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Review on the Estimation of Static Deformability Modulus of Rocks and their adoptability in Different Rock Masses

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Abstract: The aim of this study is to review the different mechanisms employed in the estimation of static rock mass deformability modulus (E_m) in rock engineering applications and to investigate the adoptability of the identified mechanisms in different rock masses. The paper discusses different evaluation criteria through experimental, empirical and other means, with their merits and demerits, including influential factors. It is known that deformability modulus of intact rock depends on the imposed stress, strain rate and the confining stress on the rock sample as well as the rock texture and structure. The results generated for E_m by different in-situ tests are different and an appropriate in-situ test based on the rock mass conditions should be employed to obtain reasonable results. Empirical criteria are found to produce results of reasonable precision if appropriately adopted for specific rock mass conditions, while the back analysis method is widely adopted as an in-situ estimation measure for the design of rock-sockets and tunnel support. It has also been reported that substantial reduction in E_m occurs due to schistosity and larger test volumes, while it is sensitive to stress and discontinuity conditions. In this work, specific recommendations are made on the estimation of E_m for different types of rock masses based on the findings and reviews reported in the literature.

Keywords: Rock mass deformability modulus, Intact rock deformability modulus, In-situ testing, Empirical methods

1. Introduction

Reasonable estimation of the rock mass deformability modulus is vital in any engineering application in rock masses as it is the best representative parameter of the pre-failure mechanical behaviour of the intact rock material and of rock mass.

When considering the definitions on rock deformability, modulus of deformability represents both elastic and plastic deformation before reaching the peak strength level, while Young's modulus (E) represents only the deformability below the proportionality limit of intact rock [1]. Though there are considerably large number of criteria available in the determination of deformability modulus of both intact rock (E_i) and rock mass (E_m), the application of the parameters into real design practice had received insignificant attention, especially in foundation design. This is mainly due to the higher cost and time needed for the evaluation of this parameter, accompanied with the differences that have been identified between the actual deformation behaviour of rock mass and that predicted by the laboratory determined E_i (due to geological anisotropy) [2], as well as by the in-situ derived E_m [3] due to test method dependency. As a way out, designers have frequently sought empirical means in the estimation of E_m, which is rapid and inexpensive, though it is conservative.

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Hence in this study, an effort is made to investigate the applicability of existing empirical criteria in the estimation of static deformability modulus of rock masses (E_m), first with respect to the general rock engineering practice and secondly with respect to their adoptability in hard crystalline rock masses.

Following aspects are covered in this work:

- Investigation of different mechanisms used in the estimation of E_m, their merits, demerits and the related influential factors on the parameter.
- Review of the adoptability of the identified mechanisms and empirical criteria into design practices in different rock masses.
- Recommendation of appropriate measures to be considered in the estimation of E_m in different rock masses.

2. Estimation of Deformability Modulus of Rock Mass

Palmstrom and Sing [4] state that there are number of laboratory tests on rock specimens as well as in-situ methods available for the direct evaluation of E_m, while there are also a substantial number of indirectly evaluated empirical (correlation) and analytical (equivalent continuum approach) criteria.

2.1 In-Situ Methods

2.1.1 Direct In-Situ Testing

When considering the adoptability of direct in-situ evaluation methods in competent hard crystalline rocks, Goodman Jack Test (GJT) is preferred over Pressuremeter Test (PT) due to several reasons. One of the main seasons is the capacity restrictions in the latter, which can apply only around 30MPa. The second reason is the failure of PT membrane in the fractured rocks. However, the results obtained from GJT needs considerable post treatments to obtain the actual E_m from the calculated E_m [5]. Considering the applications in deeper and directly inaccessible test locations such as rock sockets, the preferred direct methods are Borehole/Goodman Jack Test (GJT) [6] and Pressuremeter/Dilatometer Test (PT) [7] as other methods do not facilitate the performance of the test at greater depths.

2.1.2 Indirect In-situ Testing

In addition, indirect geophysical methods such as Resonant Column Testing and Ultrasonic Pulse Testing on intact rock as well as Down-the-hole and Cross-hole sonic logging on rock mass are available to estimate E_i and E_m respectively, which are found to produce results which agree reasonably with the results obtained from direct in-situ tests [8].

2.2 Indirect Methods

2.2.1 Estimation of E_m using Young's Modulus (E_i) of Intact Rock

ASTM D7012-07 [9] specifies the laboratory estimation of intact rock Young's modulus (E_i), in combination with the unconfined compressive strength (σ_c) and the Poisson's ratio of the intact rock sample. With the stress-strain behaviour during compression, it is possible to obtain average, tangent and secant Young's moduli. The (E_i) so obtained is used to estimate (E_m) using empirical and analytical methods described undersections 2.2.2 and 2.2.3, respectively. In the absence of directly determined E_i, it is possible to estimate the same using the characteristic value of modulus ratio (MR, the ratio, E_i/σ_c, which is generally found to be in a specific range for a particular rock) and the σ_c [10].

2.2.2 Empirical Methods

There are a substantial number of empirical formulae proposed by different researchers to be used in the general rock engineering practice. Information obtained through a comprehensive literature review is reported in Table 1 and most of the listed relationships have been established through statistical treatments performed on databases containing in-situ test data from different rock engineering applications and rock lithologies. Careful observation of equations reveals that there are mainly two types of equations to estimate the rock mass deformability modulus. In the first category, E_m is directly related to one or more parameters, while in the second category, E_m is expressed in terms of E_i accompanied by other related parameters. Moreover, some authors have proposed relationships among different combinations of parameters to facilitate the application of the relationship based on the parameter availability. In addition to the above set of equations, a separate set of relationships has been proposed by different authors to estimate the Young's modulus (E_i) of intact rock as reviewed by Zhang [11]. These relationships are mainly related to the petrophysical properties such as porosity, density, hardness, water content, Schmidt hammer rebound number, and P and S wave velocities of different lithologies. Nevertheless, it is not practical to apply all the equations presented in Table 1 for all the rock engineering applications encountered due to the limitations in quality and adequacy of test data, necessity for further establishment, and the differences

Table 1 - Existing Empirical Relationships to Estimate the Rock Mass Modulus

Related Parameter/s	Empirical Correlation	Equation Number	Author/s	Development basis and remarks from author/s
Intact Parameters	***Relationships with Intact Young's modulus (E_i)***			
	$E_i = E_0\sigma_3^{\alpha}$;$\sigma_3$- lateral confining stress, α-factor very low for hard rocks, high for weaker rocks, E_0-Young's modulus determined by Unconfined compression, E_i –Intial tangential modulus to accommodate nonlinearity and stress-dependency	… (1)	Kulhawy [12]	Results of a series of investigations by different researchers for 87 different types of rocks, in which data available for one rock type tested at several orientations having a total of 115 modulus values. Correlations obtained with least-squares fit.
	$\frac{1}{E_m} = \frac{1}{E_i} + \frac{1}{JS*k_n}$; JS -joint spacing, k_n- joint normal stiffness	… (2)	Kulhawy [13]	Assumes that E_m depends on the deformation of the monolith rock and the deformation of discontinuities.
	$E_{mt@50\%} = E_{it@50\%}e^{(-1.15*10^{-2}*J_f)}$; $J_f = J_n/n*r$ J_n –No. of joints/m in the direction of the loading/ major principal stress, n -inclination parameter depending on the orientation of the joint, r – roughness/ the frictional coefficient on the joint or joint set of greatest potential for sliding	… (3)	Ramamurthy [14]	
	$E_m = 0.5E_i$	… (4)	Palmström & Singh [4]	For massive rocks with few or no joints. Considering the scale effects on E_m for massive rocks, relationship between σ_c of test samples and blocks.
	Relationships with intact unconfined compressive strength (σ_c)			
	$E_m = 215\sqrt{\sigma_c}$ (MPa)	… (5)	Rowe & Armitage [15]	Upon the back analysis of a large number of load test results on piles socketed into weak rock.
	$\log_{10} E_m/\sigma_c = 2.73 - 0.49\log_{10}\sigma_c/P_a$	… (6)	Prakoso [16]	
	$E_m = 0.2\sigma_c$ (GPa)	… (7)	Palmström & Singh [4]	For massive rocks with few or no joints. Same as Equation (4) and applying an average $MR = 400$ for rocks

Table 1 - Existing Empirical Relationships to Estimate the Rock Mass Modulus (Contd.)

Related Parameter/s	Empirical Correlation	Equation Number	Author/s	Development Basis and Remarks from authors
Rock Mass Parameters	***Relationships with rock mass unconfined compressive strength (σ_{cm})***			
	$E_m = \sigma_{cm}/0.79\epsilon_{crit}$ (MPa); $\epsilon_{crit} = \sigma_c/E_i$, ϵ_{crit}- Critical Strain	… (8)	Li [17]	
	$E_m/E_i = (\sigma_{cm}/\sigma_c)^{2/3}$	… (9)	Galera et al. [18]	Data bases of [20], [23], other in-situ data bases, plot in linear and logarithmic scales, with $\sigma_m = \sigma_c e^{(RMR-100)/24}$
	Relationships with Rock Quality Designation (RQD)			
	$E_m/E_i = 0.0231$ RQD $- 1.32$	… (10)	Coon & Merritt [19]	Only applicable for RQD≥ 64.
	$E_m/E_i = RQD/350$; RQD<70	… (11.a)	Bieniawski [20]	Reduction factor on intact rock modulus to be adopted.
	$E_m/E_i = 0.2 + [(RQD - 70)/37.5]$; RQD>70	… (11.b)		
	$E_m/E_i =$ J ; J- Average joint spacing, related to RQD	… (12)	Kulhawy & Goodman [21]	
	RQD>57%; Equation (10)	… (13.a)	Gardner [22]	
	RQD<57%; $E_m/E_i = 0.15$	… (13.b)		
	$E_m/E_i = 0.2 * 10^{(0:0186RQD-1:91)}$-Lower bound	… (14.a)	Zhang & Einstein [3]	
	$E_m/E_i = 1.8 * 10^{(0:0186RQD-1:91)}$-Upper bound	… (14.b)		
	$E_m/E_i = 10^{(0:0186RQD-1:91)}$-Mean	… (14.c)		
	Relationships RMR			
	$E_m = 2RMR - 100$ (GPa); $RMR > 50$,	… (15)	Bieniawski [20]	Only applicable for RMR< 50.
	$E_m/E_i = 0.1 + [RMR/(1150 - 11.4RMR)]$	… (16)	Kulhawy [13]	Reduction factor on intact rock modulus; *Em/Ei*< 1.0.
	$E_m = 10^{(RMR-10)/40}$(GPa); 0 <RMR< 90	… (17)	Serafim & Pereira [23]	Upon back analysis of dam foundation deformations and based on data bases of [20] and [23]
	$E_m/E_i = \left(0.0028RMR^2 + 0.9e^{RMR/22.82}\right)/100$	… (18)	Nicholson & Bieniawski [24]	
	$E_m = 10^{(RMR-20)/38}$(GPa)	… (19)	Mehrotra (1992) [25]	
	$E_m = 300\, e^{0.07RMR} * 10^{-3}$(GPa)	… (20)	Kim [26]	
	$E_m/E_i = 0.5[1 - \cos(\pi * RMR/100)]$	… (21)	Mitri et al. [27]	
	$E_m = 10^{(RMR-10)/40} - 0.562$(GPa)	… (22)	Mohammad (1998) [28]	
	$E_m = 0.1\,(RMR/10)^3$(GPa)	… (23)	Read et al. [29]	

Table 1 - Existing Empirical Relationships to Estimate the Rock Mass Modulus (Contd.)

Related Parameter/s	Empirical Correlation	Equation Number	Author/s	Development Basis and Remarks from authors
Rock Mass Parameters (Cont.)	***Relationships RMR (Contd.)***			
	$E_m = 7(\pm3)\sqrt{10(\mathrm{RMR}-44/21)}(\mathrm{GPa})$	… (24)	Diederichs & Kaiser [30]	
	$E_m = 0.0736\, e^{0.0755RMR}(\mathrm{GPa})$	… (25)	Gokceoglu et al. [31]	A database of in-situ plate load and dilatometer tests
	$E_m/E_i = e^{(RMR-100)/17.40}$	… (26)	Ramamurthy [32]	
	$E_m/E_i = e^{-0.0035[5(100-RMR)]}$	… (27)	Ramamurthy [33]	
	$E_m/E_i = 10^{[(RMR-100)((100-RMR/4000)\exp(-RMR/100)]}$	… (28)	Sonmez et al. [34]	
	$E_m = 0.3228\, e^{0.0485RMR}(\mathrm{GPa})$	… (29)	Chun et al. [35]	
	$E_m = e^{(RMR-10)/18}(\mathrm{GPa})$	… (30.a)	Galera et al. [18]	Same as Equation (9). Improvement by 10% of (17)
	$E_m = 0.0876RMR\ (\mathrm{GPa})\ ;RMR \leq 50$	… (30.b)		Improvement by 15% of (17)
	$E_m = 0.0876RMR1.056(RMR-50)0.015(RMR-50)^2(\mathrm{GPa})\ ;RMR > 50$	… (30.c)		Improvement by 15% of (17)
	$E_m/E_i = e^{(RMR-100)/36}$	… (31)	Galera et al. [18]	Improvement by almost 40% of (18)
	$E_m = 0.0003RMR^3 - 0.0193RMR^2 + 0.315RMR + 3.4064$	… (32)	Mohammadi & Rahmannejad [36]	
	$E_m = 110\, e^{-\left(\frac{RMR-110}{37}\right)^2}(\mathrm{GPa})$	… (33)	Shen et al. [37]	
	$E_m/E_i = 1.14\, e^{-\left(\frac{RMR-116}{41}\right)^2}$	… (34)	Shen et al. [37]	
	$E_m = 113\, e^{-\left(\frac{RMR-113}{39}\right)^2}(\mathrm{GPa})$	… (35.a)	Muhammad & Mohammad [28]	
	$E_m/E_i = 1.12\, e^{-\left(\frac{RMR-124}{57}\right)^2}$	… (35.b)		
	Relationships with GSI			
	$E_m = 0.1451\, e^{0.0654GSI}(\mathrm{GPa})$	… (36)	Gokceoglu et al. [31]	A database of in-situ plate load and dilatometer tests
	$E_m = 0.33\, e^{0.064GSI}(\mathrm{GPa})$	… (37)	Hoek, (2004) [11]	
	$E_m/E_i = S^{(a)^{0.4}};\ S = e^{(GSI-100)/9};\ a = 0.5 + (e^{-GSI/15} - e^{-20/3})/6$	… (38)	Sonmez et al. [25]	Using a later version of GSI
	$E_m/E_i = S^{1/4};\ S = e^{(GSI-100)/9};E_i = 50\ GPa$	… (39)	Carvalho (2004) [38]	
	$E_m/E_i = e^{GSI/21.7}/100$	… (40)	Yang [7]	Reduction factor on intact modulus, based on GSI [39]
	$E_m = 0.0912\, e^{0.0866GSI}(\mathrm{GPa})$	… (41)	Ghamgosar [40]	

Table 1 - Existing Empirical Relationships to Estimate the Rock Mass Modulus (Contd.)

Related Parameter/s	Empirical Correlation	Equation Number	Author/s	Development Basis and Remarks from authors
Rock Mass Parameters (Cont.)	***Relationships with Q***			
	$E_m = 25\log_{10} Q$(GPa); $1 < Q < 400$	… (42)	Hoek et al. [41]	Generally, recommend for hard rocks
	$E_m \approx 10\,(Qc)^{1/3}$ (GPa); $Qc = Q\,\frac{\sigma_c}{100}$	… (43)	Barton [42]	
	$E_m = 1.5E_i^{0.14} Q^{\,0.6}$ (GPa)	… (44)	Singh (1997) [43]	
	$E_m = 8Q^{\,0.4}$[6]; $1 < Q < 30$	… (45)	Palmström & Singh [4]	For moderately jointed rock masses. Data bases of [20], [23], and others, adjustment of in-situ test results for the blasting damage and replot.
	$E_m = E_i e^{(0.8625\log Q - 2.875)}$ (GPa)	… (46)	Ramamurthy [32]	
	$E_m = E_i e^{-0.0035[250(1-0.3\log Q)}$ (GPa)	… (47)	Ramamurthy [33]	
	Relationships with RMi			
	$E_m = 5.6RMi^{\,0.375}$; $0.1 < RMi < 1$	… (48)	Palmström & Singh [4]	For moderately jointed rock masses
	$E_m = 7RMi^{\,0.4}$; $1 < RMi < 30$	… (49)	Palmström & Singh [4]	Recommended for Massive rocks (limited accuracy for σ_c< 100 MPa), with few no joints. Same as (45)
Relationships as a combination of parameters	***Relationships with σ_c and GSI***			
	$E_m = \sqrt{\frac{\sigma_c}{100}}\,10^{(GSI-10)/40}$ (GPa); $\sigma_c < 100MPa$	… (50)	Hoek & Brown [44]	Practical observations, back analysis of excavation behaviour in poor quality rock masses, modifying (17).
	$E_m = \tan(\sqrt{1.56 + [\ln GSI]^2})\,\sqrt[3]{\sigma_c}$(GPa)	… (51)	Beiki et al. [45]	Upon a database of 150 data sets using the genetic programming approach.
	Relationships with σ_c, GSI and RQD			
	$E_m = \tan[\ln(GSI)]\log(\sigma_c)\sqrt[3]{\mathrm{RQD}}$(GPa)	… (52)	Beiki et al. [45]	Upon a database of 150 data sets using the genetic programming approach.
	Relationships with D and GSI			
	$E_m = (1-0.5D)\,10^{(GSI-10/40)}$(GPa), σ_c >100MPa	… (53)	Hoek et al. [46]	D- degree of disturbance due to blast damage
	$E_m/E_i = S^{(a)^{0.4}}$; $S = e^{(GSI-100)/9-3D}$; $a = 0.5 + (e^{-GSI/15} - e^{-20/3})/6$	… (54)	Sonmez et al. [25]	
	$E_m/E_i = S^{3/4}$; $S = e^{(GSI-100)/9-3D}$	… (55)	Carvalho (2004) [28]	
	$E_m = 10^5[(1-0.5D)/(1+e^{(75+25D-GSI)/11})]$(GPa)	… (56.a)	Hoek & Diederichs [38]	From a large number of in situ measurements from China and Taiwan on studies carried out for degree of disturbance due to blast damage and stress relaxation.
	$E_m = E_i[0.02 + (1-0.5D)/(1+e^{(60+15D-GSI)/11})]$ (GPa)	… (56.b)		
	$E_m = 120(1-42D)\,e^{-([GSI-120+42D]/46)^2}$(GPa)	… (57.a)	Muhammad & Mohammad [28]	
	$E_m/E_i = (1+0.52D)\,e^{-([GSI-120-52D]/54)^2}$	… (57.b)		
	$E_m = (1-0.5D)\sqrt{\frac{\sigma_c}{100}}\,10^{(GSI-10)/40}$ (GPa); $\sigma_c \le$100MPa	… (58)	Hoek et al. [46]	

Table 1 - Existing Empirical Relationships to Estimate the Rock Mass Modulus (Contd.)

Related Parameter/s	Empirical Correlation	Equation Number	Author/s	Development Basis and Remarks from authors
Relationships as a combination of parameters	***Relationships with RQD and WD*** $E_m = 0.135[E_i(1 + RQD/100)/WD]^{1.1811}$(GPa) WD- degree of weathering	… (59)	Kayabasi et al. [47]	With half of the data points in the entire database
	Relationships with σ_c, RQD and WD $E_m = 0.001[[(E_i/\sigma_c)(1 + RQD/100)]/WD]^{1.5528}$(GPa)	… (60)	Gokceoglu et al. [31]	Linear, power, exponential and logarithmic functions were separately considered and developed a chart to estimate E_m, which relates E_m, RQD and modulus ratio (Ei/σ_c)
	Relationships with J, MR and σ_c $E_m = j * MR * \sigma_c$; MR- modulus ratio	… (61)	BS 8004:1986 [48]	
	Relationships with Q and σ_c $E_m = 10\ (Q\ \sigma_c/100)^{1/3}$; $\sigma_c = 100\ MPa$	… (62)	Barton [42]	
	Relationships with Q and H $E_m = H^{0.2}Q^{0.36}$ (GPa); H- overburden	… (63)	Singh (1997) [43]	
	Relationships with RMR and H $E_m = 0.3H^{\alpha}10^{(RMR-20)/38}$ (GPa); α = 0.16 or 3.0 (higher value for poor rocks), when σ_c <100 MPa and H (in meters and ≥ 50m)	… (64)	Verman (1997) [43]	Upon the concept of Equation (1) and transformed it to rock mass considering the lateral stress is expressed in terms of depth (H)
	Relationships with RMR, σ_c and σ_{cm} $E_m/E_i = (\sigma_{cm}/\sigma_c) = e^{(RMR-100)/22}$	… (65)	Ván & Vásárhelyi [49]	
	Relationships with MR, RMR and σ_{cm} $E_m/\sigma_{cm} = MR * e^{2(RMR-100)/100}$	… (66)	Ván & Vásárhelyi [49]	
	Relationships with Joint Parameters $E_m = -7.192 + 0.06469\sigma_c + 0.20418RQD + 0.30974JS + 0.38384JC + 0.1716GW$ (GPa)	… (67)	Nejati et al. [50]	

in stress-strain behaviour of rock mass which depends on the loading pattern of different structures, ranging from dams, tunnels, slopes, and foundations to socketed anchors and shafts.

Moreover, many have pointed out that results of the rock mass classifications may vary considerably depending on the attributes of the rock engineer, the measuring system applied and the type of the project. Thus, parameters that are derived based on such systems will have a greater ambiguity [51]. In order to estimate the E_i, (when the laboratory data is not available) the following equation proposed by Hoek and Diederichs [38] can be objectively adopted:

$$E_i = MR * \sigma_c \quad \dots (68)$$

where MR is the Modulus Ratio, and σ_c is the uniaxial compressive strength.

2.2.3 Analytical Methods

Li [52] proposes a graphical method to represent the deformation modulus of rock, which ultimately yields a useful analytical solution to determine the E_m for jointed rocks.

Ebadi et al. [53] have incorporated the effects of lateral stress (intermediate principal stress and minimum principal stress) on the analytical solutions proposed by Li [52] for the E_m of jointed rocks.

Zoorabadi [54] identifies the shortcomings of the empirical approach and proposes an extended analytical solution to estimate the E_m of jointed rock, which has been earlier proposed by Li [52]. This is with a combination of the geometrical properties of discontinuities and E_i and finally incorporating the confining effects.

2.3 Back Analysis Method

Back analysis methods in rock engineering practice are classified into three broad categories: stress back analysis, displacement back analysis, and strain back analysis. Out of the three options, the displacement measurement is the easiest and most convenient method and hence it is widely adopted in the derivation of rock related engineering parameters [55].

Historically, back analysis has been used often to estimate the E_m in tunnels and especially in rock sockets [15, 56, 57, 58, 59, 60]. In some of the studies, the technique has been used to develop new equations or verify the validly of existing empirical equations to estimate E_m while some have been used to verify the compatibility of different design criteria, with the comparison of results derived through back analysis and in-situ E_m. Moreover, in some cases, this technique has been used simultaneously to develop empirical equations to estimate E_m, while verifying the accuracy of new design criteria.

3. Review on the Adoptability of Existing Rock Mass Deformability Estimation Mechanisms in Different Rock Masses

It has been identified that there are a substantial number of controlling factors on the value of E_m, either in the direct or indirect form. In order to choose the most appropriate value for the design, it will be beneficial for a rock foundation engineer to consider the aspects discussed below.

3.1 Factors to be considered in using Laboratory and In-situ Test Results

3.1.1 Factors that affect the Deformability Modulus of In-tact Rock

Modulus of elasticity of intact rocks increases with increase in the rate of the applied stress, and so does the axial strain at failure. The diametrical strain at failure decreases with increase of loading rates [61]. Similar observations have been made by Malik et al. [62] for brittle Basalt, which is highly sensitive to strain rate.

Hsieh et al. [63] report that actual elastic behaviour of in-tact rock samples is non-linear, due to the coexisting processes of closure, sliding and compaction of pre-existing cracks within the elastic stress range of the intact sample, and hence the derived tangent modulus is found to be stress dependent.

It has been shown that intact rock deformation modulus increases significantly with the confining stress [11]. This fact has even been confirmed by Ebadi et al. [53] by using analytical techniques for Schist.

However, it has been revealed that, when rocks are tested under tri-axial conditions, the non-linearity and the stress dependency of their elastic behaviour is minimal for hard, crystalline or homogeneous rocks of low porosity, while these factors significantly affect porous, clastic or closely jointed rocks [12].

3.1.2 Factors that Affect the Deformability Modulus of Rock Mass

It has been shown that rock mass moduli values obtained from different test methods even for the same rock mass produce significantly different results [3]. According to Bieniawski [20], a single testing method, such as the flat jack test (FJT), can lead to a wide scatter in the results even where the rock mass is very uniform.

When comparing the results obtained from different test methods, generally, the values obtained through GJT and the plate loading test (PLT), both produce lower results compared to plate jacking test (PJT) and on average these should be multiplied by a factor Rp = 2.5 to be compared with the PJT measurements [4]. Moreover, Palmström and Singh [4] comment that PJT measured by extensometers in drill holes gives generally the best results. Interestingly, Pressuremeter/Dilatometer test has been found to produce lower results compared to GJT, PLT and FJT results [5].

Contradictorily, Galera et. al. [18] suggest that Borehole Expansion Tests (mostly PT) are found to produce the best results.

3.2 Factors to be Considered in using Results obtained from Empirical Estimation Methods

Annexure 1 comprehensively discusses the merits and demerits of the empirical equations reported in Table 1. This is a summary of the reviews made by different authors on the respective equations following the comparison of results reached through in-situ tests, reanalysis with additional in-situ data or re-evaluation and refinement of the same equations through advanced statistical packages, techniques and evaluation methods.

It is also noted that the empirical methods do not consider either the effect of scale and stress on rock mass deformability or the anisotropy of rock mass deformability.

It has been identified that the estimated values from various empirical methods can be very different for some of the rock masses. It is also noted that the highest or lowest estimated values are not from a single empirical method. For example, an empirical method may give the highest or lowest estimated value for one rock mass but an estimated value in the mid-range for a different rock mass. One possible reason is that the empirical methods were developed based on databases of different sources.

Therefore, it is difficult or impossible to decide which method is the most accurate for a given rock mass. Nevertheless, most of the experienced researchers advise the use of at least two or more empirical methods in the evaluation of E_m, coupled with a direct estimate from one in-situ testing and one obtained from an indirect geophysical method.

3.3 Adaption of Results Obtained from Different Mechanisms for Different Rock Masses

3.3.1 Rock Mass Parameters

Quoting the findings of many researchers, Zhang [11] states that it becomes a very challenging task to precisely determine the E_m value for a rock mass due to different types of discontinuities such as joints, bedding planes, folds, shear zones and faults contained in natural bedrock masses.

Basically, deformation modulus of a rock mass is made up of two components: one due to deformation of the intact rock; the other due to the deformability of the joints and discontinuities [64] and hence it depends on the Young's Modulus of rock type and shear strength of joints [65].

Generally, it has been observed that the deformation behaviour of better-quality (say RQD >50%) rock masses is controlled by the geological discontinuities; while for poorer-quality rock masses (say RQD<50%) the deformation of the intact rock pieces contributes to the overall deformation process [44]. Within better-quality rock masses, the intact deformability is mainly controlled by the embedded weaker intact rock pieces [35].

The effects of joints have been analytically established by Li [52], who concluded that the deformability modulus ratio (E_m/E_i) (also referred to as modulus reduction ratio [19]) of a rock block containing a through going single joint set reaches its minimum (around 0.3) when the loading angle (measured from normal to the discontinuity plane), $\theta = 0^0$ when $k_s \geq k_n/2$ (k_s-joint shear stiffness, k_n-joint normal stiffness) and the ratio reduces to its minimum level at around $\theta = 45^0$ when

$k_s = 0$. In both cases he has observed that E_m/E_i ratio approaches 1 when $\theta = 90^0$. Based on his stereographic projection analysis, he concluded that E_m reduces to as far as $0.15E_i$ to $0.4E_i$ in rock masses of 3 joint sets of different dips and dip directions with constant joint spacing, shear and normal stiffness levels. Following the findings of Li [52], Ebadi et al. [53] have analytically shown that, for a rock block containing several joints, the variation of the E_m/E_i ratio is insignificant for increase in the lateral stresses when $\theta < 70^0$, while this ratio dramatically increases and approximately is the same for all lateral stress ratios (σ_1/σ_3 ranged from 0 to 5 and σ_1/σ_2 ranged from 1 to 3.3) when $\theta > 70^0$. Moreover, the analysis has also revealed that E_m increases with increase in joint spacing due to the lesser extent of rock fracturing, and increases of E_m beyond a spacing of 0.1 m is insignificant, e.g., it does not reach E_i even at a spacing larger than 1.0 m. Theoretically, Ebadi et al. [53] have observed that increase in E_i causes increase in E_m in a rock block with a single joint set and it is mainly due to comparatively lesser overall rock displacement contributed by the intact zones having greater elastic range.

When considering the rock discontinuity parameters, Rock Quality Designation (RQD) is the simplest parameter; but it is only one of the joint related factors that affect the deformation modulus of rock masses and it does not cover other joint related characteristics [3]. Therefore, expressions based on RQD provide least reliable results [19, 20, 65]. Moreover, Zhang and Einstein [3] highlight the fact of directional variation of RQD in fractured rocks coupled with insensitivity of RQD to discontinuity frequency, which intensifies the deviation of actual E_m from values derived through RQD dependent empirical formulae. However, Zhang [11] proposes volumetric discontinuity frequency or core boring, scanline sampling and/or wave velocity measurements at different directions to determine an average RQD for the rock mass to eliminate the directional dependence of RQD and thus E_m. Despite the deficiencies of RQD, it has been identified to have a greater indirect bearing on the rock mass deformation modulus [66].

It is recommended that Rock Mass Rating (RMR), which is the next common parameter used in the evaluation of E_m, should not be applied for massive rock masses [4], while it is observed to produce better results when it is employed in jointed rock masses. Yang [5] identifies the inherent drawbacks in using RMR, especially in estimating the parameter for very poor-quality rocks [44]. Nejati et al. [50] state that RMR based empirical equations provide satisfactory results. They also observe that five RMR rating parameters have a direct but different individual level of influence on the E_m value. The influence is greater from joint related parameters and is least from groundwater conditions.

When considering applicability of Q system, which is popular mainly in the tunnelling field, Palmström and Singh [4] recommend it to be adopted in estimating the E_m in very strong (σ_c> 150 MPa), massive rocks. Similar to RQD, the directional dependency of Q on estimating E_m has been proposed to be eliminated by adopting an oriented Q_0 and normalised Q_c using an oriented RQD_0, and a J_r/J_a(J_r-rating for joint surface roughness of least favourable set or discontinuity, J_a -rating for joint alteration, discontinuity filling of least favourable set or discontinuity) ratio relevant to the loading or measurement direction [42].

Yang [5] proposes Geological Strength Index (GSI) as an alternative to RMR to estimate E_m, to capture the missing information in RMR through the physical appearance of the recovered core sample material. However, this has been later challenged by Galera et al. [18] quoting the inherent empiricism involved in GSI estimation instead of advanced quantitative data, and it is only recommended to be adopted for weak poor-quality rock masses with RMR<20.

Amongst the different rock mass classification systems, a more recently developed Rock Mass Index (RMi) is found to produce better estimates in jointed rock masses compared to Q system, while it performs better than both RMR and Q systems in massive rocks masses [4]. As described earlier, use of RMR is preferred over equations with RQD alone, or use of GSI. Ramamurthy [32] points out that neither of the aforementioned classification systems produces satisfactory results on modulus ratio (MR), as the change in the MR from very good to very poor-quality rocks is insignificant and thus proposes joint factor (J_f) model to estimate the modulus ratio, which is found to be more sensitive to rock quality. Similar argument has been made by Sonmez [34] on RMR, Q and GSI, who observes that these systems yield unacceptably high deformation moduli (greater than intact elastic modulus) for high quality rock masses composed of soft intact rock zones and suggest to give more emphasis on the deformation behaviour of intact rock zones than the discontinuity conditions for such rock masses. Generally, the deformation modulus evaluated from classification systems seems to be valid only for the strongest rocks and found to generate significantly higher values for weak rocks than the relevant in-situ value [4].

Considering the deficiencies identified in empirical equations based on different rock mass classification systems, researchers have sought relationships which involve both intact as well as rock mass classification parameters. Most of the findings are encouraging as equations which incorporate E_i with rock mass classification parameters are found to produce better results [5], [19], [34]. E_i seems to be considerably highly correlated with E_m as well as the other rock mass classification parameters, while the correlation between σ_c and E_m is found to be the least [5].

3.3.2 Rock Anisotropy

The directional dependency of the engineering parameters of rock masses arises predominantly due the discontinuity orientation and its engineering behaviour, and secondarily due to the effects of rock grains.

The mechanical effects rock mineralogy and rock texture become important in the evaluation of E_i in rock masses where the discontinuity spacing is considerably large. The most conveniently identifiable feature with respect to above aspects is schistosity, common in metamorphic rock masses.

Quoting a number of references, Zhang [11] states that around 75%-45% reduction in E_m is observed through a change of the direction of deformation modulus measured parallel to stratification plane to that measured perpendicular to stratification plane. In order to alleviate this effect, Sonmez et al. [34] propose to adopt E_i and RMR in the directions parallel to and perpendicular to such laminations and to come up with a two moduli approach in the corresponding two directions.

3.3.2 Stress Dependency

Another main drawback in the empirical equations is the disregard of the stress factor. Similar to the case of in-tact rock, E_i, Torbica and Lapčević [65] state that E_m is also stress dependent (both vertically imposed and lateral) and they suggest that the variation of E_m in jointed rock mass approaches the behaviour of E_m of monolithic rock beyond a certain depth because shear strength of rock joints tends to increase to the state representing monolithic conditions when depth is large. This has been analytically proven and the stress effect is pronounced in jointed rock masses, especially towards the ground surface due to greater deformability of discontinuities along with block rotation [54]. Schock [67] has experimentally proven a similar finding where dynamic elastic modulus approaches the value of its static equivalent due to the closure of voids.

3.3.3 Scale Effect

Obviously, scale effect has a great bearing on E_m as larger the test volume greater the effects of discontinuities. Based on analysis of a large number of laboratory data and corresponding field test data, it has been revealed that volumetric change in the test sample from laboratory scale (~10^{-3} m^3) to field scale (~10^3 m^3) will cause a reduction of around 67%; the larger the test volume, lower the E_m and lesser the variability of results [18].

As test volume increases, this reduction can even be between 20% to 60% of the instrumented laboratory uniaxial compression test values on intact samples as identified by Heuze [68].

This fact is evident in most of the instances as directly obtained E_m values are different from the E_m magnitudes derived from the back analysis of the elastic component of the load-displacement curves of actual structural load applications, especially as the latter cases depend on the actual volume of rock influenced by the load application.

3.4 Adoptability of E_m into Design Practise

Based on the above discussion there are two basic means of obtaining the E_m for the design, viz., through field tests and through empirical means.

When adopting the direct in-situ test results, many practitioners recommend to perform at least two types of in-situ tests (e.g., plate bearing test and dilatometer test) in the same location to alleviate the discrepancies between results obtained from different in-situ test methods. However, except for very sensitive structures, such strategies are limited in practice due to time and cost involved.

In adopting empirical systems, Sonmez et al. [34] propose at least one rock classification system to be used to incorporate the effects of discontinuity properties, while Barton et al. [69] propose to adopt multiple rock mass classification systems for the same site to arrive at a reasonable value for E_m.

When using the rock mass classification systems, the limitations of the respective classification systems should be borne in mind and it is recommended not to apply any correlations or transition equations between the systems (as suggested by different authors), as mathematical equations which are of different levels of accuracy can produce substantially misleading results and may give rise to incorrect values. Instead, as a good practice, the various parameters involved in the actual systems should be given their relevant ratings and the classification value for each system needs to be arrived independently [4]. Moreover, when obtaining the rock mass classification parameters, it will be imperative to obtain only the significant and intrinsic parameters of the rock which reflect the rock mass behaviour and each parameter must represent itself exclusively. Most importantly, parameters so obtained should be easily measurable and be linked in such a way that the quality of the rock mass is reflected in terms of its strength and modulus to capture the reduction in strength and deformability from its intact form [32].

To optimise the procedure, National Academies of Sciences, Engineering, and Medicine (NASEM) [39] proposes a sequence of steps to be adopted in the design process. It suggests to initiate with a site in-situ test (such as borehole jack) and then to predict E_m by an appropriate empirical correlation and carryout a cross-check on the in-situ measured values. As the third step, it proposes to perform a geophysical method (such as downhole seismic; compression wave velocity), which generally provides a reasonable upper-bound check on the rock mass modulus. In order to reconfirm an upper bound solution, NASEM [39] proposes to perform laboratory uniaxial compression tests, to test the consistency with the observations of Heuze [68] to ascertain whether the field rock mass modulus values are in the range of 20% to 60% of intact rock modulus. To check the accuracy reached, it mentions that the mean value of E_m determined from the in-situ tests shall be in the range of the values predicted from the empirical correlations.

Since it has been identified that rock mass classification systems provide unreasonably higher E_m values for weak massive rocks, it is recommended to estimate the deformation value by laboratory test results and reasonably adjust for the scale effect [4].

In mines, the intact rock properties (E_i) are simply downgraded and used as inputs for numerical modelling. The output of the model is then calibrated based on the actual observations made from rock mass behaviour, through which the rock mass parameters are fine-tuned. Based on the results of large number of data bases, numerical modelling, and experience of mining site-based practitioners, it has been established that the relationship $E_m \approx 30\% - 50\%$ of E_i can be adopted in mine designs [70]. Considering the wide uncertainty involved in obtained values for E_m, Bieniawski [20] recommends an in-situ modulus of deformation with an accuracy of more than 20% will be sufficient for practical design purposes.

The values so obtained shall be further fine-tuned for the disturbance, which is usually a common phenomenon in mines and tunnels due to blasting and mechanical excavation. Appropriate disturbance factors (D) ranging from 0 (excavation with minimum disturbance) to 1 (production/poor blasting) have been proposed by Hoek et al. [46] for different excavation criteria and the respective factors can be appropriately incorporated to the empirical formulae used to obtain the E_m.

The importance of rock mass deformability arises in the design of rock sockets for pile foundations when elastic solutions are employed in the estimation of the bearing capacity components (both skin friction and end bearing) of the rock sockets. Rowe and Armitage [15] propose to use Equation (5), in the estimation of E_m for their proposed elastic solutions, with a partial safety factor of 0.70 to compensate any uncertainties, while Williams and Pells [59] propose Equation (12) to estimate the reduction in lateral confinement (E_m/E_i) in the estimation of ultimate skin friction of rock sockets. O'Neill et al. [71] propose modulus reduction factors presented in Table 2 by considering the joint characteristics, for the design of rock sockets. In the design of rock sockets, Load and Resistance Factor Design for Bridge Design Specifications (LRFDBDS) [72] proposes to adopt the least of the two values obtained for E_i, obtained directly from intact core sample test and from Equation (69).

Table 2 – Modulus Reduction Factors Based on RQD Levels [71]

RQD (%)	E_m/E_i	
	Closed Joints	Open Joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

$$E_m = E_i \left[\frac{E_m}{E_i}\right]_t \qquad \dots (69)$$

where,

E_i – Obtained from intact core sample test, and

$\left[\frac{E_m}{E_i}\right]_t$ - Obtained from Table 2.

4. Conclusions and Recommendations

Based on this review work, following conclusions and recommendations can be made.

1. The intact deformability modulus (E_i) results obtained from laboratory tests shall be post treated for stress and strain rate dependency as well as for the confining stress, especially for porous, clastic or closely jointed rocks.

2. PT test is found to produce the lowest results among other in-situ methods, followed by GJT, PLT and FJT. The highest values are produced by PJT and the value is generally around 2.5 times the GJT and PLT generated values.

3. Considering the practical adoptability and accuracy, PJT measured by extensometers in drill holes generally produces the best results, while PT is found to perform well for weak but better quality (less fractured, RQD>50%) rock masses. For strong crystalline and highly fractured rock masses, GJT is preferred over PT.

4. Most of the empirical estimation criteria are insensitive to stress, scale and anisotropy. The actual deformability behaviour of the rock mass is stress dependent both vertically and horizontally, while the effects of vertical stresses become negligible after a certain depth. E_m reduces from 20% to as far as 67% as the test volume increases from laboratory scale to in-situ scale. Orientation of discontinuities further downgrade the estimated E_m value of a jointed rock mass with a single set of joints to a value of around 0.3 E_i, when the loading angle is 0^0 at higher joint shear stiffness levels and reaches its minimum value when the loading angle is 45^0 in cases where joint shear stiffness is minimum. When the loading angle is 90^0, E_m can be approximated as E_i under any joint shear stiffness level. Moreover, the reduction in E_m will be as low as 0.15 E_i in a rock mass of 3 sets of joints. It has been further reported that lateral stress effects on E_m of jointed rocks has a profound effect when the loading angle increases beyond 70^0. The value obtained depends also on joint spacing (especially <0.1 m) and schistosity (~75% to 45% reduction).

5. Depending on the rock mass quality and the intact rock strength levels, following adjustments are recommended.

   (a) Since the deformation behaviour of better-quality (RQD>50%) rock masses is controlled by the geological discontinuities, special attention is needed and results should be adjusted for discontinuity orientation as mentioned in Conclusion 4.
   (b) However, in better-quality rock masses, if weaker intact rock zones occur in between the discontinuity planes, then it is emphasised to be alert on intact deformability behaviour in such zones and appropriate post treatments on intact parameters should be carried out as mentioned in Conclusion 1, and for the anisotropic characteristics such as schistosity as mentioned in Conclusion 4.
   (c) Attention and treatments as recommended in Conclusion 5(b) can be recommended for poorer quality rock masses as well as the deformation of the intact rock pieces contributes to the overall deformation process for such rock masses.

6. Based on the summary of reviews on empirical equations presented in Annexure 1, Equation (1) is found to perform well in accommodating the lateral confining stress correction on E_i. For the estimation of E_m,

   (a) for the purposes of general rock engineering applications:

      i. RMR based: Equations (15) and (17) are recommended for moderately jointed rocks (RMR>30), while Equations (19), (23) and (28) have been found to perform comparatively well in all the rock mass types. For weak rock masses, Equation (29) has been proven to work well.
      ii. Q based: Equation (42) is found to be valid for rock masses with $Q > 1$, while Equation (43) performs well for hard-fractured (strong-poor quality) rock masses. For weak-fractured (poor quality) rock masses, Equation (44) is recommended. Moreover, the overburden factor (H) incorporated in Equation (63) produces satisfactory results for weak fractured (poor quality) rock masses under dry or nearly dry conditions.
      iii. GSI based: Equation (50) is found to perform well in weak (σ_c<100MPa) rock masses. The disturbance factor (D) incorporated in Equations (56.a) and (56.b) is found to perform well in all rock mass types.

   (b) for the purpose of foundation designs, σ_c based Equation (5) is recommended in rock-socketed pile foundation designs, while RMR based Equation (18) and GSI based Equation (40) are also recommended in the design of foundations. However, Equations (53) and (58), which are based on GSI, are found to be inappropriate for the foundation design aspects, especially in weathered rock masses.

   (c) for massive (least jointed) -weak rocks, it is recommended to adopt the laboratory estimated E_i and adjust for scale effect [4].

   Nevertheless, it should be noted that this review is based on the existing empirical mechanisms recommended by different researchers with different data bases, parameters and analytical techniques. Hence the above conclusions and recommendations may be subjected to change based on future findings.

7. Considering the outcomes mentioned in Conclusion 6, it is obvious that, different empirical criteria generate different values of E_m for the same type of rock mass. Hence practitioners recommend to use at least two or more empirical methods in the same location and estimate a reasonable value for E_m. If the results need to be more accurate, additional one direct in-situ test and one indirect geophysical method are further recommended to be performed to alleviate the discrepancies arising from different mechanisms.

8. Rock mass deformability estimated through back analysis mechanism is found to be used in many rocks related engineering applications, especially in tunnel support design, and is quite popular in rock-socket design in the construction of bored piles due to the comparatively greater depth at which the parameter needs to be estimated using direct in-situ test method.

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Annexure 1- Reviews on Existing Empirical Relationships to Estimate the Rock Mass Modulus

Equation Number	Review comments	Review Reference
(1)	➢ Confirmed the adoptability through reverse analysis based on the measured data from several tunnels	Verman (1997) [43]
(5)	➢ Design value which could be used for settlement calculations can be $E_d = 150\sqrt{\sigma_c}$ applying partial factor of 0.7 for pile design	[15]
(10)	➢ RQD<60% range is not covered and only an arbitrary modulus reduction factor value is proposed.	[3]
(13)	➢ For RQD=100%, E_m is assumed to be equal to E_r. This is obviously unsafe in design practice because RQD=100% does not mean that the rock is intact. There may be discontinuities in rock masses with RQD=100% and thus E_m may be smaller than E_r even when RQD=100%.	
(15)	➢ The obvious deficiency of this Equation is that it gives negative modulus values when RMR < 50 ➢ Recommend for moderately jointed rock masses for 55 < RMR < 90, which is the originally recommended range. {Using the data bases of [20], [23], Clerici (1993), CSMRS and Thorpe et al., (1980), adjustment of in-situ test results to one comparable base and then readjustment for the blasting damage and plot} ➢ Yielded highly scattered results ➢ May not provide a reasonable fit for the field measured data, since they are not applicable for a wide range of RMR values ➢ Only applicable for good quality rock masses with RMR > 50 ➢ Exhibited reasonable fit to the field data ➢ Generated the most scattered results	[11], [50] [4] [31] [64] [28] [38] [47]
(17)	➢ Recommend for moderately jointed rock masses of 30 < RMR < 55 and perform poorly for RMR<30 ➢ Performs well for good quality rocks; for poor quality rocks it over predicts ➢ Overestimates for lower range i.e. RMR< 10 [Mohammad, 1998] and upper range i.e. RMR > 90 [38] ➢ Reasonably fits to the available case history data, covers a wider range of RMR values than Equations (15) and (42) ➢ Exhibited reasonable fits to the field data ➢ Generated acceptable results	[4] [11], [41] [28] [41] [38] [47]
(18)	➢ Exhibited good results ➢ Based on the evaluation carried out by [31], could be adopted in foundation design. Equation (18) provides better estimate on E_m compared to equation (58) and equation (59) which over-predict E_m. ➢ Provides better predictions (especially for Dilatometer test data). But Equation (59) provides the best predictions. ➢ More representative curve fit to the data ➢ Provides better prediction performance compared to Equation (14.c), (23), (38), (39), (53), (56.b) and (58) ➢ Poor estimates of the deformation modulus for massive rock because of the poorly defined asymptotes ➢ Exhibited the most acceptable results	[31] [5] [31] [34] [45] [38] [47]

Annexure 1- Reviews on Existing Empirical Relationships to Estimate the Rock Mass Modulus (Contd.)

Equation Number	Review comments	Review Reference
(19)	➢ Provides better prediction performance compared to Equation (14.c), (23), (38), (39), (53), (56.b) and (58)	[45]
(20)	➢ Extremely overestimates the deformation modulus rock mass for RMR >85	[28]
(21)	➢ Appears to overestimate the rock mass deformation modulus. When RMR ≥ 57, the ratio of deformation modulus of rock mass to Young's modulus of intact rock using Equation (21) is greater than 0.6, which contradicts Heuze[68] findings ➢ Overestimates E_m ➢ Produced higher scattered results, based on Root Mean Square Error (RMSE) results ➢ Overestimate E_m than observed the in-situ values ➢ Exhibited reasonable fits to the field data but give relatively poor fits to the full range of data ➢ Generated the most scattered results	[64] [29] [31] [34] [38] [47]
(22)	➢ Overestimates the deformation modulus rock mass for upper range of RMR	[28]
(23)	➢ Exhibited reasonable fits to the field data	[38]
(24)	➢ Estimates a very high value of deformation modulus for low range of RMR and very low value for upper range of RMR ➢ Exhibited reasonable fits to the field data	[28] [38]
(27)	➢ Overestimates the E_m/E_irock mass for lower range of RMR	[28]
(28)	➢ Provides better prediction performance compared to Equation (14.c), (23), (38), (39), (53), (56.b) and (58)	[45]
(29)	➢ Corresponds to weak rocks	[28]
(32)	➢ Due to a polynomial equation with poor asymptote, it over estimates the deformation modulus RMR < 25 and overestimate for RMR when approaching to 100	[28]
(38)	➢ Overestimate E_m for lower values of RMR (RMR<50) but yield lower values for higher RMR (60<RMR<80) ➢ Predicts E_m in plus or minus factor of two, i.e., in between the boundary lines where, lower line represents a 100% overprediction and upper line represents a 50% underprediction ➢ Poor estimates of the deformation modulus for massive rock because of the poorly defined asymptotes and give relatively poor fits to the full range of data	[34] [25] [38]
(39)	➢ Poor estimates of the deformation modulus for massive rock because of the poorly defined asymptotes and give relatively poor fits to the full range of data	[38]
(40)	➢ Proposed for foundations design (For preliminary design or where in-situ tests are not available) ➢ Reduction factor needs to be adopted on E_i, based on GSI	[5] [39]
(41)	➢ Extremely overestimates the deformation modulus rock mass for GSI > 80	[28]

Annexure 1- Reviews on Existing Empirical Relationships to Estimate the Rock Mass Modulus (Contd.)

Equation Number	Review comments	Review Reference
(42)	➢ Recommend for moderately jointed rock masses for Q>1 ➢ May not provide a reasonable fit for the field measured data, since they are not applicable for a wide range of RMR values ➢ Valid only for Q > 1 as, when Q<1, estimate negative value of $E_{m.}$ ➢ Applicable for hard rocks Q>1. However, it served very well, for instance in UDEC-BB distinct element (i.e., jointed) 2- dimensional modelling of the Gjvik cavern [16], but already a need for a depth or stress correction was recognized ➢ Can be used for fractured hard rocks	[4] [40] [28] [42] [43]
(43)	➢ Is the most representative when compared with the Equations (15) and (17). ➢ Suitably fits the data on which it was developed ➢ Can be used for fractured hard rocks ➢ Exhibited reasonable fits to the field data	[31] [28] [43] [38]
(44)	➢ Revealed to be used for weak fractured rocks	[43]
(47)	➢ Found to be more realistic compared to Equation (46) as it produces a reasonable E_m/E_i ratio	[28]
(50)	➢ Agrees quite well with the other results ➢ For weak rocks (σ_c<100MPa), Equation (50) produced the best predictions, followed by Equation (59) ➢ Limited to weak rocks (σ_c<100MPa), it extremely overestimates the deformation modulus of rock mass due to its poor asymptote ➢ Provides better prediction performance compared to Equation (14.c), (23), (38), (39), (53), (56.b) and (58) ➢ Exhibited acceptable results ➢ Sometimes overestimates the deformation E_m in such manner that the obtained value is larger than Young's modulus of the monolith rock	[10] [31] [28] [45] [47] [58]
(51)	➢ Based on limited data with GSI range from 26 to 82, majority occurring in range of 45 to 65 and gives negative deformation modulus values when extrapolated to extreme	[28]
(53)	➢ Exhibited the best results for a weak marly rock mass, having a mean uniaxial compressive strength of 18.60 MPa. ➢ However, it has been found that this equation was not appropriate for foundation design on weathered rock. {From field test results in Hong Kong for igneous, volcanic and metamorphosed sedimentary rocks} ➢ Limited for strong rock (σ_c>100MPa) mass, which overestimates the value of E_m for upper range of GSI.	[31] [5] [28]
(54) (55)	➢ Both equations assumes that the ratio of the modulus ratios of rock mass and intact rock is equal to unity, when GSI=100 but in both equations the damage factors are not incorporated properly, which cause to produce the same value of deformation modulus for all value of D (i.e. 0 to 1) when GSI = 100.	[28]
(56.a)	➢ Has good prediction performance. ➢ Provides better prediction performance compared to Equation (14.c), (23), (38), (39), (53), (56.b) and (58).	[28] [45]

Annexure 1- Reviews on Existing Empirical Relationships to Estimate the Rock Mass Modulus (Contd.)

Equation Number	Review comments	Review Reference
(56.b)	➢ Assumed to be more authentic {With larger in-situ tests database, variety of rock masses considered and a mathematical function selected which truly represents the trend of the scatter data} ➢ The use of equation is limited due to lack of user friendliness {Intact rock samples for E_i are not always taken from behind the in-situ test site and are not always true representative of the rock mass. Due to practical identification difficulties in obtaining the correct value for D, although guidelines for selection of D are available in [46]}	[28]
(58)	➢ Exhibited the best results for a weak marly rock mass, having a mean uniaxial compressive strength of 18.60 MPa. ➢ Not appropriate for foundation design on weathered rock and over-predicts E_m.{ field test results in Hong Kong for igneous, volcanic and metamorphosed sedimentary rocks} ➢ Equation yields some unexpected E_m values, particularly for GSI>60. ➢ Limited for weak rock (σ_c<100MPa) mass and overestimates the rock mass at upper range of GSI. ➢ Give reasonable fits to the field data.	[31] [5] [34] [31] [38]
(59)	➢ Exhibited a high predictive capability ➢ Over-predicts E_m ➢ Provides the best predictions (especially for Dilatometer test data) ➢ For weak rocks (σ_c<100MPa), produced the better predictions, but Equation (50) was best performed.	[31] [5] [31] [31]
(60)	➢ Although the equation yields good prediction for authors' database, it nevertheless needs to be further evaluated with other databases for foundation design	[5]
(61)	➢ Yields results significantly lower than those obtained from other laboratory and field tests	[8]
(63)	➢ Can be used for weak fractured rocks but dry or nearly dry rocks	[43]
(64)	➢ Has a shortcoming in the estimation of a values for other RMR values as Verman (1997) [43] has only proposed it for RMR=31 and RMR=68	[65]