100 \quad (\text{For RMR} \geq 50) \tag{6}$$

Later, Serafim-Pereira (1983) proposed the more known expression,

$$E_m(\text{GPa}) = 10^{\frac{(\text{RMR}-10)}{40}} \tag{7}$$

Figure 10 shows graphically both expressions and their comparison.

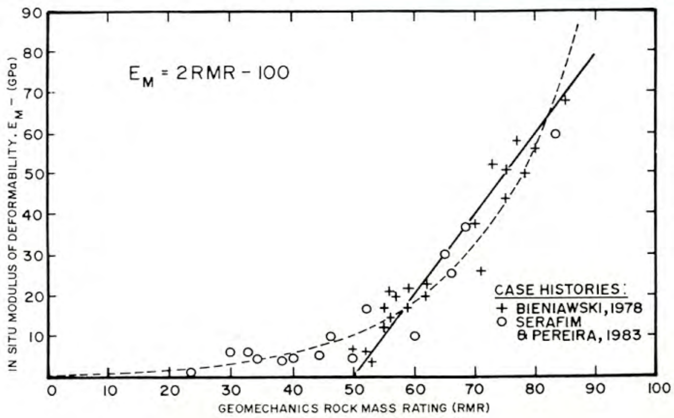


Figure 10. Correlation between the in-situ modulus of deformation and RMR (Bieniawski, 1989).

Nicholson and Bieniawski (1990) derived the following relation considering not only RMR but also the Young's modulus of the intact rock E_i:

$$\frac{E_m}{E_i} = \frac{1}{100} \cdot \left(0.0028 \cdot \text{RMR}^2 + 0.9^{\frac{\text{RMR}}{22.82}} \right) \tag{8}$$

Finally Hoek *et al*. (1995) suggested a correction to the Serafim-Pereira expression, using a factor of $\sqrt{\frac{\sigma_c^i(\text{MPa})}{100}}$, and interchanging GSI (Geological Strength Index) with RMR, as follows

$$E_m(\text{GPa}) = \sqrt{\frac{\sigma_c^i(\text{MPa})}{100}} \cdot 10^{\frac{(\text{GSI}-10)}{40}} \qquad (9)$$

Figure 11 shows graphically this Hoek *et al.* (1995) relation.

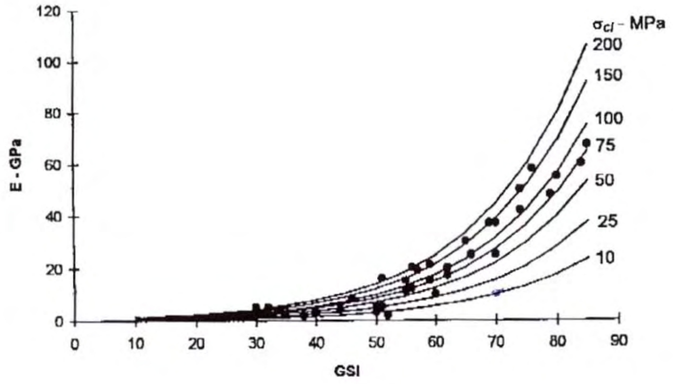


Figure 11. Proposed relationships between GSI or RMR with the intact rock strength (σ_{ci}) and in situ modulus of deformation E_m (Hoek *et al.*, 1995).

The use of RMR and not GSI is strongly recommended because GSI introduces more empirism in a classification that itself is empirical, as was stated in a recent review by Palmström (2003) who warned as follows *"Visual determination of GSI parameters represents the return to quality descriptions instead of advancing quantitative input data as in RMR, Q and RMi systems. GSI was found mainly useful for weaker rock masses with RMR<20.*

As GSI is used for estimating input parameters (strength), is is only an empirical relation and has nothing to do with rock engineering classification".

6. New correlations between RMR and rock mass deformation modulus

6.1. Database

The information presented here is derived partially from bibliography (Bieniawski, 1978; Serafim-Pereira, 1983; and Labrie *et al.* (2004)) but mainly from pressuremeter and dilatometers measurements made by Geocontrol during the last decade.

The amount of available data classified by its lithology is the following:

- igneous rocks: 270
- metamorphic rocks: 108
- detritic sedimentary rocks: 175
- carbonate sedimentary rocks: 101
- bibliography: 48

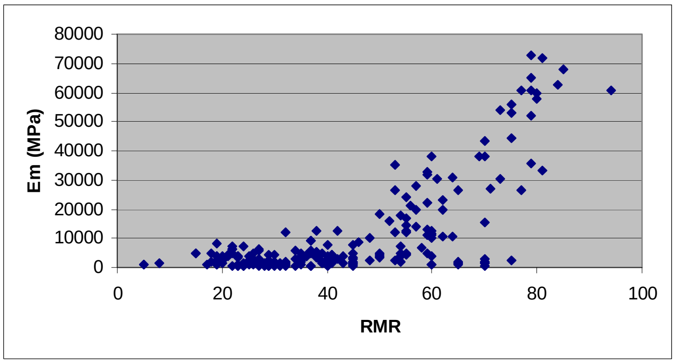
This represents 702 data in which the E_m from in situ tests, RMR and RQD are known.

In 123 of these data also the uniaxial compressive strength (σ_c^i) and Young's modulus of the intact rock (E_i) are also known.

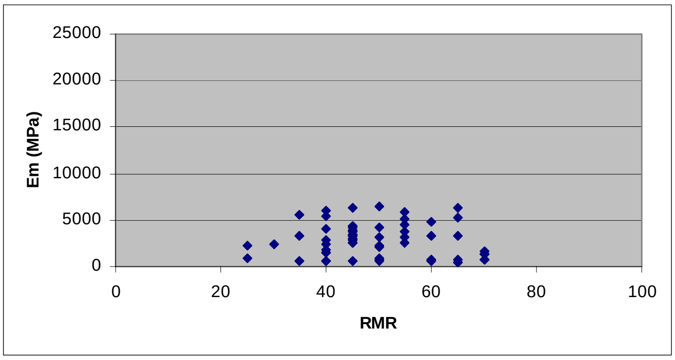
Figures 12.a, b, c and d show the available data classified by the lithology. This classification is based on the ISRM and Goodman lithological classifications of rock masses.

Figure 13 shows all the data jointed in the same graph and it can be observed that the differences due to the lithology are insignificant.

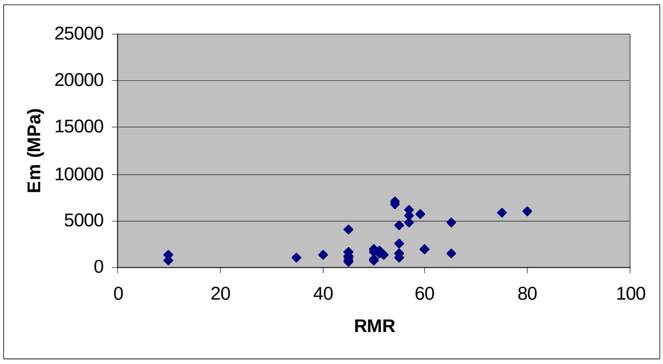
The first objective has been to compare the pressuremeter and dilatometer results, which represents the rock mass modulus E_m, with RQD and RMR. In **Figure 14.a and b** both comparison are shown.



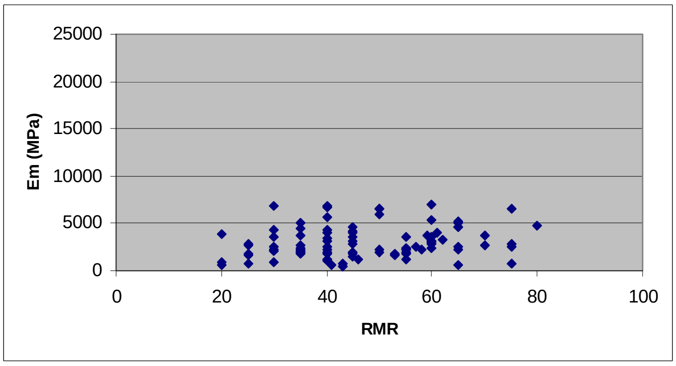
(a) Igneous rocks



(b) Metamorphic rocks



(c) Sedimentary carbonate rocks



(d) Sedimentary detritic rocks

Figure 12. Database according to the lithology.

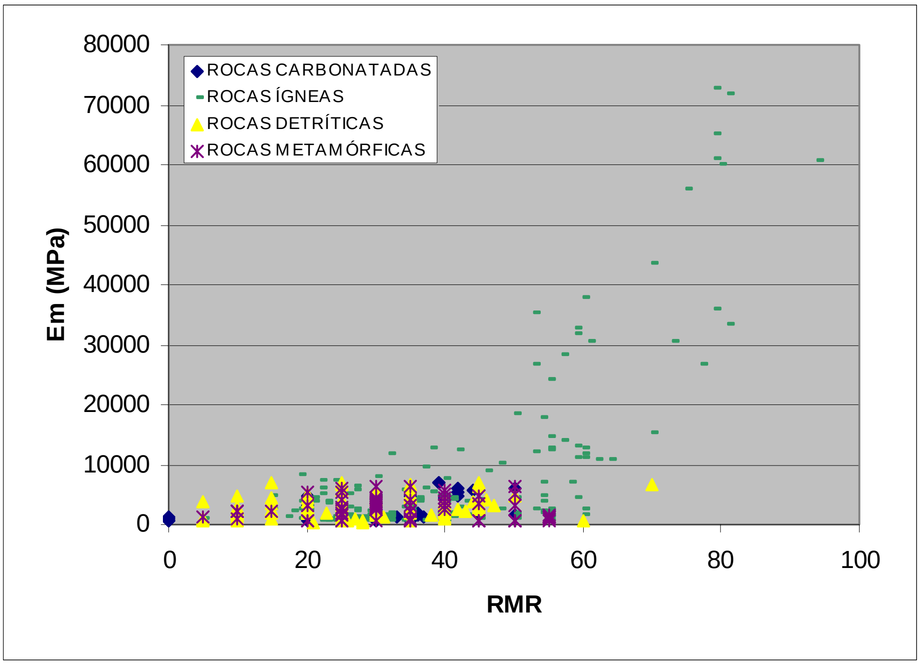
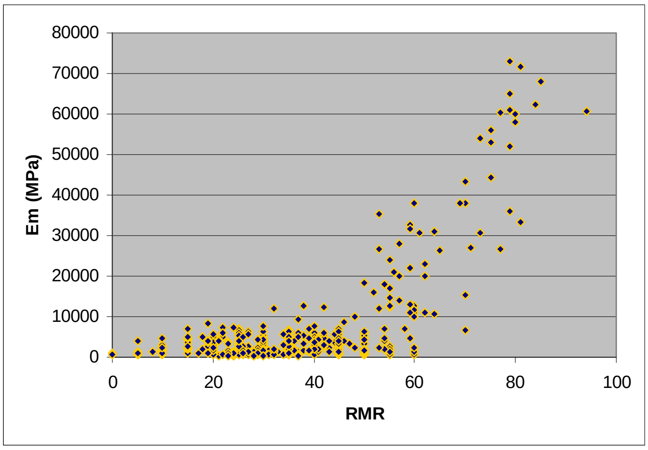
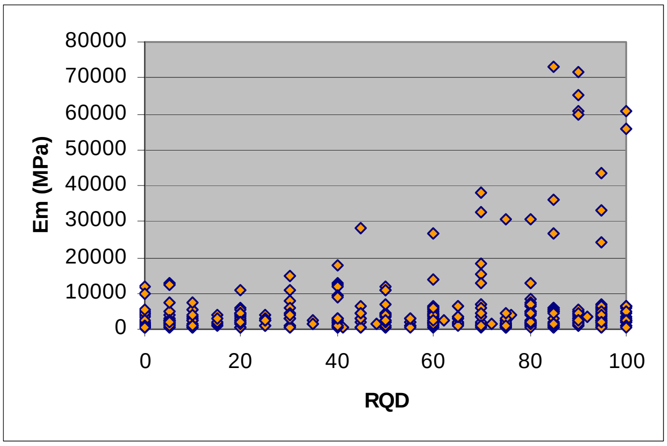


Figure 13. Relation Em (MPa) *vs.*RMR according to the lithologies.



(a) Modulus of deformation *vs.* RMR



(b) Modulus of deformation *vs*. RQD

Figure 14. Comparison of the E_m versus RMR and RQD.

It is evident that RMR provides a better trend of the data, since RQD is only one of the five major components of the RMR classification.

This figure clearly shows that RMR is more reliable to estimate the deformation modulus than RQD alone by providing a lesser scatter of data.

6.2. Analysis of the data

The second step has been to scrutinize the data, excluding those with the following limitations:

- Weathering grade bigger or equal than IV.
- Pressuremeter or Dilatometer modulus lesser or equal than 0.5 GPa.

The reason for this filter is to remove data with a “soil behaviour” in which the application of RMR classification is inappropiate as not constituting a “conventional” rock mass.

Also in those cases in which $E_m \leq 10$ GPa, 15 points were added to the value of RMR because a drained modulus was considered.

Celada *et al.* (1995) analyzed the relation between drained and undrained modulus as:

$$\frac{E_u}{E} = \frac{3(1-\nu)Kw + E \cdot n}{2(1+\nu)(1-2\nu)Kw + E \cdot n} \quad (10)$$

where Kw is the balk modulus of the water and n is the porosity.

Figure 15 shows this relation. It can be concluded that:

- If E is bigger than 10 GPa, $E_u/E \simeq 1$ and no significant difference exists between both modulus.
- If E is smaller than 10 GPa and with a drained Poisson's ratio of 0.3, $E_u/E \simeq 1.15$, so the undrained modulus is around 15% higher than drained modulus.

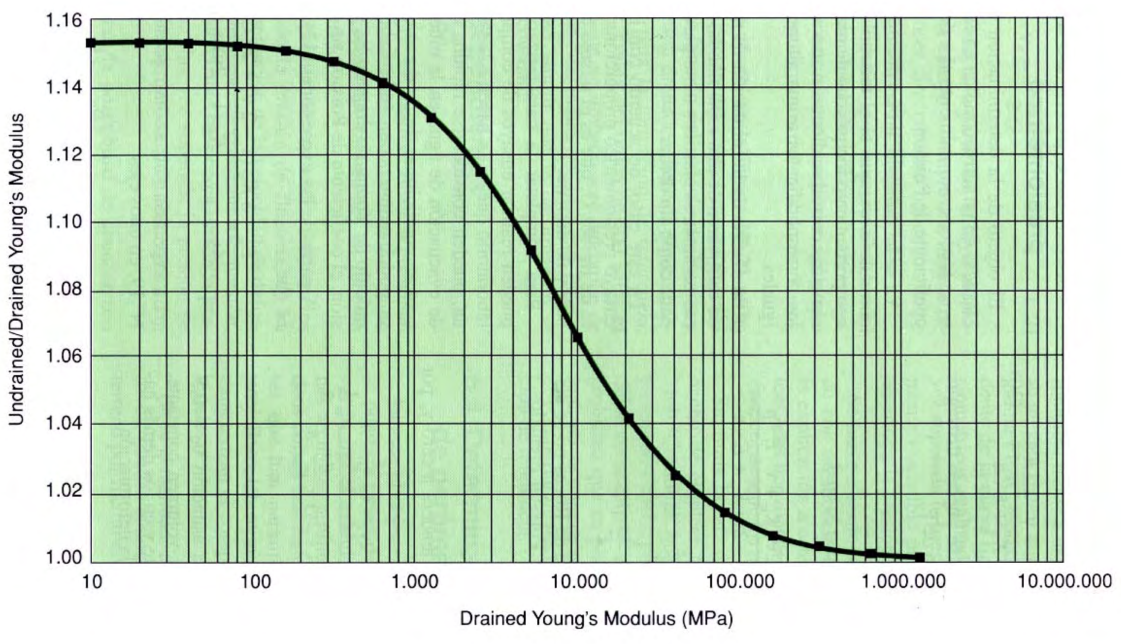


Figure 15. Comparison between Undrained and Drained Modulus (Celada *et al.*, 1995).

Finally, the third step has been to perform a sensitivity and quality analysis of data, using the following criteria:

- comparison E_i *vs.* σ_c^i
- comparison E_i *vs.* E_m, and
- comparison E_m/E_i *vs.* RMR.

Excluding any data with anomalous ratios, the final database consists of:

- 427 cases in which E_m and RMR are considered reliable.
- 98 cases in which E_m, E_i, σ_c^i and RMR are considered reliable.

6.3. Discussion

With these data, several correlations have been investigated to estimate rock mass deformability by improving on the existing relationships described in item 5.

Experience shows that with the current correlation usually the deformation modulus E_m estimated is higher than the modulus measured by means of borehole expansion tests such as pressuremeters and dilatometers.

Two new different types of relations are proposed:

- without considering E_i values
- including E_i values

In the first case also the values of σ_c^i are included using this expression:

$$\sigma_m = \sigma_c^i \cdot e^{(RMR-100)/24} \quad \text{(Kalamaras and Bieniawski, 1995)} \qquad (11)$$

In all the cases the coefficient of regression r^2 has been calculated as follows:

$$r^2 = 1 - \frac{\sum (y_i - y_i')}{\sum (y_i - \bar{y})^2} \tag{12}$$

where y_i is the value of Em, $\bar{y}$ is the mean and y_i' are the adjusted values.

a) New correlations between RMR and rock mass deformation modulus

Figure 16 shows all the new correlations considered and also the Serafim-Pereira expression.

(i) considering $\sigma_m = \sigma_c^i \cdot e^{(RMR-100)/24}$ it is derived:

$$E_m(GPa) = 147.28 \frac{\sigma_m}{\sigma_c^i} - 0.202 \cdot RMR \tag{13}$$

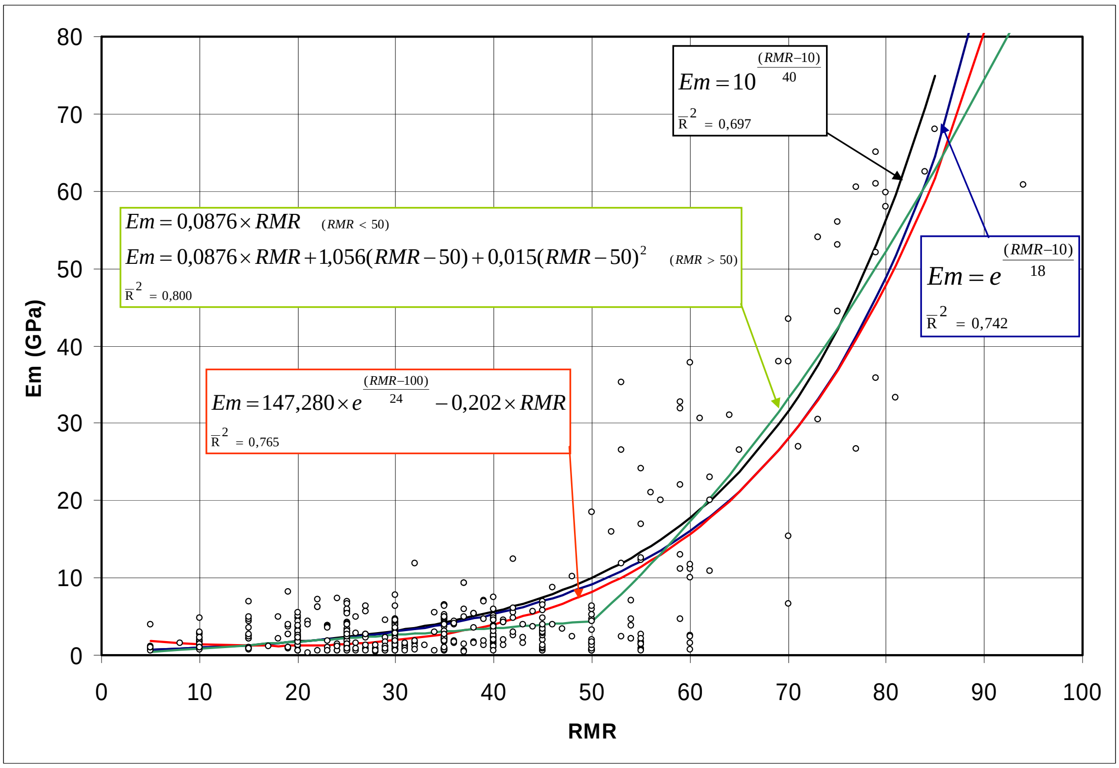


Figure 16. New correlations between RMR and rock mass deformation modulus E_m.

The coefficient of regression, r^2, obtained is 0.765, that is higher than the one obtained in the regression of the data following Serafim-Pereira, namely, a $r^2 = 0.697$.

(ii) The second relation is an improvement of Serafim-Pereira, as follows:

$$E_m = e^{(RMR-10)/18} \tag{14}$$

The coefficient of regression $r^2 = 0.742$, that improves by 10% the estimation of E_m.

(iii) Finally, following the original estimation, a threshold of RMR = 50 is derived:

$$E_m(GPa) = 0.0876 \cdot RMR \qquad \text{for } RMR \geq 50 \tag{15}$$

$$E_m(GPa) = 0.0876 \cdot RMR + 1.056(RMR - 50) + 0.015(RMR - 50)^2 \qquad \text{for } RMR > 50 \tag{16}$$

This above correlation gives a coefficient of regression $r^2 = 0.8$, that improves by more than 15% the estimation of Serafim-Pereira.

b) New correlation between RMR and rock mass deformation modulus including E_i

Figure 17 shows the relation $E_m = E_i \cdot e^{(RMR-100)/36}$.

The coefficient of regression, r^2, is 0.656 which is smaller than that given by the previous correlations but it makes a more reliable estimation as E_i is considered and improves by almost 40% the estimation due to Nicholson and Bieniawski (1990) which gives r^2=0.472.

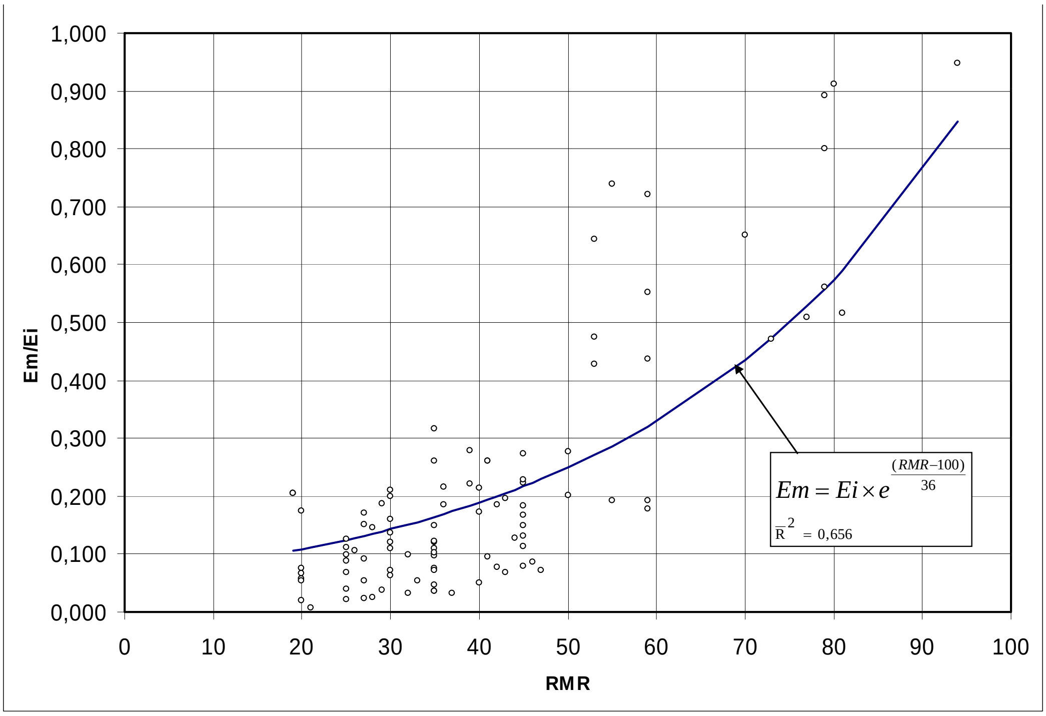


Figure 17. Correlation between RMR and rock mass deformation modulus ratio including E_i.

7. Summary and conclusions

(1) The Borehole Expansion Tests, mostly Flexible dilatometers, were found to be the best in situ test for the determination of the rock mass deformation modulus.

(2) The empirical E_m – RMR correlations present a smaller scatter than the previous correlations E_m – RQD.

(3) Several empirical correlations have been studied to estimate rock mass deformation modulus E_m. Most of them provide an overestimation of the value E_m.

(4) Considering 427 data collected from the published literature and our own data, the best coefficient of regression is obtained considering a threshold of RMR = 50. A linear regression is suggested for values smaller or equal to 50, while a polynomial expression is recommended for values of RMR bigger than 50.

(5) A new relation between RMR and Em/Ei is recommended, considering 98 data. This expression is

$$E_m = E_i \cdot e^{(RMR-100)/36} \qquad (17)$$

representing a useful tool for estimation of the rock mass deformation modulus.

Considering that rock mass strength $\sigma_m = \sigma_c^i \cdot e^{(RMR-100)/24}$ and equation (17), it results in the following expression:

$$\frac{E_m}{E_i} = \left(\frac{\sigma_m}{\sigma_c^i}\right)^{2/3} \qquad (18)$$

providing another useful relationship for rock mass characterization.

(6) Nevertheless, the presented correlations should be used realizing that some factors are ignored such as directional effect of jointing.

8. Bibliography

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