INTERMEDIATE GEOMATERIALS: PROPERTIES AND FOUNDATION CAPACITY USING THE PRESSUREMETER

GÉOMATERIAUX INTERMÉDIAIRES : PROPRIÉTÉS ET CAPACITÉ DE FONDATION AU MOYEN DU PRESSIOMÈTRE

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ABSTRACT

In the first part of this article, the modulus and unconfined compression strength of Intermediate Geo-Materials (IGM) is discussed. A distinction is made between the modulus of the intact IGM E_I and the modulus of the IGM mass E_M as well as the unconfined compression strength of the intact IGM q_{uI} and the unconfined compression strength of the IGM mass q_{uM}. In the second part, the pressuremeter (PMT) test parameters are discussed to show that the PMT does measure the IGM mass values. In the third part of this article, the axial capacity of bored piles in IGM is discussed, including the ultimate side friction f_u and the ultimate bearing pressure p_u. A database of instrumented load tests is assembled from publicly available documents, and these two parameters, f_u and p_u, are correlated with the pressuremeter limit pressure p_L. Then, the French guidelines to estimate f_L and p_u in IGM using p_L are evaluated using the database.

RESUME

Dans la première partie de cet article, on discute du module et de la résistance à la compression simple des géomatériaux intermédiaires (IGM). Une distinction est faite entre le module de l'IGM intact E_I et le module du massif IGM E_M ainsi que la résistance à la compression simple de l'IGM intact q_{uI} et la résistance à la compression simple de la massif IGM q_{uM}. Dans la deuxième partie, on discute des paramètres d'essai du pressiomètre (PMT) pour montrer que le PMT mesure les valeurs du massif IGM. Dans la troisième partie, on discute de la capacité axiale des pieux forés en IGM , y compris le frottement latéral ultime f_u et la pression portante ultime p_u. Une base de données d'essais de chargement sur pieux instrumentés est assemblée à partir de documents accessibles au public, et ces deux paramètres, f_u et p_u, sont corrélés avec la pression limite pressiométrique p_L. Ensuite, les règles francaises pour estimer f_u et p_u dans les IGM en utilisant p_L sont évaluées à l'aide de la base de données.

Keywords: Intermediate Geo-Materials (IGM); Pressuremeter (PMT); Deep Foundation; Shallow Foundation; SPT N.

1. Introduction

Intermediate GeoMaterials or IGM are intermediate between strong soils and weak rocks. IGM straddle the fields of soil mechanics and rock mechanics. This creates a problem as both fields do not use the same approaches to model the behavior of the material. For example, the consolidation theory exists in soil mechanics, but elasticity dominates in rock mechanics. Also soil strength is modeled differently in soils and in rocks. This paper addresses the difference between intact and mass parameters of IGM and follows by studying and making recommendations on the ultimate friction and ultimate point pressure of drilled shafts versus pressuremeter limit pressure.

2. IGM definition

IGMs vary widely, spanning from highly compacted sand and gravel to firm tills, to less dense sandstones, weathered limestone, and weathered granite. Typically, they exhibit Standard Penetration Test (SPT) N values between 50 and 200 blows per 0.3 m, uniaxial compressive strengths between 500 to 5000 kPa, and pressuremeter limit pressures between 2.5 and 50 MPa. These values were obtained from O'Neill (1996) and associated correlations. A distinction is made between the intact IGM parameter (modulus E_I or unconfined compression strength q_{uI}) and the IGM mass parameter (modulus EM or unconfined compression strength q_{uM}). According to O'Neill (1996), IGM can be classified in three categories:

1. *IGM Category 1* are argillaceous geomaterials, including heavily over-consolidated hard clays, clay shales, claystone, siltstone, saprolites, mudstones. The unconfined compressive strength (q_{uI}) of these materials typically falls between 500 and 5000 kPa and demonstrates a tendency for significant strength reduction upon exposure to water and to smearing when drilled.
2. *IGM Category II* are calcareous geomaterials including limestone, limerock and argillaceous

geomaterials not prone to smearing when drilled. Category two is distinguished from category one by experiencing a compression strength loss of less than 40% of the original q_{uI} after being exposed to water for three days at 350 kPa confining pressure.

3. *IGM Category III* are highly compact granular geomaterials, including residual rock fragments, fully decomposed rock, and glacial till. Standard Penetration Test (SPT) values typically range between 50 and 100 blows per 0.3 meters.

Some elementary rock mass indices can be useful in describing IGM. They are the Recovery Ratio (RR %) defined as the length of core recovered divided by the length cored expressed as a percentage, and the Rock Quality Designation (RQD %) defined as the cumulative length of core segments longer than 0.1 m divided by the length cored expressed as a percentage; therefore, RQD is always less or equal to RR. The Rock Mass Rating (RMR, Bieniawski 1989) is a more advanced rock mass index. It is a value between 0 and 100 equal to the sum of ratings for several indicators of rock mass features and properties. These indicators include unconfined compression strength of the rock substance (q_{uI}), RQD, joint spacing, joint condition, joint orientation, and ground water condition; therefore, RMR is always larger than RQD.

The tests involved to characterize the properties of IGMs are laboratory tests and in situ tests among which is the pressuremeter. The smaller scale laboratory tests such as the unconfined compression test define the intact IGM parameters while the pressuremeter, the flat jack test, (Deklotz and Boisen 1970), the plate test (Bieniawski 1978) and the Radial Jacking Test (RJT) also known as the Goodman Jack Test (Goodman et al. 1968) define the IGM mass parameters. This is based on the volume of material tested with the laboratory test involving a volume of about 0.0007 m^3 and the in-situ test about 1 m^3 or 1500 times more IGM mass volume.

3. Modulus and strength of IGM: intact and mass values

Modulus. The modulus of the rock/IGM substance or intact rock/IGM, E_I, ranges from 2000 MPa to 100000 MPa with concrete at about 30000 MPa. The ratio between the modulus E_I and the unconfined compression strength q_{uI} of the intact rock/IGM ranges from 150 to 600 with the lower values for the softer rock/IGMs, like shales and sandstones.

The tensile strength of intact rock/IGMs ranges from 1 MPa to 15 MPa with the lower values for softer rock/IGMs like shale. The shear strength of intact rock/IGM leads to cohesion intercepts in the range of 5 to 40 MPa and friction angles from 30 to 50 degrees. So, the major difference in strength parameters between IGM and soils is the cohesion, not the friction angle.

Correlations between the ratio of the rock/IGM mass modulus E_M over the intact rock/IGM modulus E_I and rock mass indices have been attempted (Figs. 1 and 2). Fig. 1 shows E_M/E_I as a function of RQD and Fig. 2 shows E_M/E_I as a function of RMR. As can be seen in both figures, the trend is clear, but so is the scatter. The regression equations in those figures are:

$\frac{E_M}{E_I} = 0.0156 \times 10^{0.0169RQD\%}$ based on 174 data records (Fig. 1) (1)

$\frac{E_M}{E_I} = 0.0237 \times 10^{0.0191RMR}$ based on 98 data records (Fig. 2) (2)

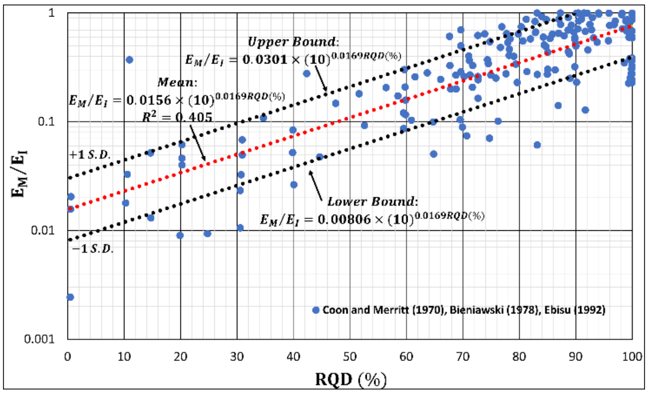

Figure 1. Ratio of rock mass modulus to intact rock modulus vs. RQD based on 174 data records (after Zhang and Einstein 2004).

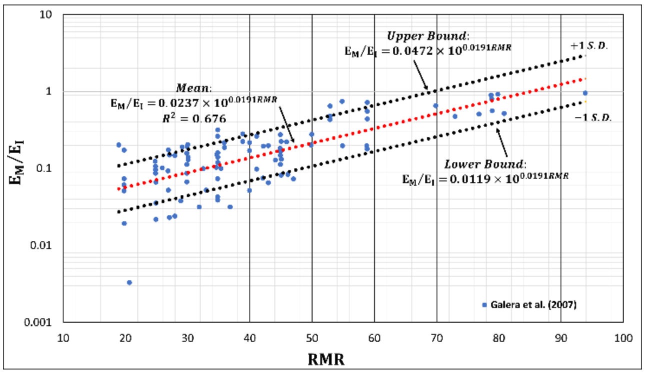

Figure 2. Ratio of rock mass modulus to intact rock modulus vs. RMR based on 98 data records (after Galera et al. 2007).

Strength. Zhang (2016) correlated the ratio q_{uM}/q_{uI} of the unconfined compressive strength of the rock/IGM mass over the unconfined compressive strength of the intact rock/IGM with the ratio E_M/E_I of the modulus of the rock/IGM mass over the modulus of the intact rock/IGM. The value of q_{uM} was obtained from triaxial tests on large, jointed rock mass specimens. He proposed:

$$\frac{q_{uM}}{q_{uI}} = \left(\frac{E_M}{E_I}\right)^{\alpha} \quad (3)$$

Where α ranges from 0.5 to 1 and averages 0.7. Other correlations have been attempted between the ratio q_{uM}/q_{uI} and RQD such as:

$\frac{q_{uM}}{q_{uI}} = \frac{10^{RQD/86}}{18}$ based on Eq.1 and 3 combined (4)

and $\frac{q_{uM}}{q_{uI}} = e^{(RMR-100)/15}$ based on Ván and Vásárhelyi (2009) (5)

Fig. 3 shows three recommendations correlating the ratio q_{uM}/q_{uI} with RQD.

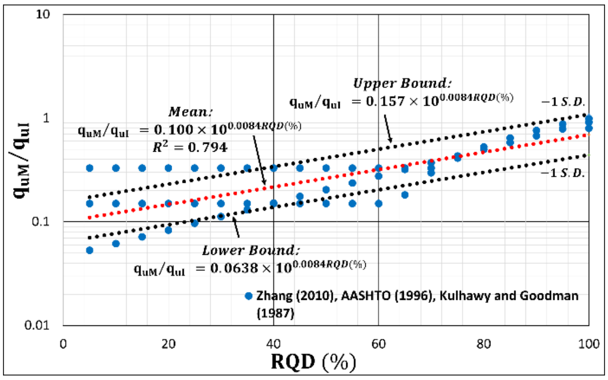

Figure 3. Ratio of rock mass modulus to intact rock modulus vs. RQD from recommendations by Zhang (2010), AASHTO (1996), Kulhawy and Goodman (1987).

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Note that the strength equation describing the failure envelope of rock masses and of soil masses are different. IGM being intermediate materials leads to presenting the two equations. The strength of rock masses is influenced by the confinement stress σ3 at depth z. This leads to Hoek (1994) strength equation for rocks which also recognizes the curvature of the strength envelope:

$$\sigma_1 = \sigma_3 + q_{uI}(m_b \frac{\sigma_3}{\sigma_{CI}} + s)^{\alpha} \quad (6)$$

Where σ1 and σ3 are the major and minor principal stresses on the rock mass element, q_{uI} is the intact rock unconfined compression strength, and mb, s and a define the curvature of the strength envelope. By comparison the Mohr Coulomb strength equation for soils is

$$\sigma_1 = \sigma_3 \left(\frac{1+sin\varphi\prime}{1-sin\varphi\prime}\right) + \frac{2c\prime cos\varphi\prime}{1-sin\varphi\prime} \quad (7)$$

4. Pressuremeter values in IGM

The PMT (Briaud, 2023), because of the large volume of material tested, is more representative of the mass modulus and mass strength values. The range of PMT modulus and limit pressure for IGM is proposed and estimated as presented in Table 1.

Table 1. Range of PMT modulus and limit pressure in IGM

PMT Modulus (MPa)	PMT Limit Pressure (MPa)	IGM Type
35 to 140	2.5 to 10	Low Strength IGM
140 to 420	10 to 30	Moderate Strength IGM
420 to 770	30 to 55	High Strength IGM

The fact that the PMT stresses a much larger volume of IGM than typical laboratory tests can be demonstrated by comparing the PMT modulus to the modulus obtained from unconfined compression tests on intact IGM. Fig. 4 shows the ratio of the PMT modulus (E_{PMT}) over the modulus obtained from unconfined compression tests on intact IGM cores ($E_{I(UC)}$) versus the RQD (Isik et al. (2008)). Fig. 4 also shows the relationship between the rock mass modulus EM and the intact rock modulus (EI(UC)) versus RQD (%) developed by Zhang and Einstein (2004).

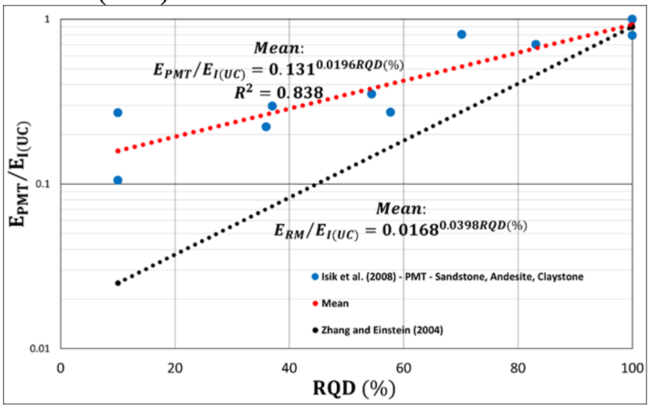

Figure 4. Correlation between $E_{PMT}/E_{I(UC)}$ vs. RQD (%). (After Isik et al. 2008).

A comparison between the modulus obtained from plate load tests (E_{PLT}) and from the PMT (E_{PMT}) in soils and IGM is shown on Fig. 5 (Cheshomi and Khalili, 2024). The regression equation is close to a one to one relationship and the R2 is reasonable at 0.812. The plate was a rigid circular steel plate with a diameter of 0.30 m.

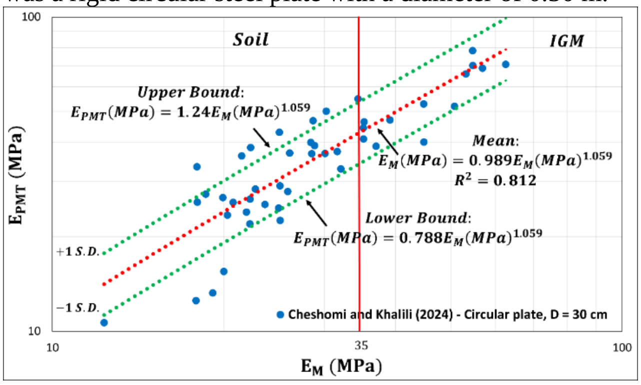

Figure 5. Correlation between $E_{PMT} - E_M$ (After Cheshomi and Khalili 2024).

The Standard Penetration test is a very common test in North and South America, however the correlation between the pressuremeter modulus E_{PMT} and the Standard Penetration Test blow count NSPT (Fig. 6) shows significant scatter. Fig. 6 is based on 172 data points gathered from various open literature sources. Data were selected from a variety of geomaterials including clay, sand, clayey sand, silty sand, silty clay, shale, limestone, alluvium, weathered rock, weathered schist, and dense weathered granite. The data for the $E_{PMT} - SPT\ N$ plot in Fig. 6 and for the $E_{PMT} - p_L$ plot in Fig. 7 were collected using the following references: (Baguelin (1978), Birid (2015), Colins (2016), Drumright and Barnard (2018), Tarawneh et al. (2018), Samtani (2020), Zaki et al. (2020). While Fig. 6 shows a definite trend, the correlation exhibits a relatively low R^2 of 0.186. As such, obtaining an IGM modulus from an SPT test is not a preferred choice.

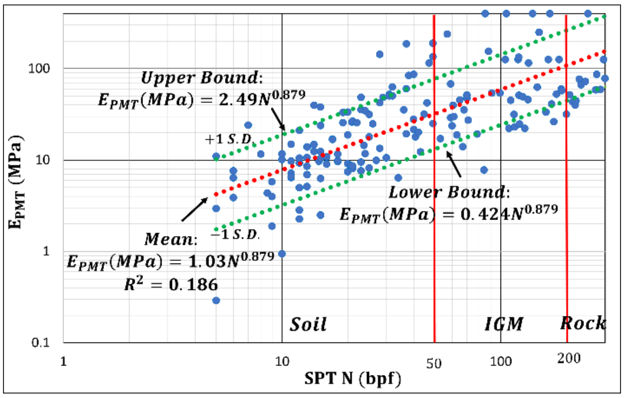

Figure 6. Correlation between $E_{PMT} - SPT\ N$ (After Cheshomi and Khalili, 2024).

Fig. 7 presents a correlation between the PMT modulus E_{PMT} and the PMT limit pressure p_L for IGM, soils and rocks. A total of 360 sets of data were collected. As can be seen, the correlation is more scattered for IGM than for soils.

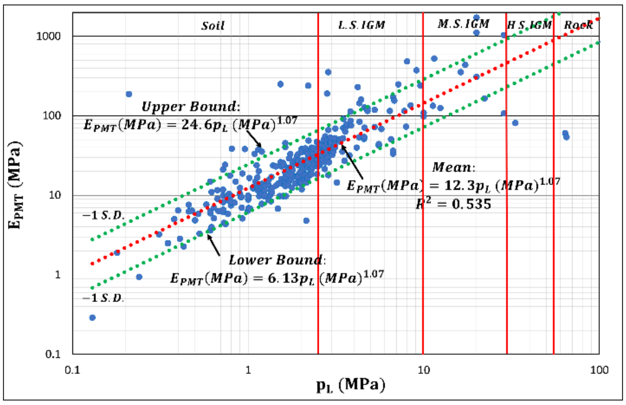

Figure 7. Correlation between $E_{PMT} - p_L$.

5. Shallow foundation design

The ultimate bearing capacity criterion is usually not the controlling factor for footings on IGM; nevertheless, it seems reasonable to use q_{uM} as the safe foundation pressure psafe in the simpler case of a shallow footing on flat ground. This comes from the ultimate bearing pressure of clays pu by Skempton equation (Briaud 2023) compared to the undrained shear strength of that clay.

$$p_{safe} = \frac{p_u}{3} \approx q_{uM} \approx \frac{p_L}{3} \tag{8}$$

Where p_L is the pressuremeter limit pressure. The elastic equation for the settlement s is:

$$s = I(1 - \upsilon^2)\frac{pB}{E_M} \tag{9}$$

Where I is a shape factor (Briaud, 2023), ν is Poisson's ratio, p is the average pressure under the shallow foundation, B is the width of the shallow foundation, and E_M is the rock mass modulus. Consolidation is rarely the controlling settlement phenomenon however creep settlement over 75 years of sustained loading may double the value from Eq. 9. The allowable bearing pressure for shallow foundations on rock/IGM varies from 1 to 10 MPa.

6. Deep Foundation Design

The most common deep foundations in IGM and rock are drilled shafts. The ultimate friction stress at the interface between the IGM and the concrete is called f_u while the ultimate point pressure at the bottom of the drilled shaft is p_u. A search of publications in the public domain allowed us to assemble a database of 38 instrumented load tests in IGM; it yielded 231 values of fu and 45 values of p_u. Generally, the drilled shafts were pushed to sufficient movement to develop the ultimate friction f_u. Sometimes, the downward movement of the drilled shaft was insufficient to reach the ultimate point pressure p_u. In this case, and if the point pressure versus point movement curve showed enough curvature, a hyperbolic extrapolation was performed to obtain the asymptotic p_u value (open circles in Figs. 9 and 10). If the point pressure versus point movement curve showed very little curvature, no extrapolation was performed and the maximum pressure reached was kept (solid dots with an upward arrow in Figs. 9 and 10). For each f_u or p_u value, corresponding IGM strength parameters were sought including q_{uI}, the unconfined compression strength of the intact IGM.

The authors are of the opinion that the pressuremeter (PMT) is one of the best tools to obtain useful parameters for the design of deep foundations in IGM. For that reason, an effort was made to generate plots of f_u vs p_L and p_u vs p_L where p_L is the PMT limit pressure. Because no PMT data was associated with any of the load tests collected, estimates of the limit pressure were obtained from the values of q_{uI} using the correlation developed by Briaud (1992).

$$q_{uI}(kPa) = 1.34(p_L(kPa))^{0.75}$$

$$\text{or } p_L(kPa) = 0.677(q_{uI}(kPa))^{1.33} \tag{10}$$

Then the plots of f_u vs. p_L and p_u vs. p_L could be generated as shown in Figs. 8 and 9. The mean regression equations are:

$$f_u(MPa) = 0.0818(p_L(MPa))^{0.538} \tag{11}$$

$$p_u(MPa) = 2.14(p_L(MPa))^{0.399} \tag{12}$$

In each graph, the best fit regression is indicated and bounds at +/- one standard deviation are identified. It seems reasonable to suggest that the equation for the line corresponding to minus one standard deviation could be used in design.

<u>Ultimate friction stress (IGM)</u>

$$f_u(MPa) = 0.0362p_L(MPa)^{0.538} \tag{13}$$

<u>Ultimate point pressure (IGM)</u>

$$p_u(MPa) = 1.140p_L(MPa)^{0.401} \tag{14}$$

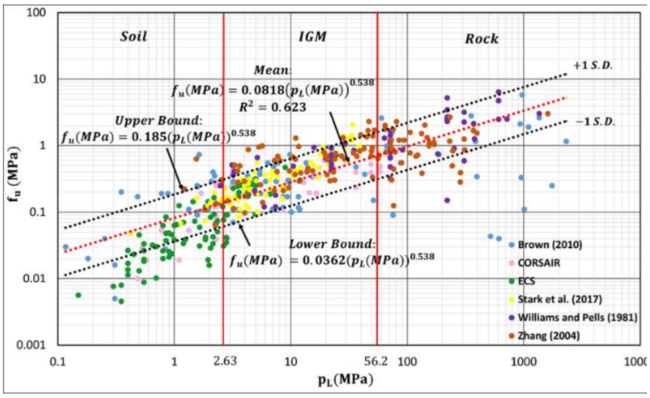

Figure 8. Ultimate friction stress versus pressuremeter limit pressure.

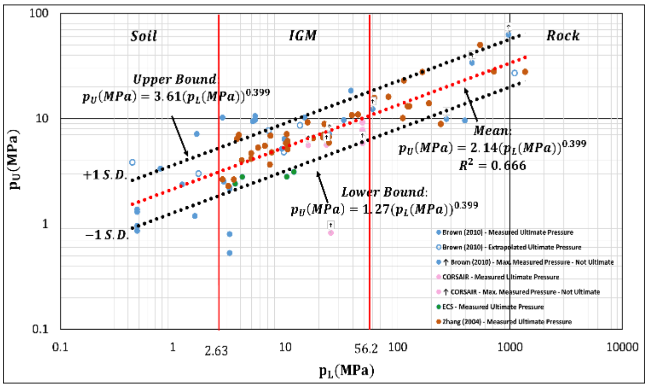

Figure 9. Ultimate point pressure versus pressuremeter limit pressure.

The zone in Figs 8 and 9 associated with the range of pL values for IGM (2.63 to 56.2 MPa) and bounded by the +/- one standard deviation regression lines was further studied to try to identify factors impacting the range of ultimate side friction and ultimate point pressure within that zone. Using RQD and RMR turned out to be unsuccessful. The only factor which could be reasonably correlated with, was the extent of weathering of the IGM. Highly weathered IGM would fit in the lower third of that zone with moderately weathered IGM in the middle and slightly weathered IGM in the top third.

The French guidance document (AFNOR, 2013) provides steps to estimate the ultimate side shear stress fu. The steps include categorizing the soil and IGM material into five groups, then selecting the pile type, then choosing the appropriate design curve, then reading fu corresponding to the PMT limit pressure pL on that curve. The three curves are Q3 for chalk, Q4 for marl and marly limestone, and Q5 for weathered rock. This document also gives the ultimate point pressure pu as :

$$p_u = k_p\, p_L + \gamma d \tag{15}$$

Where k_p is the PMT bearing capacity factor, γ is the soil total unit weight, and d is the depth of embedment of the pile. The value of k_p in Eq. 15 for most IGM in the guidelines is 1.45. The curves corresponding to the equations in those recommendations are presented in Fig. 10 and 11 along with the data collected.

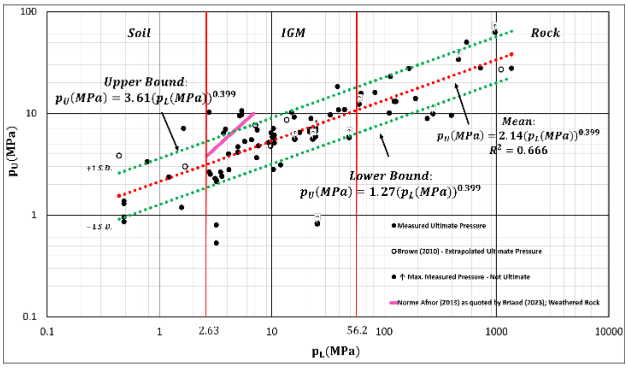

Figure 10. Evaluation of existing guidelines to predict the ultimate point pressure of drilled shafts based on the limit pressure pressuremeter p_L.

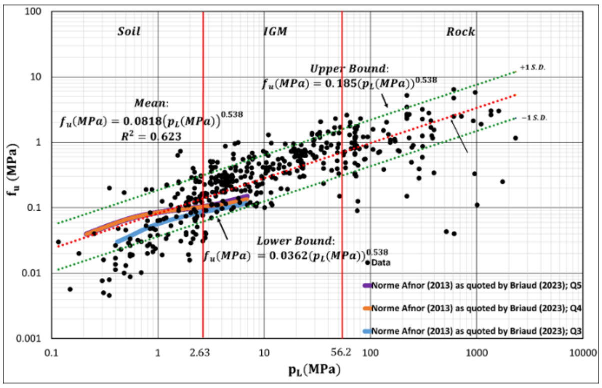

Figure 11. Evaluation of existing guidelines to predict the ultimate side shear stress of drilled shafts based on the pressuremeter limit pressure p_L .

7. Conclusion

The pressuremeter is one of the best and economical tools to obtain IGM properties suitable to predict foundation behavior. Among the reasons are that the volume of IGM mobilized during the PMT test is large compared to laboratory tests, that the PMT test is not unlike a load test, and that the PMT gives IGM properties over a large range of strains and stresses. Since the IGM properties that matter in foundation behavior prediction are those of the IGM mass and not those of the IGM intact material, the PMT is recommended.

The modulus of the IGM mass E_M tends to be smaller too much smaller than the modulus of the intact IGM E_I. The ratio E_M/E_I is documented by using data from rocks. It is found that E_M/E_I increases from 0.01 to 1 when RQD increases from 0 to 100% and from 0.05 to 1 when RMR increases from 20 to 100%.

The ratio of the PMT modulus over the unconfined compression test modulus is also shown to decrease with RQD yet compares very closely with the plate test modulus. The correlation between the PMT modulus and the SPT blow count is scattered and it is not recommended to use N to obtain a modulus except as a very crude preliminary estimate. A comparison between PMT modulus and limit pressure is presented and a PMT parameter-based IGM classification is suggested.

Drilled shafts are typically the deep foundation of choice in IGM. A database of instrumented load tests on drilled shafts where the side load and point load were measured separately was assembled. As a result, 38 instrumented load tests in IGM were organized and analyzed; it yielded 231 values of the ultimate side friction fu and 45 values of the ultimate point pressure p_u. For each f_u or p_u value, corresponding IGM strength parameters were sought including q_{uI}, the unconfined compression strength of the intact IGM, and N, the SPT blow count. Correlations between q_{uI} and p_L were used to create a plot of f_u and p_u versus p_L. Regression equations are presented. The data is compared to current recommendations for strong soils in AFNOR (2013). The comparison shows that the guidelines tend to overestimate p_u and underestimate f_u.

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