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THE DEFORMATION MODULUS OF ROCK MASSES
- comparisons between in situ tests and indirect estimates

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SUMMARY

Three methods of in situ deformation modulus (Em) measurements of rock masses have been described, analysed and compared. The plate jacking (PJT) test, where the deformations are measured by extensometers in drill holes, gives generally the best results. A factor of 2.5 has been found between PJT and the Goodman jack test and the plate loading test. From analyses of the results it has been pointed out that the damage from blasting of the test adit reduces the magnitude of test results with a factor between 2 and 4.

The existing equations for indirect estimates of the rock mass deformation modulus from classification systems have been analysed and adjustment suggested. Taking into consideration the uncertainties connected to in situ deformation measurements caused by blast damage, test procedure, and test method, a good characterization of the ground may give comparable or possibly better Em values using the RMi or the RMR system than the in situ tests. The RMR system gives, however, too high values for Em in massive rock.

1. INTRODUCTION

The static modulus of deformation is among the parameters that best represent the mechanical behaviour of a rock and of a rock mass, in particular, when it comes to underground excavations. This is why most numerical finite element and boundary element analyses for studies of the stress and displacement distribution around underground excavations are based on this parameter. The deformation modulus is therefore a cornerstone of many geomechanical analyses.

All *in situ* measurements of the static modulus of deformation used today are time-consuming and imply notable costs and operational difficulties. Because of this, the deformation modulus is often estimated indirectly from classification systems. In other cases the modulus is assumed based on the experience of the engineering geologist or from literature data.

The aim of this paper is to outline some aspects of field deformation measurements, and, from results of these, to review indirect estimates based on descriptive systems for characterization or classification of rock masses. Results from several in situ deformation tests have been analysed and compared with the rock mass conditions at each site. The test results have also been compared with deformation values estimated from rock mass classification systems and from the rock mass index (RMi), a system for characterization of rock masses. The Central Soil and Materials Research Station (CSMRS), New Delhi has performed most of the tests in India, Bhutan and Nepal.

The existing indirect expressions to estimate the deformation modulus have also been reviewed and compared. It is hoped that the conclusions drawn can help in arriving at more accurate estimates and usage of the modulus of deformation of rock masses.

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2. ON DEFORMATION AND ELASTIC MODULI

2.1 Definitions

Deformability is characterized by a modulus describing the relationship between the applied load and the resulting strain. The fact that jointed rock masses do not behave elastically has prompted the usage of the term modulus of deformation rather than modulus of elasticity or Young's modulus. The commission of terminology, symbols and graphic representation of the International Society for Rock Mechanics (ISRM) has given the following definitions (ISRM, 1975):

Modulus of elasticity or Young's modulus (E): The ratio of stress to corresponding strain below the proportionality limit of a material. Values of E for various rocks are shown in Table A-1 in the Appendix.

Modulus of deformation of a rock mass (Em): The ratio of stress (p) to corresponding strain during loading of a rock mass, including elastic and inelastic behaviour (w_d), as shown in Figure 1.

Modulus of elasticity of a rock mass (Eem): The ratio of stress (p) to corresponding strain during loading of a rock mass, including only the elastic behaviour (w_e), see Figure 1.

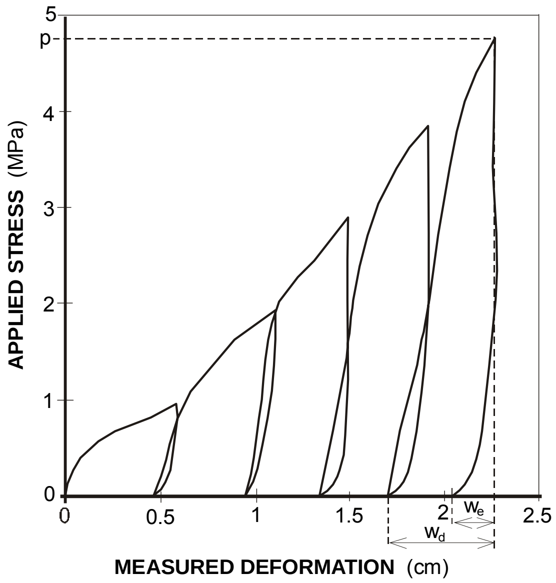


Figure 1: Typical stress versus deformation curve recorded in a deformability test of a rock mass (measured at Tala Hydropower project, Bhutan)

2.2 Earlier experience

Bieniawski (1989) wrote that "*Unfortunately, few projects to date have featured a sufficient number of different tests to allow a meaningful comparison of in situ test data. Very different in situ results may be obtained depending on the test method. Under these circumstances, it is not helpful to discuss the precision of in situ methods. Even in an extensive in situ test program in fairly uniform and good quality rock mass conditions, deformability data may feature a deviation of 25% or as much as 10 GPa for an average in situ modulus of 40 GPa. The tests involving full scale prototype behaviour (tunnel relaxation) give different results by comparison with other in situ tests. The choice of the design value for the in situ modulus of deformation thus becomes a matter of engineering judgement. This means that it is difficult to rely on any one in situ method alone; two or more methods should be used to crosscheck the results.*"

As is also stated by several authors, it is known in virtually all methods of field modulus measurement or estimation that they give values which vary from laboratory values by significant amounts due to jointing in rock mass. For instance, Farmer and Kemeny (1992) found that the deformation modulus on intact rock samples is in the order of 5 to 20 times higher than in situ values. The variation in blockiness or degree of jointing in rock masses may often cause a major part of this large variation. Part of it may also occur from changes in test boundary conditions, from poor test design or incorrect analysis as is discussed in this paper.

Several investigations point out that the in situ deformation modulus (Em) is not constant, but depends on the stress conditions, being generally higher in areas subjected to high rock stresses than in rock masses under low stresses. However, this may also be due to better rock mass quality where the higher stress occurs.

In addition, different equipment and techniques used to arrive at the design modulus value of rock masses give often different results. As later described in Sections 5 and 6, some of this can be explained by different procedures, preparation work of the test site, measurement accuracy, blasting damage of the test adit, etc. Thus Clerici (1993) concluded that "*when the value of the modulus of deformation is determined, even by direct measurement, the aim cannot be to define an absolute value, but rather to define a magnitude for the modulus"*.

A source to create confusion about the measurements and values of deformation modulus comes from poor definition or wrong term for the type of test performed. The *plate loading test* (PLT) and *plate jacking test* (PJT) are especially sensitive to this, as both are often named *plate bearing test*. But only PJT uses extensometer measurements in bore holes. This is further described in Section 3.

3. IN SITU MEASUREMENTS OF DEFORMATION MODULUS

All in situ deformation tests are expensive and difficult to conduct. They are mostly conducted in special test adits or drifts excavated by conventional drill and blast having a span of 2 m and a height of 2.5 m. The length of such adits varies with local conditions from some ten metres to several hundred metres. Initial preparations at each test site are particularly time consuming. The interpretation of measured in situ data is another difficult aspect, which requires experience from those involved.

Today, the following three types of in situ tests are mostly used to determine the modulus of deformation:

1. <u>Plate jacking tests (PJT)</u>
   Two areas diametrically opposite in the test adit are loaded simultaneously, for example using flat jacks positioned across the test drift as shown in Figure 2, and the rock displacements are measured in boreholes behind each loaded area.
2. <u>Plate loading tests (PLT)</u>
   While the PJT records the displacements in drill holes beyond the loading assembly of flat jacks, the PLT measures the displacements at the loading surface of the rock, as shown in Figure 2.
3. <u>Radial jacking tests (Goodman jack test)</u>
   The Goodman jack consists of two curved rigid bearing plates of angular width 90^o which can be forced apart inside an NX size bore hole by a number of pistons. Two transducers mounted at either end of the 20 cm long bearing plates measure the displacement.

In addition to these three types the following in situ deformation tests can be used:
- flat jack tests;
- cable jacking tests;
- radial jack tests;
- dilatometer tests;
- pressure chamber.

The effect of Poisson's ratio is one of the parameters used for the calculation of modulus value in an in situ test. Sharma and Singh (1989) found that it is not much variation in the values of the deformation modulus if the value of the Poisson's ratio is between 0.1 and 0.35.

The modulus value increases with the increase in applied pressure during the measurement. This is due to the closure of cracks or joints in the rock mass under stress, making the material stiffer at higher stresses. The first cycle should never be considered from the determination of the modulus values as most of the closure of joints takes place during this process.

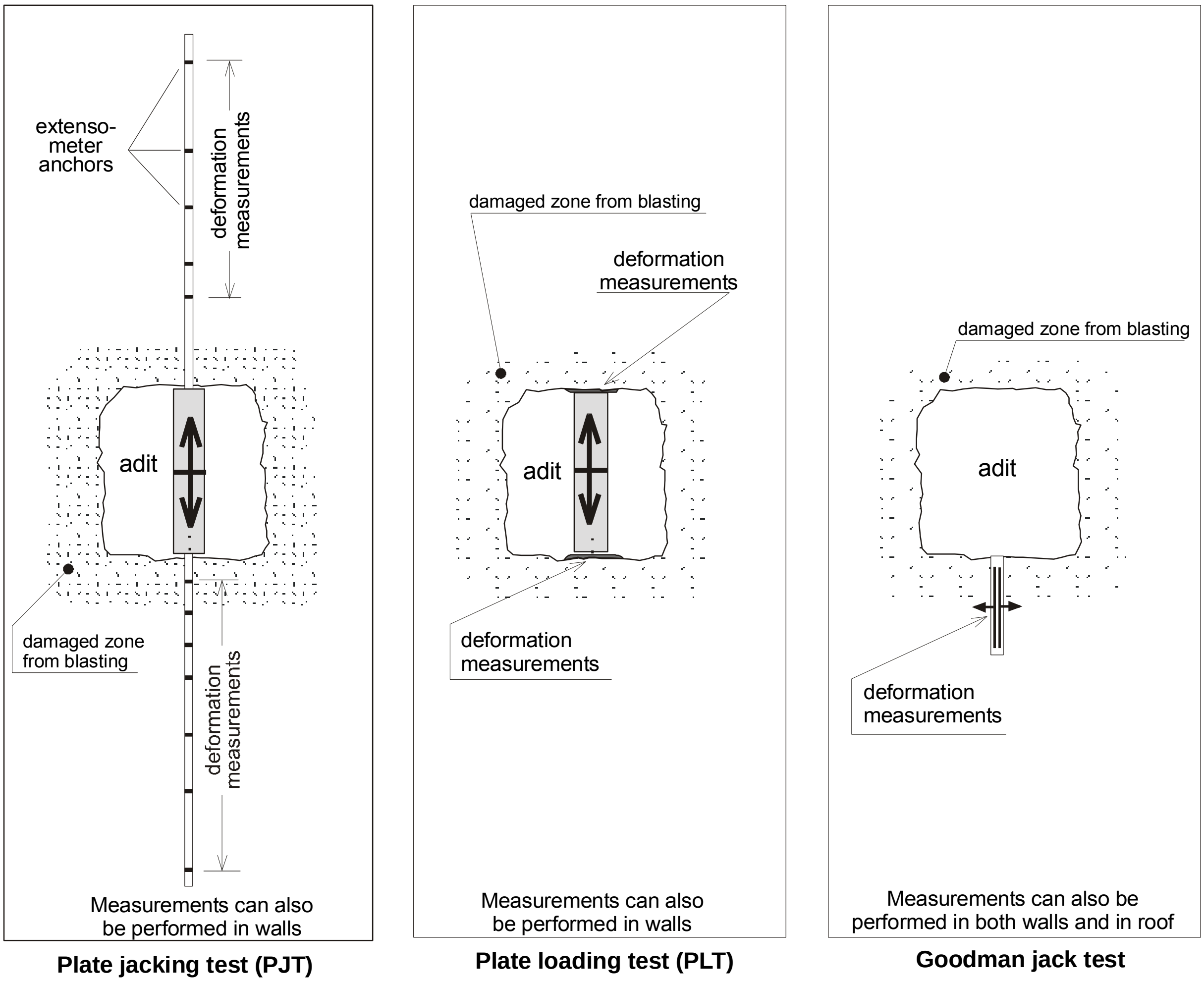
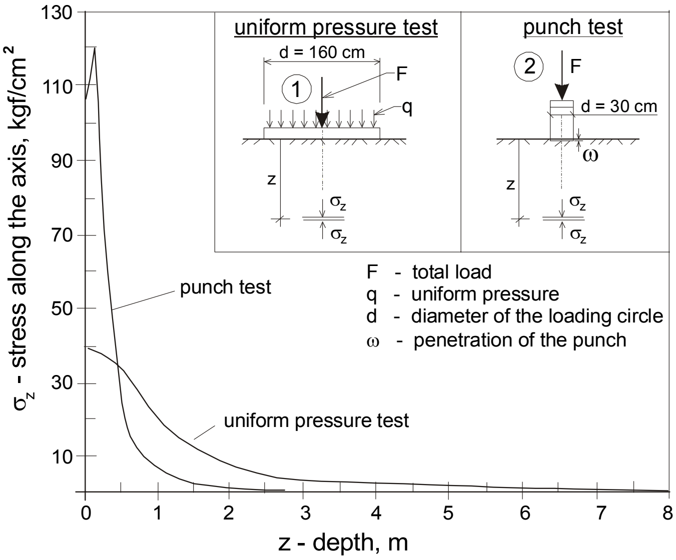


Figure 2: Principles of three main methods for in situ deformation measurement.

For the PJT tests it has been found that for 5 MPa to 6 MPa applied pressure, very small or no displacements are observed below 2.5 m, i.e., 3 times the diameter of the flat jack. This has earlier been shown by Serafim and Guerreiro (1968), see Figure 3 both for small rigid loading plates with diameter 0.3 m (punch tests) and large plates with diameter 1.6 m (uniform pressure tests). For 0.8 m diameter loading plate in the PJT the approximate reduction in stress along the bore hole is estimated as shown in the table of Figure 3.



DEPTH (z)	STRESSES (σ_z)		
	Test results of Serafim and Guerreiro		Assumed for the 0.8 m loading plate of the PJT
	uniform pressure test	punch test	
0 m	100%	100%	100%
0.5 m	85%	121%	80%
1 m	52%	24%	40%
2 m	37%	1.5%	15%
4 m	8%	0.5%	3%

Figure 3: The distribution of stresses along beyond the loading plates (from Serafim and Guerreiro, 1968, with modification of the table; the stresses in the right column are assumed).

4. INDIRECT ESTIMATES OF THE DEFORMATION MODULUS

Caused by the high cost and often measurement difficulties of in situ tests, the value of the modulus of deformation is often estimated indirectly from observations of relevant rock mass parameters that can be acquired easily and at low cost. These parameters are then applied in approximate equations, such as:

$Em = 2RMR - 100$	for RMR > 50	[Bieniawski, 1978]
$Em = 10^{(RMR - 10)/40}$	for RMR < 50	[Serafim and Pereira, 1983]
$Em = 25 \log_{10} Q$	for Q > 1	[Grimstad and Barton, 1993]
$Em = E_{r\ stat} \times Em_{dyn} / E_{r\ dyn}$		[Clerici, 1993]
$Em = 5.6\ RMi^{0.375}$	for RMi > 0.1	[Palmström, 1995]
$Em = \sqrt{\frac{\sigma_c}{100}} \times 10^{\left(\frac{GSI-10}{40}\right)}$	for σ_c < 100 MPa	[Hoek and Brown, 1998]

Here,
- Em = Modulus of deformation of rock mass (in GPa)
- RMR = Rock Mass Rating system (Bieniawski, 1973)
- Q = Q system (Barton et al., 1974)
- σ_c = Uniaxial compressive strength (in MPa) of intact rock measured on 50 mm diameter samples
- RMi = Rock Mass index (Palmstrom, 1995)
- GSI = Geological Strength Index (Hoek and Brown, 1998)
- $E_{r\ dyn}$ = Dynamic elasticity modulus of intact rock
- $E_{r\ stat}$ = Static elasticity modulus of intact rock
- Em_{dyn} = Dynamic in situ deformation modulus

The RMR, Q, and RMi systems are briefly described in Appendix B.

The use of more than one indirect procedure has been proposed by many authors, so that the results obtained can be compared and their reliability checked. The RMR system has probably been applied most frequently for deformation modulus estimates, see Figure 4. Clerici (1992) found that the equation developed by Serafim and Pereira (1983) gave values less than ± 15% from values measured in situ.

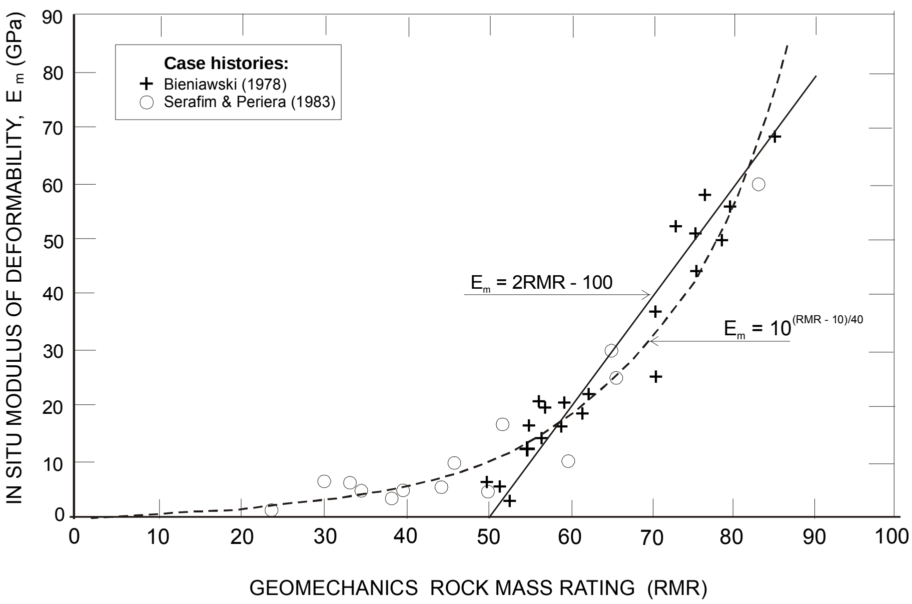


Figure 4: Correlation between the in situ modulus of deformation and the RMR system (from Serafim and Pereira, 1983)

The indirect procedures to estimate the deformation modulus are simple and cost-effective, especially as compared with the in situ tests. The latter should, however, be used whenever time and means available allow for them. As dealt with in Section 6.2, also the in situ measurements are prone to errors.

5. DAMAGE IN THE TEST ADITS FROM EXCAVATION BLASTING

5.1 The damaged zone in tunnels and adits

The rock mass modulus of deformation in test drifts has been found to vary considerably between the two walls and between roof and floor. Such differences may largely be due to blast damage caused by the excavation process as described by Singh and Rajvansi (1996) and by Singh and Bhasin (1996). The damage is mainly caused by development of cracks, displacement along existing joints, and disturbance of stresses. The effect of the blasts will vary with several features, such as rock properties, the amount of explosive used, the distance between the blast holes and the number of holes initiated at the same time, etc.

The zone around the tunnel influenced by blasting consists of two main types:

1. *The damaged* zone, close to the tunnel surface, is dominated by changes in rock properties, which are mainly irreversible. It includes rocks in which new cracks have been created, see Figure 5, existing cracks have been extended, and displacements along cracks have occurred.
2. The *disturbed* zone occurs beyond the damaged zone, in which the changes are dominated by changes in stress state and hydraulic head. Here, the stress redistribution will cause block movements, aperture changes on natural joints, and/or elastic deformation of the rock. The changes from blasting in material properties, such as seismic velocity, Young's modulus, etc. are expected to be insignificant.

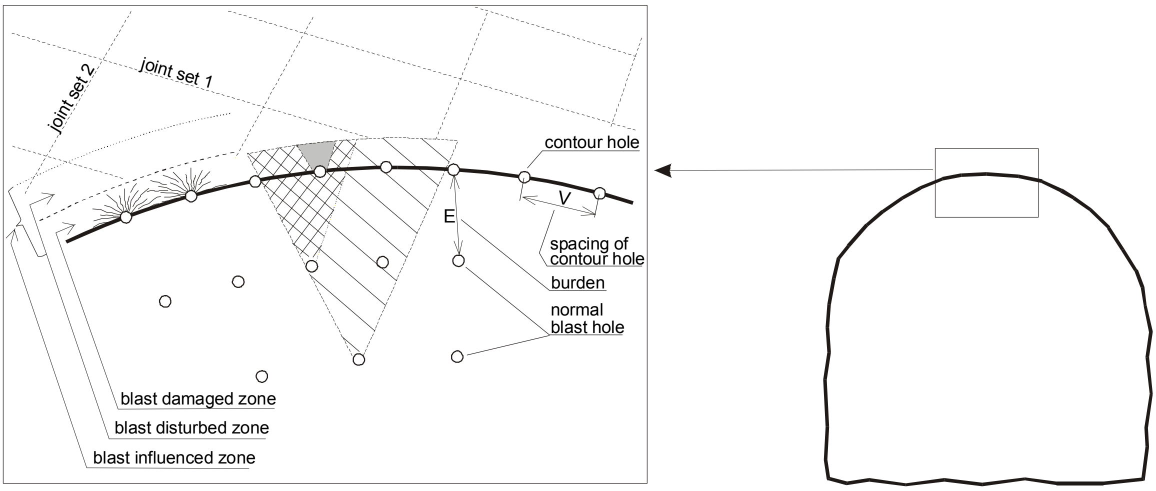


Figure 5: Damage zones from different hole rows (based on Holmberg et al., 1979). Reduced charges in the contour holes create a smaller damage zone than the normal blast holes. Also the burden and the charges in the holes nearest to the contour holes are important features in cautious blasting to reduce blast damage.

Generally, there is no distinct boundary between the two zones as the changes in rock properties and rock stress with distance from the rock wall of the excavation are gradual. In addition to the damaged and disturbed zone, a redistribution of stresses takes place during the excavation of a test adit. Thus, there is a compressive layer around the adit, which in addition to the stress anisotropy, may vary according to the blast damage and the rock preparation prior to the deformation test.

Various investigations of the damaged zone have been presented:

- Russian test blasting (Vovk et al, 1974) of single holes in large granite blocks using TNT explosives found occurrence of:
  - crushed zone approx. 5 to 17 times the bore hole diameter
  - fractured zone approx. 12 times the bore hole diameter
  - induced zone approx. 25 to 35 times the bore hole diameter

- At the Underground Research Laboratory (URL) in Canada it was found that the tunnel floor was more damaged than the rock in the wall and roof, and that the extent of damage here was at least 1 metre, which was attributed to a higher charge density and explosive energy used in the blast holes at the floor (Martin et al., 1990).

- For the ZEDEX (Zone of Excavation Disturbance Experiment) at the Aspö test site, Sweden it was found that the blast damage was 0.3 – 0.4 m in the floor and 0.05 – 0.15 m in the walls of the tunnel (Emsley et al., 1997). Here cautious blasting was applied in the roof and walls with reduced charges in the contour and in the holes nearest to the contour holes. The larger damage in the floor was explained by a higher charge in the holes.

5.2 Probable blast damage in test adits and its effect on the deformation modulus

The test adits for deformation tests are most often excavated manually by air legs and single hole blasting. With 22 mm glynite or similar explosive in the contour holes, the charge per meter bore hole is approximately 0.4 kg, for which a damaged zone of 0.8 m is formed, see Figure 6. Although loose rock is removed during the careful preparation of the test site, it is highly probable that the damaged zone is in the order of 0.5 m here.

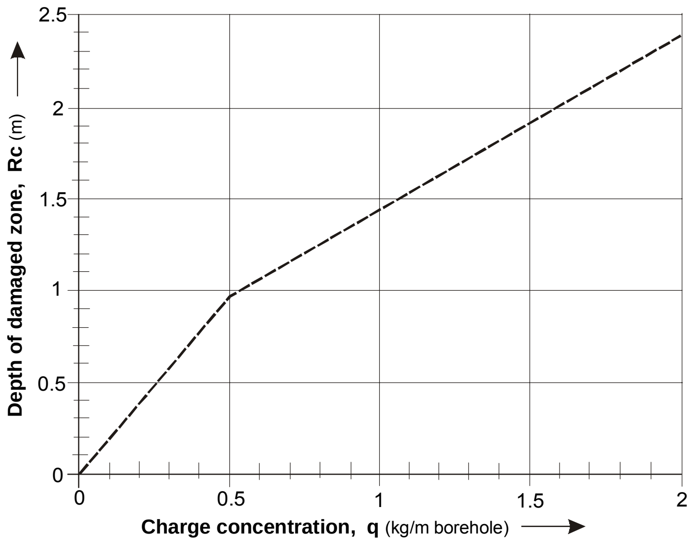


Figure 6: Chart to find the blast damage zone from charge concentration (from Ouchterlony and Olsson, 2000). For test adits where 22 mm glynite or similar explosive has been used, q = 0.4 kg/m and hence the damage zone Rc = 0.8 m

The damage from blasting is significant, particularly near the surface of the adit and it therefore strongly influences the Plate Loading Test (PLT). This is a main reason why the deformation modulus determined on basis of surface displacements by PLT generally gives much lower values than displacement measurements in drill holes in the Plate Jacking Test (PJT), as has been pointed out by Sharma et al (1989) and several other authors, see Table 2. Therefore the PJT based on borehole displacement measurements is better suited for in situ deformation measurements.

Also in the PJT measurement the damage from blasting influences on the nearest anchors of the extensometer, which are 0.2 to 0.3 m from the loading surface.

Ideally, the nearest anchor should be 0.5 to 0.8 m beyond the loading plate to avoid inaccurate measurements; the longest distance in the invert where generally larger charges are used. The stress level here according to Figure 3 is some 80 to 50% of the loading stress.

5.3 Comparison between conditions in a bored and a blasted test adit

For evaluating the effect from the blasting, two parallel tests were performed in an adit at the Lakhwar hydroelectric project in India as described by Singh and Rajvansi (1996). The rock mass comprises jointed dolerite and hornblende rhyolite. The following works were done:

1. One test site was excavated by *boring* a slot around the tunnel surface to avoid blast damage. A 6 m long section of the 2.1 × 2.5 m the adit was excavated by 76 mm core drilling forming a slot around

the tunnel surface, see Figure 7. Then the central rock material was removed by very careful blasting and chiselling. By this, the rock masses at the test location were undamaged. After site preparation, the PJT tests were performed.

2. The other test site was excavated by *drill and blast.* It was located 2.5 m further into the adit where the 2.1 × 2.5 m adit was enlarged by careful blasting to 4 × 4 m size.

Both measurements were performed upwards and downwards. The results are shown in Table 1.

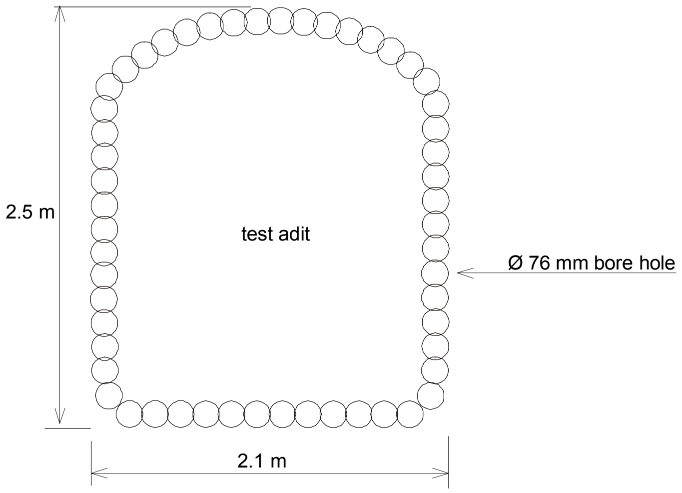


Figure 7: The procedure for excavating the bored test adit at Lakhwar dam project

Table 1: PJT deformation modulus (Em) at 6 MPa stress level in the bored test site of the adit (chainage 50) and in the blasted test site 2.5 m from the bored.

TEST SITE	VERTICAL HOLE UP FROM ROOF			VERTICAL HOLE DOWN FROM INVERT		
	Test section in upward hole	Em (GPa)	Ratio Em bore / Em blast	Test section in downward hole	Em (GPa)	Ratio Em bore / Em blast
in bored adit	0.13 to 3.68 m	49.2	5.4	0.29 to 4.72 m	23.9	9.6
in blasted adit	0.72 to 4.22 m	9.1		0.41 to 3.90 m	2.5	

As shown, there is a great difference between the deformation modulus between the two sites. As the rock mass conditions are similar in the two test locations (Singh and Rajvansi, 1996), it is probable that most of this difference can be explained by effects from blasting disturbance.

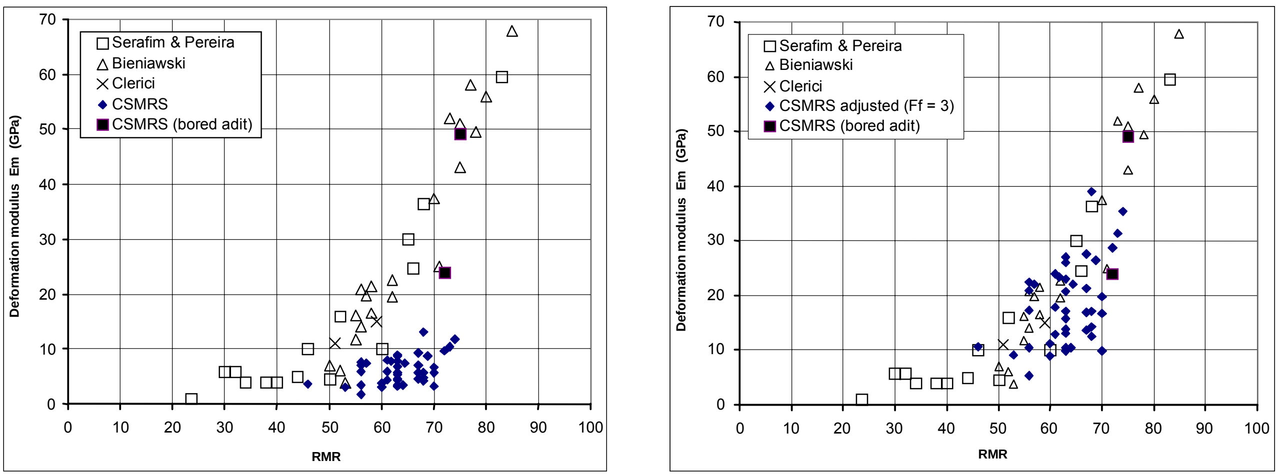


Figure 8: Left: CSMRS deformation measurements compared to those of Bieniawski, of Serafim and Pereira and of Clerici. The two measurements conducted in the "bored" adit are within those of Bieniawski etc. Right: The CSMRS measurements have been adjusted for blast damage with Ff = 3

From Figure 8 and Table 1 it is clear that, except for measurements in the bored adit, the deformation tests performed by CSMRS are lower than the real, probably because of the blasting damage in the adit. To compensate for this, a blast damage or field factor Ff = 3 has been applied to adjust the CSMRS results. The value of Ff has been estimated from the results of the CSMRS bored adit, and from comparison with the results of Bieniawski, Clerici, and Serafim and Pereira as seen in the right diagram of Figure 8.

6. COMPARISON OF THE DEFORMATION MEASUREMENTS RESULTS

6.1 General

The CSMRS has performed in situ deformation tests with the Goodman Jack and Plate Jacking during the last two decades at most of the important river valley projects in India, Nepal, and Bhutan. The procedures and suggested method of the International Society of Rock Mechanics (ISRM, 1979) have been closely followed for all the tests.

The experience is that the different procedures used for in situ measurements provide values that often differ from one another by as much as 100%. This is inevitable, not least due to the fact that the rock mass structure differs from one test to another. As the modulus is notably sensitive to the presence of joints, the rock mass conditions at each test site should be carefully described as part of the test procedure. By comparing the variations in rock mass quality some of the difference in test results may be explained.

From the test results of CSMRS it has been possible to compare and correlate the in situ measurement, as is shown in Sections 6.2 and 6.3.

6.2 Correlations between the various types of in situ measurements

As earlier pointed out by several researchers (Bieniawski, 1979; Heuze and Amadei, 1985), the value obtained by the various in situ deformation tests will not give the same deformation modulus. Based on CSMRS experience this may partly be explained by:

A. *Plate jacking test (PJT) with bore hole extensometer measurement*: Here, the deformations are measured inside the drill hole from the damaged zone towards the undisturbed rock masses.
B. *Plate loading test (PLT) with surface measurement*: The lower deformation modulus measured at the rock surface in these tests can be explained by the fact that these measurements are made in the damaged zone from blasting.
C. *Goodman jack test (GJT) performed inside the drill hole*: Also the Goodman jack tests have been found to give lower values of the moduli because, in hard rock, the loading platens deform. The main reason is that the displacement devices record the increase in bore hole diameter plus deformation of the loading plates.

From the measurements performed by CSMRS the ratio between these types of deformation measurements are given in Table 2, where also some experience published by other authors is shown.

Table 2: Ratio between plate jacking test (PJT) and other types of field deformation measurements, compiled from Singh et al. (1994), Sharma et al. (1989), Bieniawski (1989), Heuze and Amadei (1985) Goodman et al. (1968), CSMRS (1999), Singh and Dhawan (1999)

Ratio	Measurements in the following hydropower projects:				Experience by		Suggested ratio (Rp) between in situ measurements
	Lakhwar	Jamrani		Tala	Bieniawski	CSMRS	
		based on Goodman's constant	based on Heuze and Amadei's constant				
PJT /PLT	1.9			4.0		2 - 3	2.5
PJT /FJT	1.75					2 - 3	2.5
PJT /GJT	2.05	2.6	2.3	2.4	approx. 2	2 - 3	2.5

PLT = plate loading test; GJT = Goodman jack test; FJT = flat jack test

Bieniawski (1979) has stated that the flat jack test is the least reliable due to difficulties with the interpretation of the results as well as the small volume of rock tested near to the rock surface. Benson et al.

(1970) suggested that the modulus values must be obtained from PJT measurements. This is also the experience of CSMRS. They are less sensitive to variations in pressure distribution than displacements directly under the loaded area. The measurements of deformation in bore holes at various depths provide a check against any gross errors (blunders) of the measurements. They also allow a better assessment of the properties at depth as the displacements outside the loaded area are influenced to a much greater extent by the behaviour of rock.

6.3 Comparison between field measurements and between indirect estimates

A. Jointed rock masses

The results from CSMRS deformation tests of jointed rocks have also been analysed to obtain information on the influence of the rock mass composition and quality on the estimated modulus values. This has been done by comparing measured modulus values with corresponding values in three classification systems. The CSMRS results from the Goodman jack (GJT) have been adjusted by a factor of Rp = 2.5 to be comparable with the plate jacking test (PJT) results (refer to Table 2). Due to the closing of cracks and joints during the test, only values from the highest test pressures applied have been used. For the same reason the first cycle has not been used in the measurement. All in situ measurements of CSMRS have then been adjusted for the blasting damage with a factor Ff = 3, according to section 5.3.

All CSMRS measurements have been performed in specially excavated test adits. The rocks in the 42 test sites at 8 hydropower projects in India, Nepal and Bhutan have been various gneisses, granite, mica schist, sandstone, mudstone, siltstone, and dolerite/hornblende rhyolite. The uniaxial compressive strength of intact rock varied from 30 to 230 MPa.

At each test site Q, RMR classification and RMi characterization values have been calculated from the site descriptions, drill core logs and laboratory tests. In some cases where the compressive strength values have not been available, they have been estimated from relevant tables in handbooks. These values and test results have been plotted in Figures 10 to 12.

From the site description given by Clerici (1993) it has been possible also to use his results. In addition, the test results of uniaxial compressive tests of a 1.0 m diameter rock mass cylinder from Stripa, Sweden (Thorpe et al., 1980) have been included. The existing equations and also the best trend from the plotted values are shown. Figure 9 applies linear scales, while for Figures 11 and 12 logarithmic scales have been chosen due to the structure of these systems.

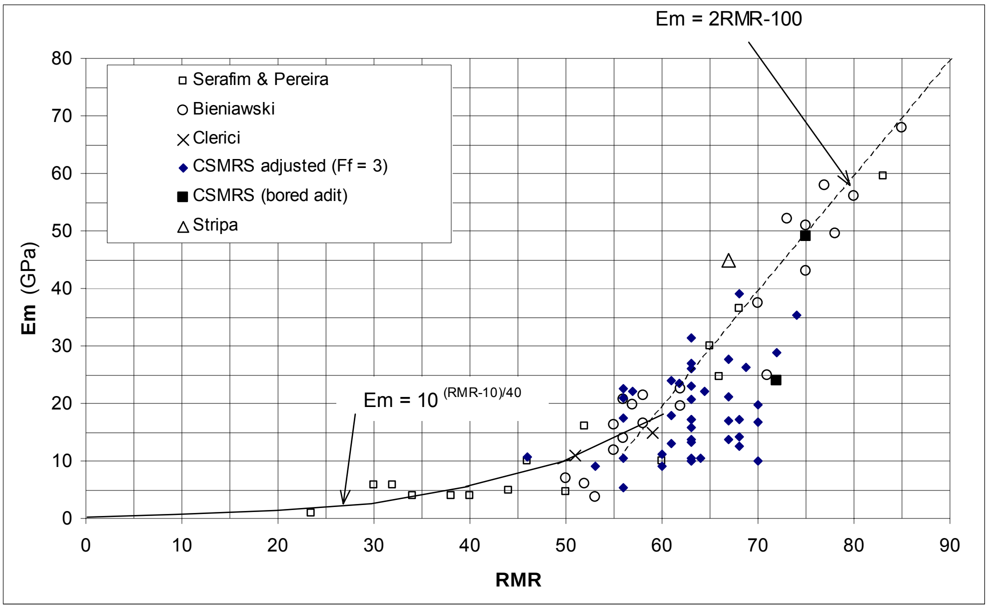


Figure 9: Connection between RMR and deformation modulus (Em). The CSMRS (blasted) results have been adjusted for blast damage with a factor Ff = 3

The Ff adjusted test values of CSMRS in Figure 9 fall within those of Bieniawski, Serafim and Pereira, and Clerici. The existing equations seem adequate, provided the equation of Bieniawski (Em = 2RMR-100) is applied for RMR > 55, and the equation of Serafim and Pereira (Em = $10^{(RMR-10/40)}$ for RMR < 60.

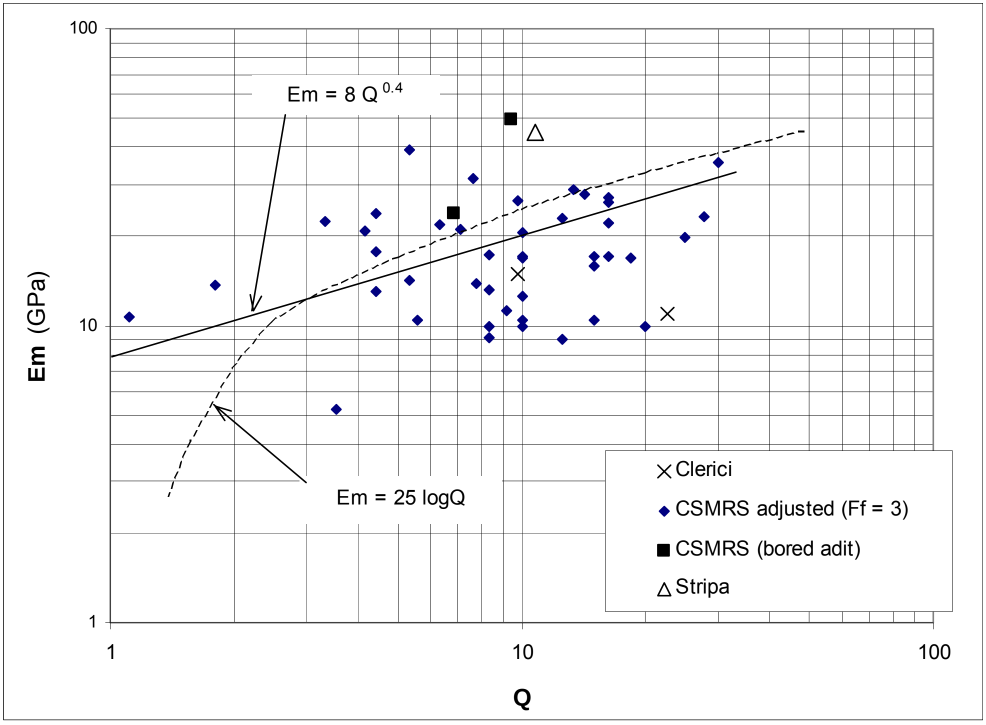


Figure 10: Connection between Q and deformation modulus (Em). The CSMRS (blasted) results have been adjusted with a field factor Ff = 3

In Figure 10 the existing equation Em = 25 log Q by Grimstad and Barton (1993) gives fair correspondence with the test result of CSMRS and poor fitting to the results of Clerici (1993) within an interval of 1 < Q < 30. The connection between Em and Q is not obvious. The best trend, which covers measurements within an interval of 1< Q < 30, is given by Em = 8 $Q^{0.4}$

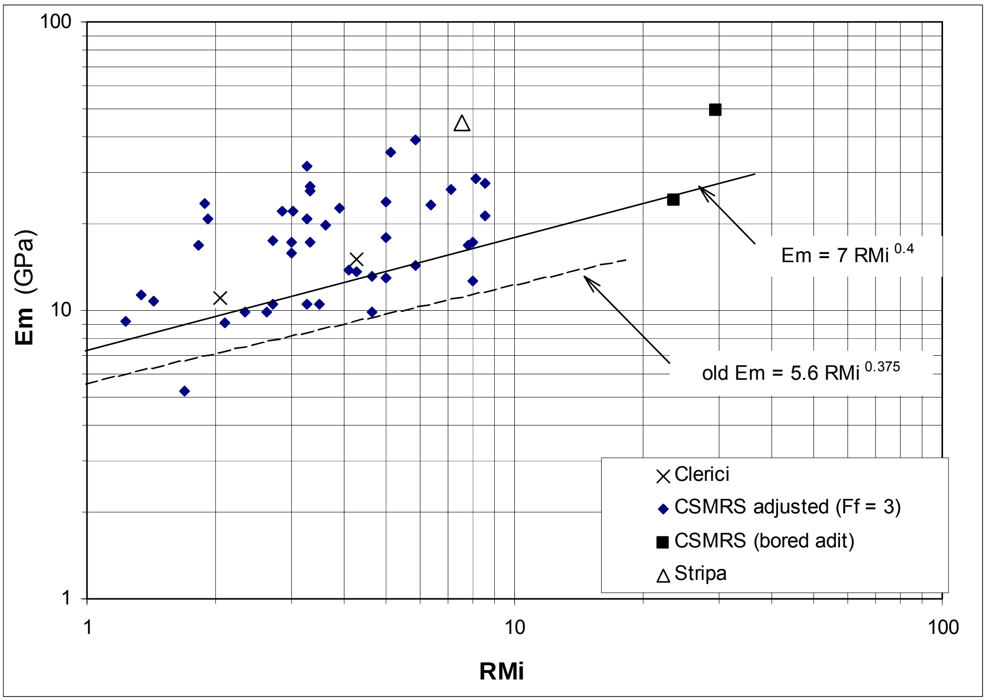


Figure 11: Connection between RMi and deformation modulus (Em). The CSMRS (blasted) results have been adjusted with a field factor Ff = 3.

As shown in Figure 11 the existing Em – RMi equation – which was based on theoretical evaluations from comparison with the RMR (Palmström, 1995) – gives too low values for Em. The expression Em = 7 RMi $^{0.4}$ gives a better correspondence within the range 1 < RMi < 30. It also fits well with the measurements of Clerici (1993).

B. Massive rock

Massive rock is a rock mass containing few or no joints. The scale effect for deformation in this type of rock mass is assumed similar to that of strength as indicated by Natau (1990). For uniaxial compressive strength between 50 mm test samples and rock blocks Barton (1990) suggested the following expression:

$$\sigma_{cm} = \sigma_c \times f_\sigma = \sigma_c\ (0.05/Db)^{0.2}$$

With $f_E \approx f_\sigma$ the deformation modulus for massive rock masses can be written as

$$Emr = E \times f_E \approx E\ (0.05/Db)^{0.2}$$

For massive rock the block diameter Db > approximately 2 m. Applying blocks within a range of Db = 2 - 4 m the scale effect $f_E = (0.05/Db)^{0.2} \approx 0.5$ giving $Emr \approx 0.5$ E.

As shown in Table A-1 in the Appendix, the modulus ratio MR = E/σ_c between the modulus of elasticity (E) and the uniaxial compressive strength (σ_c) for intact rock samples varies from 106 to 1600. For most rocks MR is between 250 and 500 with average MR = 400; i.e. E = 400 σ_c.

Based on Emr ≈ 0.5× E = 0.5 (400 σ_c) = 200 σ_c the values of RMR, Q, and RMi have been calculated for various massive rock masses composed of rocks with different uniaxial compressive strengths. The results in Table 3 are graphically presented in Figure 12, from which the following can bee seen:

– The values of Em in massive rock based on RMR give significantly higher values for all the rock strengths than Em calculated from laboratory test adjusted for scale effect.
– As the Q system does not apply input of the rock strength, the Em for massive rock mass based on Q has the same value for all rock strengths. For σ_c < approximately 150 MPa the value is considerably higher than the Em value estimated from laboratory tests adjusted for scale effect.
– The values of Em for massive rock mass found from the RMi equation are also higher than the values based on the laboratory results adjusted for scale effect, especially for weak rocks (σ_c < 10 MPa). The RMi estimates give, however, the best results of the three systems in massive rock.

Table 3: Various estimates of the deformation modulus in massive rock.

Uniaxial compressive strength	**σ_c =**	**4 MPa**	**20 MPa**	**60 MPa**	**200 MPa**	**Equation used**
Lab. test (50 mm sample	E =	1.6 GPa	8 GPa	32 GPa	80 GPa	E = 400 σ_c
Lab.test adjusted for scale effect	Emr =	0.8 GPa	4 GPa	16 GPa	40 GPa	Emr = f_E × 400 E = 200 σ_c
RMR *) (for massive rock)	RMR =	81	82	87	92	
	Em =	62 GPa	64 GPa	74 GPa	84 GPa	Em = 2 RMR - 100
	ratio Em / Emr =	78	16	6	2	
Q *) (for massive rock)	Q =	50	50	50	50	
	Em =	42 GPa	42 GPa	42 GPa	42 GPa	Em = 25 log Q
	new Em =	38 GPa	38 GPa	38 GPa	38 GPa	new Em = 8 Q $^{0.4}$
	ratio Em / Emr =	53	11	3.5	1	
RMi *) (for massive rock)	RMi =	2	10	30	100	
	old Em =	13 GPa	16 GPa	19 GPa	23 GPa	old Em = 5.6 RMi $^{0.375}$
	Em =	9 GPa	18 GPa	27 GPa	44 GPa	Em = 7 RMi $^{0.4}$
	ratio Em / Emr =	12	4	2	1.1	

*) Input values used: joint spacing = 3 m; RQD = 100; 2 joint sets, rough, tight and fresh joints, no water or stress influence

When the type of rock is known, Em in massive rock can be found from

$$Em = MR \times f_E \times \sigma_c$$

or, with $f_E \approx 0.5$ the deformation modulus in massive rock can be found from

$$Em = 0.5\ MR\ \sigma_c$$

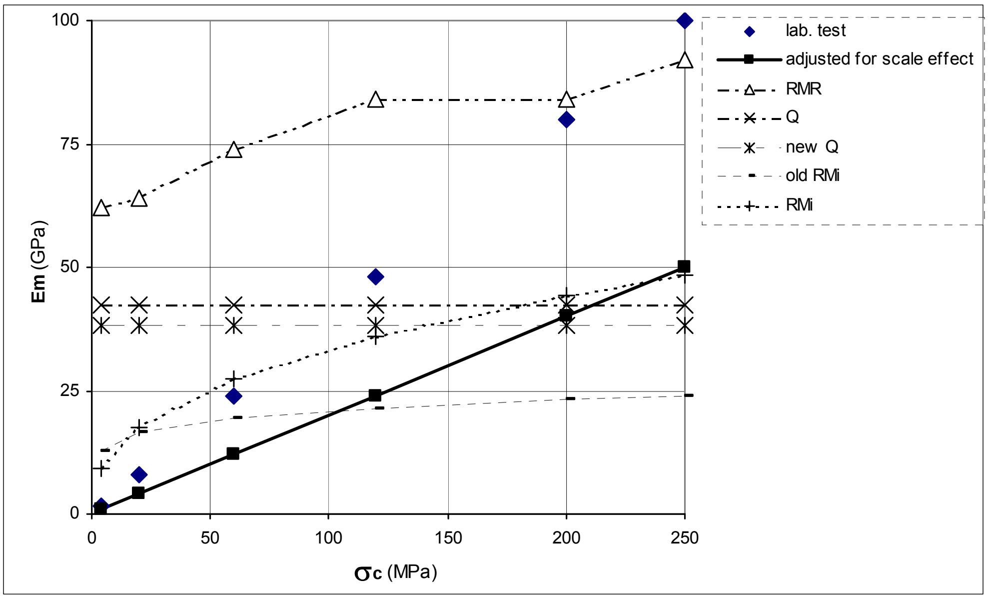


Figure 12: The deformation modulus (Em) in massive rock calculated from laboratory tests, RMR, Q, and RMi systems

7. DISCUSSION

The experience of CSMRS is that the rock mass modulus of deformation in a test drift often varies between the two walls and between roof and floor. Much of this comes from different rock mass qualities and different types of tests, as well as blast damage and how the preparation of the test site is performed. Some of the variation can also be explained from the difference in the modulus ratio MR = E/σ_c for the various types of rocks, as is shown in Table A-1 in the Appendix.

As mentioned earlier, the in situ measurements lack several sources of inaccuracy, such as site preparation and blasting damage at the test site, in addition to the measurement method and test procedure. Therefore, good site characterizations of the rock mass and use of an appropriate indirect method may in many cases give better results than expensive in situ measurements.

It is generally known that in situ tests of the deformation modulus of rock masses are subjected to measurement errors, both from equipment, test site preparation and blasting damage in the test adit. As Clerici (1993) states, accurate values can, therefore, seldom be found. But in order to arrive at the best possible results the persons involved in the tests must know the limits and problems involved in the tests.

As pointed out in this paper, the use of classification systems may be valuable tools in assessment of the deformation modulus, being aware of the limits these systems have. Clerici (1993) and Bieniawski (1989) recommend the use of more than one of these, so that the results obtained may be compared and their reliability checked. When estimating the deformation modulus using different classification systems, it is not recommended to use the correlation / transition equations between the systems, as mathematical equations tend to be rude and may give wrong values. Instead, the various parameters involved in the actual system should be given their relevant ratings and the classification value for each system calculated from these.

The deformation values used for *massive* rocks have been estimated. Field measurements are required to verify the assumptions made. The comparisons made indicate, however, that the deformation modulus calculated from classification systems seems valid only for the strongest rocks and that they give significantly higher values for weak rocks than the real.

It is a steadily increasing trend in the fields of engineering geology and rock mechanics to substitute geological reality by mathematical idealisations. This lack of interest in uncertainty and field observations easily leads to reducing the quality of the input parameters. It is, therefore of the utmost importance that experienced people with a background from practical rock construction are widely used in the collection of data to be used in the classification and characterization systems as well as in numerical models and mathematical analyses.

8. CONCLUSIONS

8.1 In situ measurements

From the many field tests of deformation performed by the CSMRS, the following conclusions can be made:

- The plate jacking test (PJT) gives the best in situ measurement results of the plate loading tests (PLT) and the Goodman jacking tests (GJT).
- The values measured by the Goodman jack test (GJT) and the plate loading test (PLT) are generally lower, in average they should be multiplied by a factor Rp = 2.5 to be compared with the PJT measurements.
- The excavation of test adits by blasting creates a zone of damaged rock around the adit, which is believed to cause reduced values of the deformation modulus measured in situ. A blast damage or field factor of Ff = 3 has been roughly estimated for the blasting influence. The measurements in PJT tests should not start closer than 0.8 m below the invert and 0.5 m from the roof to avoid the damaged zone.
- Due to the closing of cracks and joints in the damaged and distressed zone, the modulus value increases with the increase in applied pressure. Therefore, the values for the highest test pressures - 4 to 6 MPa, both for the plate jacking tests (PJT) and for the Goodman jack tests GJT) - should be used. For the same reason the deformations in the first cycle should not be included in the measurements.
- The effect of Poisson's ratio on modulus of deformation is almost insignificant within the interval of ν = 0.1 to 0.35.

8.2 Indirect estimates from classification or characterization systems

Table 4 gives a summary of the findings in Figures 9 to 12.

Table 4: Summing-up of indirect estimates of the deformation modulus of rock masses (Em).

System or method	**Em** (GPa) **in *moderately jointed* rock masses**		**Em** (GPa) **in *massive and slightly jointed* rock masses**
RMR	Em = 2 RMR – 100	for 55 < RMR < 90 (Bieniawski)	The RMR system should not be applied for massive rock masses
	$Em = 10^{(RMR - 10)/40}$	for 30 < RMR < 55 (Serafim and Pereira)	
Q	*Existing equation*	*Best trend*	The Q system should only be applied for very strong, massive rocks (σ_c > 150 MPa)
	Em = 25 logQ (for Q > 1)	$Em = 8\ Q^{0.4}$ (for 1 < Q < 30)	
RMi	$Em = 5.6\ RMi^{0.375}$ (for RMi > 0.1)	$Em = 7\ RMi^{0.4}$ (for 1 < RMi < 30)	$Em = 7\ RMi^{0.4}$ limited accuracy for σ_c < 100 MPa
Estimate from laboratory test adjusted for scale effect	Not applicable		$Em \approx 0.2\ \sigma_c$ or, when the rock type is known: $Em \approx 0.5\ \sigma_c \times MR/1000$ *)

*) MR = E /σ_c is the modulus ratio in Table A-1 in the Appendix. It varies with the type of rock.

In addition to the results in Table 4 the following conclusions can be drawn:

- Estimates based on the RMR and RMi systems show better deformation modulus values for *jointed* rock masses than the Q system. A reason may be that the Q system does not use input for the intact rock.
- The two existing equations for estimating Em by Bieniawski and by Serafim and Pereira in the RMR system seem applicable for *jointed rock* within their recommended range. However, the latter does not seem to cover values of RMR < approximately 30.
- New, adjusted equations for Em have been suggested both for the Q and the RMi systems.
- RMi gives better estimates of Em for *massive* rock than the Q and the RMR systems.
- For weak, massive rocks the deformation value should be estimated from laboratory test results adjusted for the scale effect.

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APPENDICES

A. Deformation modulus of intact rocks

Table A-1: Typical values of uniaxial compressive strength and elasticity modulus of some rocks with correlation between elasticity and strength. The rocks are grouped according to Goodman's intact rock classification (1989).

Average values from tests of rock samples		Tests of rocks world-wide				Scandinavian rocks tested at SINTEF			
		σ_c	E	E / σ_c	Number of tests	σ_c	E	E / σ_c	Number of tests
	ROCK	MPa	GPa			MPa	GPa		
Crystalline texture	Dolomite	86	38	443	8	110	49	443	2
	Limestone	107	47	441	81	74	71	961	25
	Marble	133	63	474	20	66	71	1074	4
	Greenschist	-	-	-	-	93	44	472	3
	Clay schist/-stone	68	38	563	2	40	21	537	6
	Micaschist	104	39	374	16	71	30	422	21
	Gneiss	130	53	406	27	130	50	385	107
	Micagneiss	-	-	-	-	89	29	330	5
	Granitic gneiss	-	-	-	-	89	29	330	5
	Granulite	90	41	451	4	-	-	-	-
	Amphibolite	212	101	474	7	107	70	660	16
	Greenstone	281	101	359	1	105	53	503	7
	Quartzite	209	58	276	28	172	56	328	7
	Anorthosite	228	90	395	2	157	86	545	2
	Diorite	173	64	368	6	130	52	403	6
	Gneissgranite	-	-	-	-	117	42	354	5
	Granite	154	48	313	71	169	42	250	20
	Granodiorite	160	51	319	2	171	20	118	2
	Gabbro	228	106	466	5	248	76	306	1
	Norite	229	82	356	8	-	-	-	-
	Olivinestone	-	-	-	-	87	113	1307	5
	Peridotite	197	55	280	1	109	164	1502	1
	Monzonite	110	28	256	8	106	61	580	4
	Andesite	152	31	206	6	-	-	-	-
	Basalt	145	50	347	25	207	82	395	3
	Diabase, dolerite	229	88	384	13	152	81	537	5
	Hyperite	-	-	-	0	245	108	441	2
Clastic texture	Graywacke	81	25	310	12	-	-	-	-
	Sandstone	109	28	257	95	147	28	189	5
	Siltstone	89	31	350	14	-	-	-	-
Very fine-grained rocks	Hornfels	111	74	668	3	-	-	-	-
	Claystone	5	2	301	2	-	-	-	-
	Phyllite	39	26	672	4	61	46	756	12
	Chalk	1	2	1606	2	-	-	-	-
	Marl, marlstone	17	2	133	9	-	-	-	-
	Mudstone	11	1	106	4	-	-	-	-
Organic rocks - coal		30	3	107	14	-	-	-	-
Average =		126	47	402		125	61	543	
Sum of tests =					500				281

B. Short description of three rock engineering systems

1. The RMR (Geomechanics) rock mass classification system

This engineering classification system, developed by Bieniawski in 1973, utilises the following six rock mass parameters:

1. Uniaxial compressive strength of intact rock material
2. Rock quality designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Groundwater conditions
6. Orientation of discontinuities

To apply the *RMR classification*, the ratings are assigned to the six parameters for each site. The typical, rather than the worst, conditions are used. Furthermore, it should be noted that the ratings, which are given for discontinuity spacings, apply to rock masses having three sets of discontinuities. Thus, when only two sets of discontinuities are present, a conservative assessment is obtained.

Bieniawski has modified the RMR ratings in 1974, 1976, 1979 and 1989.

2. The Q rock mass classification system

The *Q-system* for rock mass classification, developed at the Norwegian Geotechnical Institute (NGI) in 1974, originally included a little more than 200 tunnel case histories, mainly from Scandinavia (Barton et. al., 1974). In 1993 the system was updated to include more than 1000 cases (Grimstad and Barton, 1993). It is a quantitative classification system for estimates of tunnel support, based on a numerical assessment of the rock mass quality using the following six parameters:

- *Rock quality designation* (RQD)
- *Number of joint sets* (Jn)
- *Roughness* of the most unfavourable joint or discontinuity (Jr)
- Degree of *alteration* or filling along the weakest joint (Ja)
- *Water inflow* (Jw)
- Stress condition given as the *stress reduction factor* (SRF)*;* composed of
  - Loosening load in the case of shear zones and clay bearing rock,
  - Rock stress in competent rock, and
  - Squeezing and swelling loads in plastic, incompetent rock.

The six above parameters are grouped into three quotients to give the overall rock mass quality related to stability:

$$Q = RQD/Jn \times Jr/Ja \times Jw/SRF$$

3. The RMi rock mass characterization system

Earlier, the rock mass index (RMi) system has been presented by Palmström (1995, 1996, and 1997). In addition to its use in support estimates, the RMi can be used in several applications, such as characterisation of rock mass strength, calculation of the constants in the Hoek and Brown failure criterion for rock masses, and assessment of TBM penetration rate.

RMi is a volumetric parameter expressing the approximate uniaxial compressive strength of a rock mass. It is given as:

1. *For jointed rock*: $RMi = \sigma_c \times JP = \sigma_c \times 0.2\sqrt{jC} \times Vb^D$ $(D = 0.37\ jC^{-0.2})$
   Figure A-1 can be used to find the value of JP
2. For massive rock: $RMi = \sigma_c \times f_\sigma = \sigma_c \times (0.05/Db)^{0.2} \approx 0.5\sigma_c$
   This equation is used where the factor for scale effect $f_\sigma < JP$

The symbols in the expressions above are:

σ_c = The uniaxial compressive strength of intact rock, measured on 50 mm samples. Some average strength values are given in Table A-1.

Vb = The block volume, measured in m^3 ; Db = $\sqrt[3]{Vb}$ is the equivalent block diameter

f_σ = the massivity parameter, which is an adjustment for the scale effect of compressive strength in massive rock (where Db > approx. 1 m). For most types of ground f_σ varies between 0.3 and 0.6 with an average f_σ = 0.5, which for most practical purposes, can be used with sufficient accuracy.

JP = The jointing parameter, which incorporates the main joint features in the rock mass. Its value can be found from Figure A-1 or from JP = $0.2\sqrt{jC} \times Vb^D$

jC = The joint condition factor, which is a combined measure for the joint size (jL), joint roughness (jR), and joint alteration (jA), given as as jC = jL × jR/jA

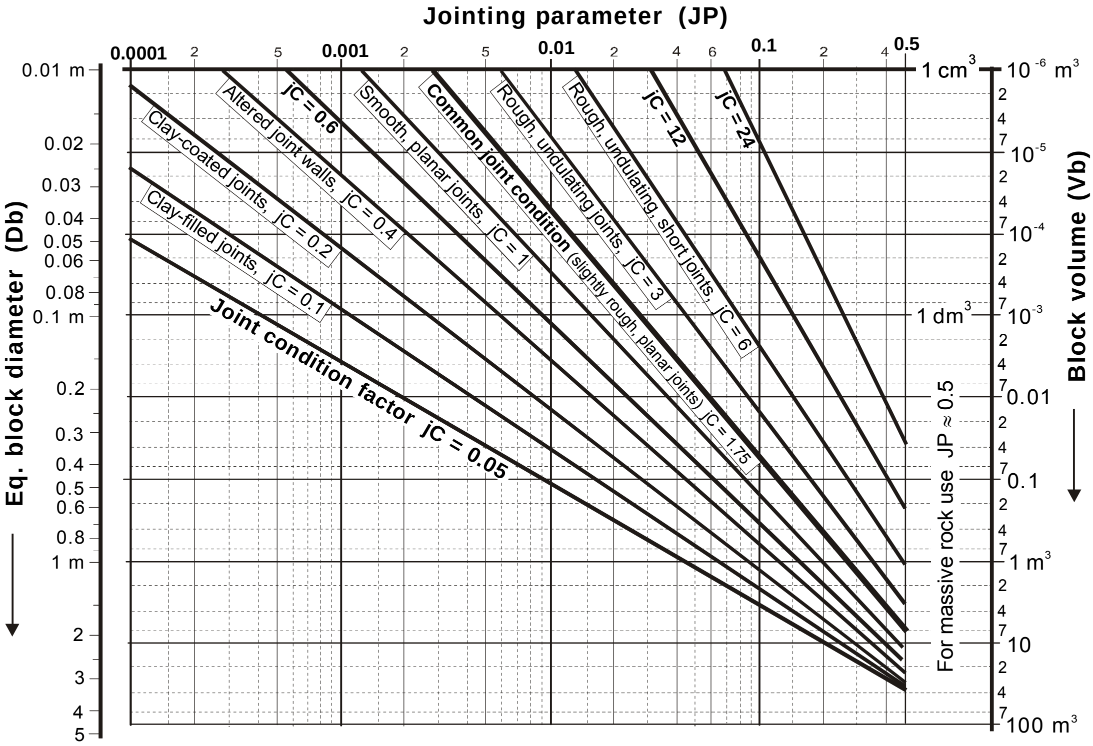

Figure A-1: Diagram to find the jointing parameter (JP) from block size and joint characteristics. Example: for Vb = 4 dm³ and jC = 3 the jointing parameter JP = 0.07 (the equivalent diameter Db = 0.16 m)

Applying the 3-D block volume as a main input to RMi leads to improved characterizing of a rock mass and consequently more accurate rock engineering calculations. By using computer spreadsheets, calculations can be made quickly as mutually dependent equations can be linked in the spreadsheet, see web page www.rockmass.net.