DMR (an adaptation of RMR), a new geomechanics classification for use in dams foundations.

DMR (adaptado del RMR), uma nova classificaçao geomecanica pra usar nas fondaçoes de barragens.

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ABSTRACT

Some topics, which difficult the effective use of RMR for dam foundations, are reviewed and a new geomechanics classification system, DMR (Dam Mass Rating), is proposed -as an adaptation of RMR- giving tentative guidelines for several practical aspects in dam engineering and for the appraisal of dam foundations in preliminary studies (overall stability against sliding, needed depth of excavation for the foundations, consolidation grouting treatment, possible consequences of excessive relationship -following Rocha concepts- between the deformation modulus of the dam and of the rock mass foundation) taking account of the effects of rock mass anisotropy and of the water saturation. The formulae for estimation of the rock mass deformation modulus E_m are also discussed. The DMR method can be useful when assessing the safety conditions of old dams, which are not well technically documented.

1 INTRODUCTION

A large dam is, almost always, a unique work, adapted to the morphology and the strength of the foundation, and also to the hydrological regime of the river. The dam, and the impounded water, interact with a great mass of terrain, very far away sometimes from the dam itself. The design and construction of a dam, are complex and casuistic, difficult to standardize. Nevertheless dams are classified and there is a “taxonomy” of the different dam classes. It is usual, when designing a dam, to refer it to precedents of similar dams in similar terrains. Needs of terrain strength and deformability quantification are quite different for each type of dam: arch, gravity (CVC , RCC or hardfill), CFRD, AFRD, rockfill, earthfill… As a rule of thumb concrete dams (and the face of CFRD/AFRD) require rock foundations whereas fill dams can be founded in soil.

Anyway it is widely accepted as good practice to fix the most important properties of a dam foundation referring them to some quality index (i.e. geotecnic zoning, seismic velocity of P-waves, weathering degree…). These properties are mainly permeability (frequently expressed in Lugeon units), shear strength of the foundation (in most cases cohesion and friction of the rock mass and/or of the governing joints), and terrain deformability. The dam must retain water, have enough safety against global sliding and adjust itself to the terrain deformations without too much cracking in service.

Therefore it is very convenient to arrange the quantitative data obtained from geologic-geotecnic field investigation attending to some previous idea about the importance of each one in face of dam design, construction and operation. And this is the concept which informs the geomechanics classifications. A very interesting precedent is the so called “Engevix preclassification” used in Itaipú to cope with the enormous amount of geotechnic data for the foundation of the long lateral wing dykes of the main dam. It was developed by Cruz (1976) and can be reviewed in Camargo et al (1978) and John (1978). Basically it is a rating system for the different properties of the rock mass attending to their effects in the dam foundation safety. Other interesting precedent is the Kikuchi (1979) classification, very well adapted to the geologically young volcanic terrains which prevail in Japan.

The RMR geomechanics classification was originally proposed by Bieniawski (1973) for use in tunnels, slopes and foundations. In fact the use of RMR has been very diverse: extremely frequent in underground works, very scarce in slopes and almost nil in dam foundations. There is only a seminal paper (by Bieniawski and Orr, 1976), no chapter on dams in the Bieniawski Jubilee Volume and very few application papers, except in only an important topic: estimation of the rock mass deformation modulus E_m. Several authors have referred to the use of RMR as a useful tool for the description of rock mass foundations (Di Salvo, 1982; Van Schalkwyk, 1982; Sánchez Sudon and Mañueco, 1991; Marcello et al, 1991; Hemmen, 2002; ,). Pircher (1982) said that *"the future seems to be in the development of quality index values e.g. RMR by Bieniawski"* and Serafim (1988) stated that *"appropriate rock mass classifications can … be used to obtain a good estimate of (shear strength and deformability) parameters"*, both in General Reports for Congresses on Large Dams.

2 DIFFICULTIES IN RMR USE FOR DAMS

Difficulties in RMR use for dam foundations derive from several points: consideration of the water pressure is very doubtful (the pore pressure ratio varies along the dam foundation, dams must operate with changing water levels…), there are no good rules for quantifying the adjusting factor for the joint orientation (which ideally should allow for the safety against total failure by horizontal shear, for local failure, for water leakage through the joints…), there are changes in properties of both the rock, the rock mass and the joints induced by watering changes (saturation ,desiccation, flow along the joints…).
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Guidelines were only offered for the general stability against horizontal sliding, which is important but it is not a very common problem (although there have been failures as in Malpasset). The dam engineer needs, when comparing possible dam sites, rapid appraisals of several other topics: general adequacy of site for each type of dam, depth of excavation of altered rock (if needed), required amount of foundation treatment (grouting). So, there cannot be only an adjusting factor and a sole guideline. Besides conditions will be different according the dam type.

An appraisal of the deformability of the rock mass is needed in order to calculate stresses, strains and deformations in dams. Hence, empirical correlations between geomechanics classifications and deformation modulus of the rock mass E_m have always been very popular. The first one of these correlations was proposed by Bieniawski (1978), and afterwards several authors have introduced modifications to improve it**.** Most of these correlations fail to consider two very important aspects: anisotropy of rock mass and water effect.

3 INFLUENCE OF WATER ON BASIC RMR

It is common to define a "basic" RMR_B independent of the work which is going to the built, as the addition of the five RMR parameters, without any adjusting factor. The fifth parameter, WR, is related to water, with a weight on RMR_B up to 15 points (15% of the maximal total).

The best method to determine the effect of water on this parameter is the use of the water pressure ratio $r_u = u/\sigma_v$ were u is the water pressure and σ_v the total vertical stress. The water rating WR can be approximated by the formula $\mathbf{WR = 10\log(1/r_u) - 1.5}$ **(valid for $0.02 < r_u < 0,7$) (1)**
to apply in RMR determination of the rating for the fifth parameter WR (see **Table 1**).

Table 1 Relationship between (WR) and r_u

WR	15	10	7	4	0
r_u (Bieniawski)	0	0-0.1	0.1-0.2	0.2-0.5	> 0.5
r_u (formula 1)	0-0.2	0.07	0.14	0.28	0.7

Around the dam $\mathbf{r_u}$ changes in every point depending on the valley geometry, the water level, and the efficiency of the grouting curtains (if exist). Therefore it would be necessary a three-dimensional flow model of the dam and the surrounding terrain to determine the $\mathbf{r_u}$ exact values. Anyway $\mathbf{r_u>0.4}$ for almost all the upstream points, so the WR parameter would get values of less than 2.5.

Furthermore the compressive strength of the rock will diminish heavily when saturated and its rating probably will halve. So a very crude way of taking account of the water effect on E_m would be to subtract around 15-20 points of the "dry" values of RMR. If we accept the Serafim-Pereira formula (see later) for determination of E_m from RMR $\mathbf{E_m\ (Gpa) = 10^{(RMR-10)/40}}$ the value of E_m (dry) would be approximately three times the value of E_m (saturated), for $10 < RMR < 70$. This result is not consistent with published data, which allow for a reduction on the order of 40%

for E_m when saturated. Therefore a rule of thumb could be to subtract 10 points of RMR (dry) to obtain E_n (water). It is interesting to note that this is congruent with prior versions of RMR (before the 1989 version which actually has become the "standard" one), and it is also the preferred method in Hoek's GSI index practice.

Anyway it seems that the water consideration is a serious handicap not only for the accurate determination of E_m by correlations with RMR, but also for the use itself of RMR in dams.

Hoek has advocated, speaking on the Hoek-Brown criteria, the use of a "dry RMR", obtained with the maximal rating of the water parameter, with simultaneous introduction of real pore pressures in the computations (see for instance the last version – "2002 edition"- in Hoek et al, 2002). No doubt that this is a sound procedure, when computing, and could be extended when dealing with geomechanics classification work in the upstream zone where the dam where pore pressures will be high.

We will define then a "basic dry RMR": **RMR_{BD}** as the addition of the first four parameters of RMR plus 15:
1) Compressive strength, tested in water conditions similar to the future ones, e.g. saturated when the rock is going to be saturated, and with the same ph of water.
2) RQD of the rock mass
3) Joint spacing of the significative governing joint(s).
4) Conditions of the significative governing joint(s).
5) Water rating WR, always 15 (as if dry)

4 STABILITY OF DAMS AGAINST SLIDING

Bieniawski and Orr (1976) proposed the following adjusting factors for the effect of the joints orientation in horizontal stability (**Table 2**) *"based on experience and on considerations of stress distributions in foundation rock masses as well as on an assumption that in a dam structure both the arch and the gravity effects are present"* (sic)

Table 2. Adjusting factor for gravity dams stability after joints orientation (Bieniawski &Orr, 1976)

TYPE OF DAM GRAVITY	VF	F	Fa	U	VU
	Very favorable	Favorable	Fair	Unfavora ble	Very unfavorable
DIP (º)	0-10	30-60	10-30 DS	10-30 US	60-90
RATING	0	-2	-7	-15	-25

DS dip downstream/**US** dip upstream

Snell and Knigth (1991) approached systematically the problem of the dam stability taking account of all the forces and stresses acting on the dams. Based in their study and others it appears that a different set of adjusting factors must be applied. **Table 3** shows these new tentative adjusting factors according the main discontinuities orientation. The numerical rating values proposed originally by Bieniawski have been retained.

Table 3. Adjusting factors for the dam stability R_{STA}, according joints orientation (Romana, 2003a)

TYPE OF DAM	VF	F	FA	U	VU
	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
FILL	Others	10-30 DS	0-10 A	-	-
GRAVITY	10-60 DS	30-60US 60-90 A	10-30 US	0-10 A	-
ARCH	30-60 DS	10-30 DS	30-60US 60-90 A	10-30 US	0-10 A
R_{STA}	0	-2	-7	-15	-25

DS dip downstream/**US** dip upstream/**A** any dip Gravity dams include CVC (Conventional vibrated concrete) and RCC (Roller compacted concrete), and hardfill concrete dams

When the dip direction of the significative joint is not almost parallel to the downstream-upstream axis of the dam the danger of sliding diminishes due to the geometrical difficulties to slide. It is possible to take account of this effect multiplying the rating of the adjusting factor for dam stability R_{STA}, by a geometric correction factor **CF**

$\mathbf{CF=(1-\sin|\alpha_d - \alpha_j|)^2}$ where α_d is the direction upstream-downstream of the dam axis and α_j is the dip direction of the significative governing joint. The value of $\mathbf{DMR_{STA}}$ (related to the dam stability against sliding) is:

$$\mathbf{DMR_{STA} = RMR_{BD} + CF \times R_{STA}} \qquad \mathbf{(2)}$$

where $\mathbf{RMR_{BD}}$ ("basic dry RMR") is the addition of the RMR first four parameters plus a water rating of 15 and $\mathbf{R_{STA}}$ is the adjusting factor for dam stability (**Table 3**).

Actually there are no data allowing to establish a correlation between the value of DMR_{STA} and the degree of safety of the dam against sliding. As a rule of thumb we can suggest:

DMR_{STA} > 60	**No primary concern**
60 > DMR_{STA}>30	**Concern**
30 > DMR_{STA}	**Serious concern**

These cannot be taken at all as numerical statements, but only as danger signals for the designer. Dam stability must always and in any case be checked by the designer taking account of the probable distribution of water pore pressure across the dam foundation and of the mobilized shear strength of the significative joints. About this question the Spanish National Committee on Large Dams states that *"The study of the dam safety against sliding requires a knowledge of the strength of the rock mass. The simple correlations between geomechanics classifications and rock mass strength are not well established for dam foundations"* (SCOLD, 1999).

5 GUIDELINES FOR EXCAVATION AND CONSOLIDATION GROUTING OF DAM FOUNDATIONS

The most usual requirement for the quality of the rock foundation for a concrete dam was something as *"good quality, sound rock, fresh, not weathered"*. Sharma (1998) is more specific demanding that *"the entire (foundation) area should be stripped to firm rock capable of withstanding the loads. Any layer of weak or soft material has to be excavated and replaced with concrete"*. He prescribes dental concrete treatment filling with concrete any open (or filled with soft fill) joint according USBR rules.

In most cases the foundation is excavated until class II rock in the central part of the valley (where the dam is higher) and until class II-III rock in the abutments. Spillways are founded, if possible, in class I rock. The most frequent parameter to be quantified in order to check the rock quality was the celerity of P-waves, measured by geophysical methods. Actually most rock masses are investigated and described with the ISRM suggested methods, which allow for a quick classification work.

It is desirable to gather data on the RMR value of dam foundations and this author is working in that. Actually some simple guidelines can be tentatively proposed (**Table 4**) for the depth of foundation excavation and for the required consolidation grouting of some few meters deep below the surface of foundation excavation.

Table 4 Tentative guidelines for dam foundation excavation and consolidation grouting (Romana, 2003a)

TIPE OF DAM	EXCAVATE TO RMR_{BD} (+)	CONSOLIDATION GROUTING ACCORDING RMR_{BD}		
		Systematic	Spot	None
EARTH	-	-	?	-
ROCKFILL	>20 (> 30)	20-30	30-50	>50
GRAVITY	>40 (> 60)	40-50	50-60	>60
ARCH	>50 (> 70)	50-60	60-70	>70

(+) minimum (desirable)

-gravity dams include CVC, RCC and hardfill concrete

-rockfill dams included are the ones sensible to settlement (with concrete –CFRD- or asphaltic –AFRD-face upstream)

6 CORRELATION BETWEEN E_M AND RMR

6.1 General

E_m modulus can have very different values depending on the direction of the principal stress. In stratified rock mass, and/or with a governing joint orientation, the equivalent elastic modulus of deformation is the weighted arithmetic mean of the deformation modulus of the strata (when the stress is parallel to them), and the weighted harmonic mean (when the stress is perpendicular to them).

Therefore the perpendicular deformation modulus is always the minimal one and the parallel deformation modulus is always the maximal one. The difference between both of them gets wider as anisotropy of the rock mass increases. Barton (1983) proposed the following formulae:

$$\mathbf{E_{min} = 0.4\ E_{mean}\ ,\quad E_{max} = 1.6\ E_{mean}}$$

and therefore $\mathbf{(E_{max} - E_{min}) / E_{mean} = 1.2.}$

This implies a relationship of 4 between maximal and minimal values of deformation modulus, which, according to Barton, is confirmed by other data published by Rocha (1964) and Bieniawski (1978), and is probably adequate for rock masses very anisotropic and/or very bedded. In homogeneous rock masses, however, the relationship between maximal and minimal values of the deformation modulus is smaller. Many authors have published data gathered from in situ tests (**Table 5**). In all the cases the RMR value to be applied is the basic one without the adjusting factor for the joints orientation proposed by Bieniawski for foundations. However, the variation of the modulus value according to direction of the principal stress suggests that it would be useful to apply some adjusting factor.

Table 5. Some data from in situ tests on relation E_{max} / E_{min}

SITE	ROCKMASS	E_{max}/E_{min}	REFERENCE
Colbun plant	Andesite	1,4	Van Sint (1993)
Ridracoli dam	Marl	1,3	Oberti et al (1986)
	Sandstone	1,4	
Tamzaourt dam	Sandstone	1,3	Jaoui et al (1982)
	Siltstone	1,9	

Water also reduces both the strength and the equivalent deformation modulus when the rock mass is saturated, a very important effect in dam foundations. The first papers neglected this water effect. The introduction of GSI (by Hoek) included a very simple way to consider the water effect: to evaluate RMR as if the rock mass were fully dry (with a value for the water parameter WR of 15) and to introduce water pore pressure in computations (see Bieniawski 2000). But that implies that neither the strength, nor the deformation modulus of the rock mass changes, and this hypothesis doesn't take account of the reduction of the strength and the deformability of the rock mass when saturated.

Figure 1 (Pells, 1993) shows a Deere-Miller diagram containing data from compression tests (failure strength and deformation modulus at 50% of failure strength) in dry and saturated Hawkesbury sandstone. Saturation implies a reduction almost proportional in both parameters, but the relationship between them would remain approximately constant.

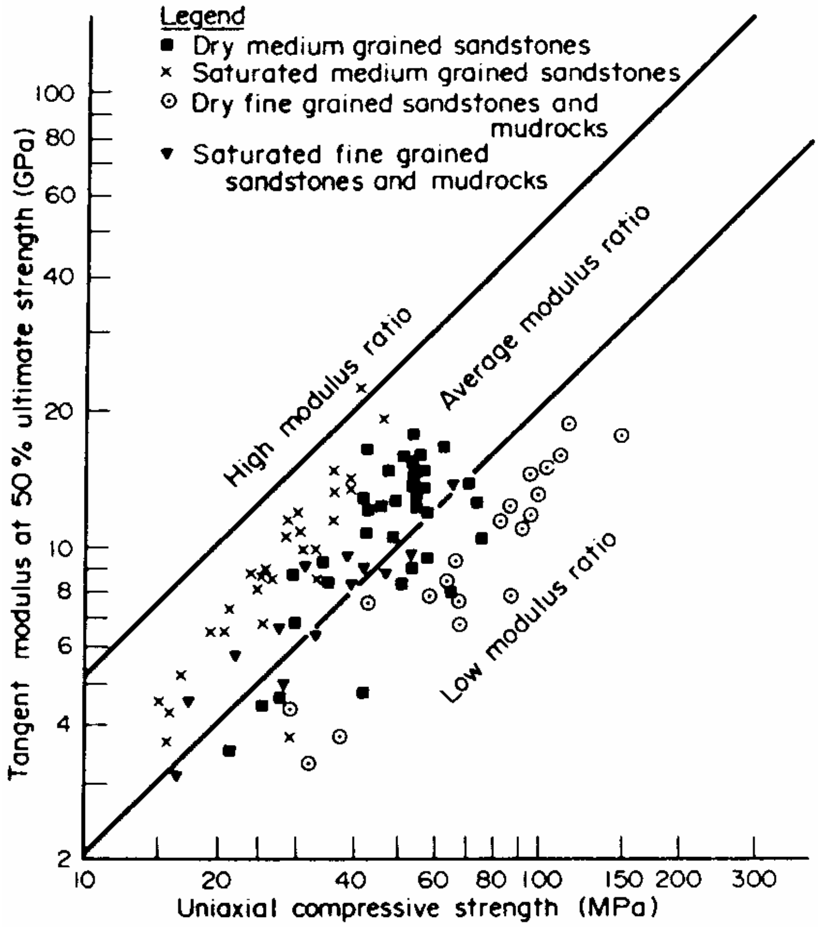


Figure 1. Strength and modulus data on Hawkesbury Sandstone (Pells, 1993)

6.2 Proposed correlations between E_M and RMR

Bieniawski (1978) proposed the correlation

$$\mathbf{E_m\ (GPa) = 2\ RMR - 100}$$

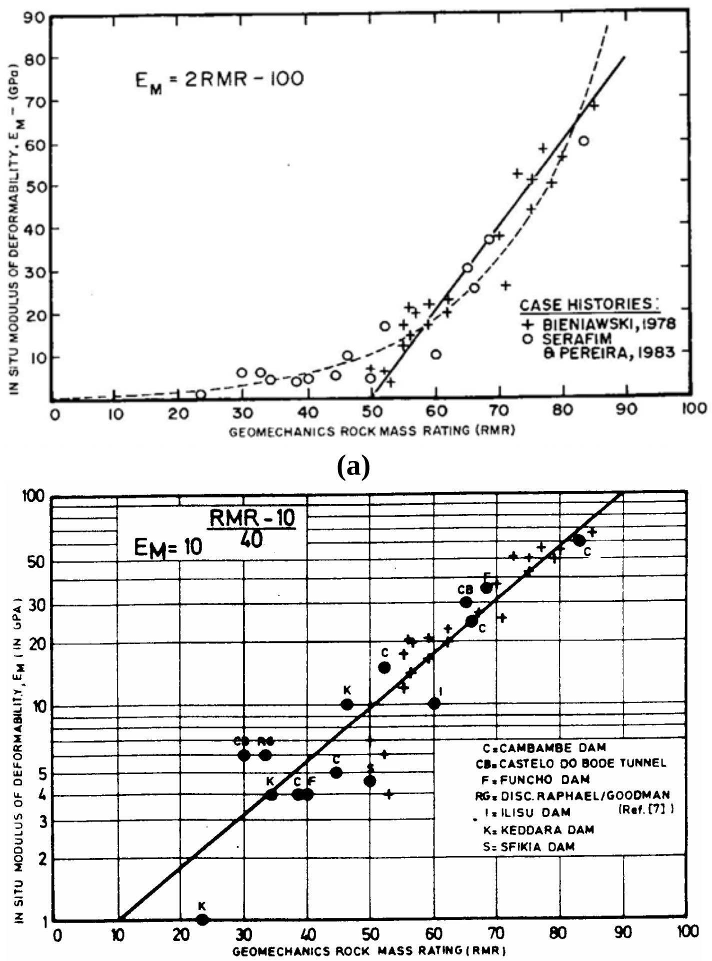
and it can be seen (**fig. 2a**) that this formula may be adequate for **RMR > 65**, has a wide scattering for **55 < RMR < 65**, (a very common interval in practice) and cannot be used for **RMR < 55**. Bieniawski recommends *"good engineering judgment"* when using his formula.

Bieniawski correlation has been modified by Serafim & Pereira (1983) for all values of RMR

$$\mathbf{E_m\ (GPa) = 10^{(RMR-10)/40}} \qquad \text{(fig. 2a,2b)}$$

a formula that has been widely accepted. This formula works better for **RMR > 34**. There were only three cases with lower RMR (22, 30 and 33) in their data bank, and the correlation was worse in all three of them.. **Figure 2a** shows Bieniawski and Serafim & Pereira correlations. Both are practically equivalent for **RMR > 65**, and both show a poor accuracy for smaller values of RMR.

In practice most of engineers follow procedures similar to the guidelines of USA Federal Energy Regulatory Commission (1999): *"**for RMR > 58** use Bieniawski formula; for **RMR < 58** use Serafim-Pereira one"*. The RMR=58 value appears to have been selected because it is the abscissa of the intersection between both curves.



(a)

(b)

Figure 2. Correlations between the *in-situ* modulus of deformation and RMR according to: (a) Bieniawski (1978) and (b) Serafim & Pereira (1983)

Hoek and Brown (1997) proposed a modification of Serafim & Pereira formula in order to take account of the influence of rock compressive strength σ_c (MPa):

$$\mathbf{E_m\,(Gpa) = (\sigma_c / 100)^{1/2} \times 10^{(GSI-10)/40}}$$

This formula is proposed for all values of RMR. The Serafim & Pereira original value would correspond to rocks of 100 MPa compressive strength, which is not usual in the rock masses studied by Serafim & Pereira, with RMR < 50. This modification by Hoek & Brown could be useful for high values of RMR and allows for the water effect testing dry and saturated probes. Hoek notes that this *"equation appears to work reasonably well in those cases where it has been applied. However, as more field evidence is gathered, it could be necessary to modify this relationship"*. In fact there is no data supporting the use of this modification for rock masses of medium or low hardness.

In a paper for the 2002 NARMS, Hoek, Carranza-Torres and Corkum have introduced an additional *"disturbance factor"* **D** $\mathbf{E_m(GPa)=(1-D/2)x(\sigma_c/ 100)^{½}x10^{(GSI-10)/40}}$

The new factor **D** *"allows for the effects of blast damage and stress relaxation"*, and can be estimated according guidelines given for tunnels, slopes, and pit-quarries, but not for dams. As excavations for dams foundations are, as a rule, very careful, D should be very low, but it cannot be 0 because of the rock mass decompression when excavating the dam foundation. Tentative guidelines (Romana 2003a) are as follows:

-Good rock mass, normal blasting	**D= 0,4**
-Any rock mass, controlled blasting	**D= 0,2**
-Poor rock mass, mechanical excavation	**D= 0,2**

6.3 Recommended formulae

With the published data there is only actual basis for the following recommendations:

-RMR_{BD} > 60 **Use Bieniawski formula**
-60 > RMR_{BD} > 35 **Use Serafim & Pereira original formula.**
-35> RMR_{BD} **No formula is sure. Use as a guess Serafim & Pereira one**

There are no data supporting normalized Hoek variations of Serafim & Pereira formula. The results obtained when applied to good-medium quality rock masses (e.g. RMR class II-III) with medium hard rocks (e.g. $_{c}$ =50 MPa) are not consistent with the published data on E_m.

6.4 Effect of anisotropy on E_m

E_m depends on the anisotropy of the rock mass and the maximum stress direction. It is possible to take account of this effect with the following rules of thumb:

-In very anisotropic rock masses E_{max}/E_{min} = 4.
You can add 8 to RMR_B to obtain E_{max} and subtract 16 to RMR_B to obtain E_{min}.
-In slightly anisotropic rock masses E_{max}/E_{min} is in the order of 1.4.
You can add 2 to RMR_B to obtain E_{max} and subtract 1.4 to obtain E_{min}.

Such values are only approximate but can be useful when making sensitivity analysis of dam – foundation stress fields to get something similar to extreme situations. The best solution would be to apply some correction factor depending on the orientation of the joint(s) which define the more competent "strata" in the rock mass

6.5 Effect of water on E_m

The effect of water on E_m has been already discussed. There are no published data and most of authors used the same value of E_m before and after impounding the dam. The most frequently used WR (water rating) were 7 and/or 10 reflecting a certain aim for compromise between WR = 15 (dry, r_u = 0) and WR = 0 (fully saturated, r_u > 0.5). The above proposed rule of thumb (to subtract 10 from RMR_B to get E_m saturated, when using Serafim & Pereira formula) is only a guess.

7 INFLUENCE OF THE FOUNDATION DEFORMABILITY ON THE DAM BEHAVIOR

7.1 General

There is a general agreement between dam engineers on the fact that two cases are dangerous for the normal behavior of a concrete dam: if E_m varies widely across dam foundation, or if E_c/E_m reaches certain values (E_c being the deformation modulus of concrete). Rocha (1964) established the most followed rule for arch dams (**Table 6**) in a paper which has become a "classic" reference for dam designers.

Table 6 Effect of E_c/E_m on arch dam behavior(Rocha, 1964)

E_c/E_m	Influence on dam	Problems
< 1	Negligible	None
1-4	Low importance	None
4-8	Important	Some
8-16	Very important	Serious
> 16	Special measures	Very dangerous

E_c/E_m < 4 allows for an easy behavior (and "*high cost tests in the foundation exploration could be dispensed with*" according Oliveira, 1990). The minimal sure (but with problems) value of E_m for an arch dam would be around 5GPa. The reported cases of arch dams founded in rock masses with E_m<5GPa show serious problems (cracking included) because of the low value of E_m.

Rocha et all (1974) presented data on the Alto Rabagao dam *"built in a very deformable foundation"*, with a modulus relationship $\mathbf{E_c/E_m}$ of 20 above elevation 830 (maximum water level 880). *"Tensile stresses (were displayed)…in some points near the foundation downstream in the left bank. (As) these tensile stresses (were) specially relevant…it was recommended …reinforcement parallel to the downstream face and …to the ground"*

Silveira et all (1991) in a paper with the title "Influence of foundation heterogeneity on safety of arch dams" which was presented at 17th ICOLD Congress in Vienna analyzed the stresses in several built arch dams with very different values of E_c/E_m both in the after-construction state and the predicable aged state after many years. Their conclusions were that *"this influence (of the heterogeneity of foundation deformability) on the behavior of arch dams for ordinary scenarios is (well) summarized (in Rocha table) and only for large heterogeneities this influence is remarkable for arch dams"*, *"the heterogeneity of foundation decreases the safety factor and thus the capability of the dam to resist to ageing. However….this reduction is only remarkable for larger heterogeneities"*

In later papers Rocha (1975, 1976) extended his work to gravity dams. $\mathbf{E_c/E_m < 8}$ would be safe and $\mathbf{E_c/E_m > 16}$ would get to moderate to big problems. The existence of joints in concrete dams helps to cope with relative deformability problems. This may be the main reason in the changes in the design of RCC concrete dams, from the first dams with almost no joints to the actual standards. Nevertheless RCC concrete gravity dams are less prone to problems than CVC concrete dams due to the lesser value of E_c.

7.2 Value of E_c in CVC and RCC concrete

Andriolo (1995) gave a detailed report comparing properties of CVC and RCC concrete. **Fig. 3** shows the data on E_c from 5 CVC dams and 13 RCC dams. The mean data gives relationships of:

$E_{RCC} = 0.40\ E_{CVC}$ (at 7 -28 days)

$E_{RCC} = 0.55\ E_{CVC}$ (at 90 days)

At 90 days E_{CVC} varies between 28 Gpa and 51 Gpa with a mean value of 39 Gpa, whereas E_{RCC} varies between 11 GPa and 32 GPa with a mean value of 22 GPa. There is a great variation, depending on the cementitious material content, but we can assume for **CVC** $\mathbf{E_{CVC} = 30/36}$ **GPa** , and for **RCC** $\mathbf{E_{RCC} = 20}$ **GPa** (less in many cases).

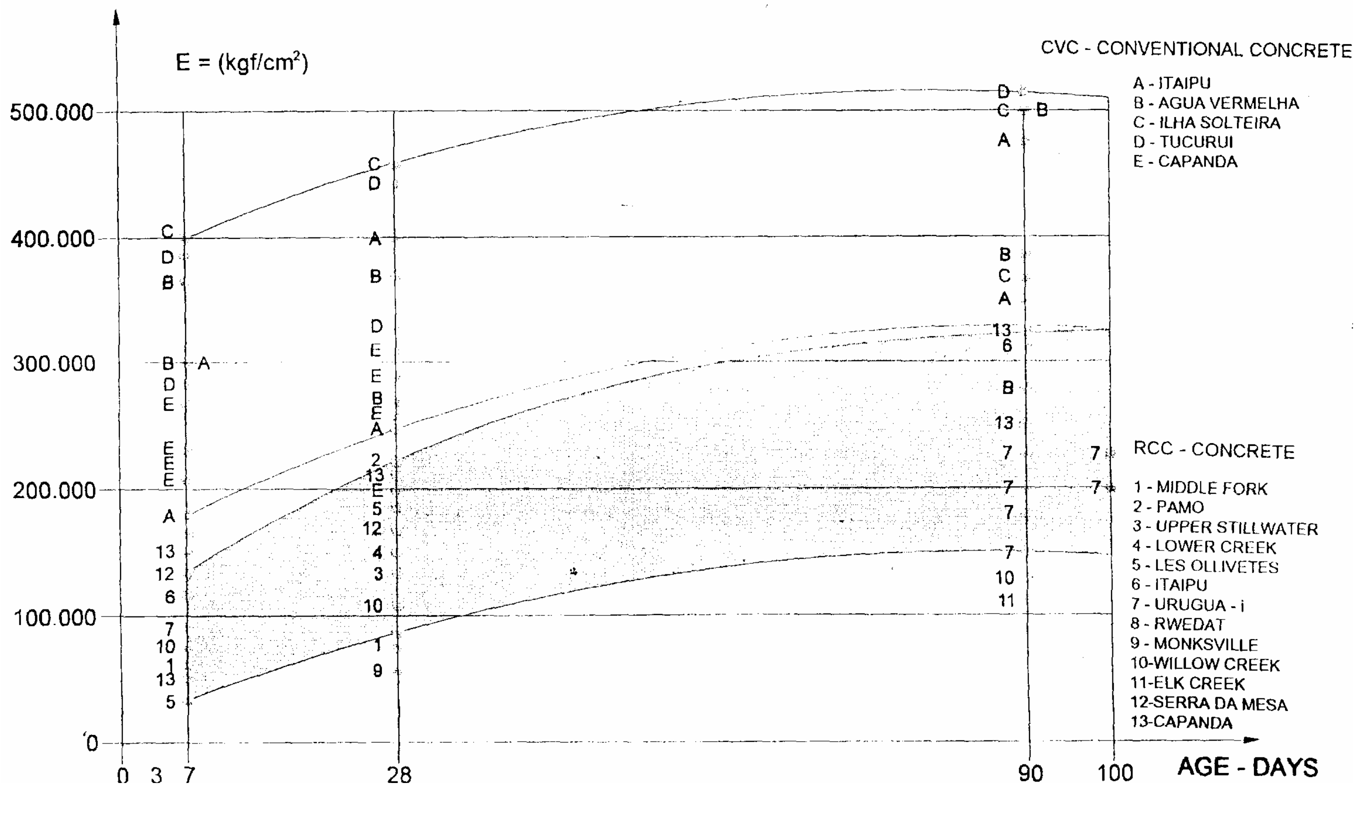


Fig. 3. Modulus of elasticity values from RCC and CVC concrete in dams (Andriolo,1995)

7.3 Value of E_{hard} in hardfill dams

There are only very few hardfill dams already built. However ICOLD, in his Bulletin on the State of Art of RCC dams (2000), proposes **E_{hard} = 10 GPa, (**or less), for a paste with compressive strength of 9 MPa, values which are congruent with the data from the Lower Monción Dam, built with hardfill and already in operation, (Capote et al, 2003), and will be used in DMR method until more data can be gathered.

The same E_c value can be adopted for the revision of old dams, built with poor concrete of low strength, and / or deteriorated by ageing.

7.4 Guidelines for DMR_{DEF}

Zeballos and Soriano (1993) have published the results of Zeballos Ph. D. thesis (Polytechnic University of Madrid), an extensive and intensive study on the effects of E_c/E_m value on gravity and arch dams **Table 7** (gathered using their data and others) shows the different ranges of RMR_{DEF} related to the different ranges of possible problems in the dam due to the differences of deformability between the dam and his foundation.

DMR_{DEF} (RMR related to deformability by the Serafim & Pereira formula)) depends on E_m (when the rock mass is saturated) and can be estimated with WR = 5 (a mean value which correspond to a nominal mean value of r_u = 0.25)

Table 7. Deformability problems in concrete dams according value of DMR_{DEF} (modified from Romana 2003)

DAM E_c (GPa)	HEIGHT (m)	Normal	Problems	Serious problems
Arch 36 GPa	< 100 100-150 150-200	>50 >65 >75	40-50 50-65 60-75	<40 <50 <60
Gravity CVC 30 GPa	< 50 50-100 100-150	>40 >50 >60	25-40 40-50 50-60	<25 <40 <50
Gravity RCC 20 GPa	< 50 50-100 >100	>35 >45 >55	20-35 35-45 45-55	<20 <35 <45
Hardfill 10 GPa	<50 50-100	>30 >40	15-30 30-40	<15 <30

8 USE OF DMR IN THE APPRAISAL OF SAFETY CONDITIONS IN OLD DAMS

At this moment a big number of old dams are in operation. Many were built before rock mechanics beginning (during the 1950-1960 decade) and were designed based only in the good engineering sense and experience of the designers (engineers and geologists). The existing information on the foundation conditions is very scarce and never is presented following the actual “good practice”, which has been developed in the last 30 years.

The analysis of the safety conditions of these old dams has become mandatory in all countries. The DMR geomechanics classification is a good method to gather orderly sound geomechanics information about the rock mass foundation and allows for a preliminary appraisal of the “weak points”. The author has used the DMR geomechanics classification, as a first tool in the safety analysis of four old (more than 50 years) gravity dams near Valencia (Balagueras, Regajo, Ulldecona and Valbona dams) with good results. All of them are in operation and show no visible signs of serious damage, but the concrete quality varies from good one (Regajo dam) to very poor (Balagueras dam)

9 FINAL REMARKS

This paper has been written in the first days of January 2004 and is based in two prior ones: Romana, 2003a (September), and Romana, 2003b (November). It presents the first results of a ongoing research. The author will be grateful to any contribution with data which confirm, or deny, these preliminary results mromana@stmr.com

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DEFINITIONS

WR	Rating of the fifth parameter (water) in RMR
RMR_B	Basic RMR, with no adjusting factor for joints orientation.
RMR_{BD}	Dry basic RMR, with no adjusting factor for joints orientation. WR = 15
R_{STA}	Adjusting factor for joint orientation (dip) related to dam stability (Table 3)
CF	Geometric correcting factor for relative orientation of joints and dam axis. $CF=(1-\sin\lvert\alpha_d-\alpha_j\rvert)^2$
α_d	Direction upstream-downstream of the dam axis
α_j	Dip direction of the significative governing joint for dam stability.
DMR_{STA}	DMR related to dam stability
	$DMR_{STA} = RMR_{BD} + CF \times R_{STA}$
E_m	Deformation modulus of rock mass.
E_{max}	Maximal deformation modulus of rock mass.
E_{min}	Minimal deformation modulus of rock mass.
E_c	Deformation modulus of concrete
E_{CVC}	Deformation modulus of conventional vibrated concrete
E_{RCC}	Deformation modulus of roller compacted concrete
E_{hard}	Deformation modulus of hardfill
DMR_{DEF}	RMR related to relative deformability, with WR = 5, and no adjusting factor for joints orientation.(Tab.7)