Hard soil and rock classification – Pressuremeter data versus tests on samples

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ABSTRACT: This paper deals with the correlation between rock properties presented by Kanji (2014), Kanji & Leão (2020) on rock samples of a wide range of strengths, with the classification of indurated and rocky soils from pressuremeter diagrams by Baud & Gambin (2013), Baud (2021). Tests were carried out simultaneously on a same jobsite, and these two approaches contribute to the characterization of rock drillability, showing a certain degree of similarity between them. The relationship between the pressuremeter modulus of the rock mass (ISO 22476-5) and a Young's modulus usable for finite element software is a stumbling block for the validation of a rock model, which remains questionable, as the actual behaviour of the rock masses is highly non-linear. Neither in-situ expansion testing nor uniaxial compression testing directly measures a single modulus representative of the rock mass, so engineering practice is to apply one or more weighting factors, primarily considering the spacing and nature of fractures within the rock mass.

Keywords: Rocks classification, High Pressure Pressuremeter Test (Dilatometer Test), Compression Strength, Ground Modulus, Drillability.

1 ROCK TESTS IN SITU AND ON SAMPLES

Several different test methodologies are available to characterize and classify a rock mass from a geomechanical point of view, and strength and modulus of intact sound rock constituent material.

1.1 Types of tests processed

The two types of tests that will be examined, and where practical, compared, will be:

- In situ tests of the pressuremeter or dilatometer type; consisting of producing the expansion of the cylindrical cavity of a borehole by a deformable probe subjected to high internal pressure, all these types of tests are now brought together in the EN-ISO 22476-5 standard; the term dilatometer, used for several decades, has seen its name gradually polluted by the "dilatometric" blade jacked into the soil which obviously finds itself in refusal as soon as the soil becomes resistant.

- Rock core tests consisting of uniaxial compression using axial loading of a cylindrical sample coupled with a measurement of the vertical displacement and the lateral deformation of the sample until its rupture (ASTM D 7012-14, IRSM 2009).

Performing both types of tests on the same site in the same rock mass provides different and complementary information, the two types of tests simultaneously providing a strain parameter, a modulus (E_M modulus or G modulus for the pressuremeter, Young's modulus known as E_{50} or Ed according to authors for the compression test) and a failure parameter (limit pressure p_{LM} reached or approached for the pressuremeter, unconfined compressive strength σ_c (French speaking common name Rc and English UCS for the core test).[1]

1.2 Comparing Pressuremeter Tests and Uniaxial Compressive Strength Tests

It is often considered that pressuremeter testing is not comparable to uniaxial strength testing, because they do not involve the same rock volume, and for the following reasons:

- The pressuremeter expansion test in a borehole measures the modulus of a surrounding volume of the rock mass, and its range of action integrates both the reaction of the rock matrix and any fractures within the zone of influence.
- The compression test measures the modulus of an element isolated from the rock mass not comprising any open fracture, even if traces of closed fractures may appear on its surface.

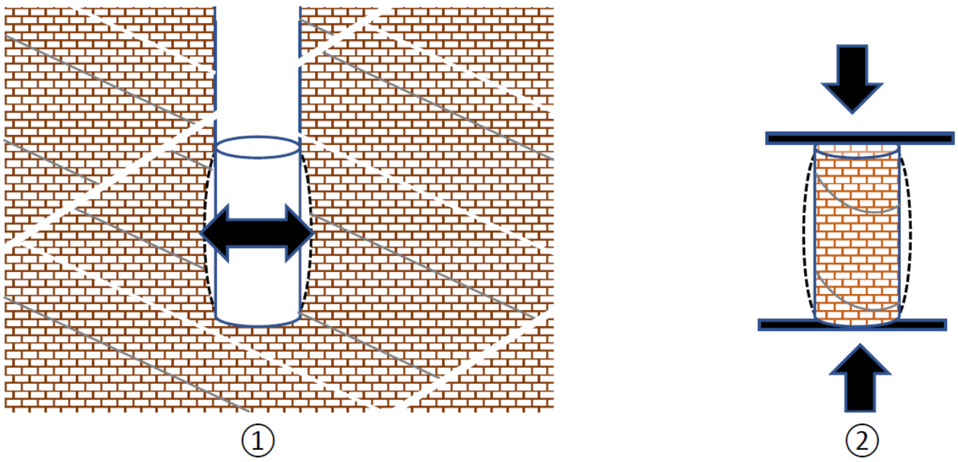


Figure 1. Difference in principle of the two tests: ① PMT ② UCT.

We can theoretically compare the two types of failure by means of the estimation of the shear strength Su:

- Theoretically the relation for uniaxial compression is $\sigma_c \leq 2.Su$, as recalled by Caquot & Kérisel (1956-1966), which is usually retained as $\sigma_c = 2.Su$.
- The relationship between Su and the limit pressure p^*_{LM} is more complex and has been the subject of many theoretical expressions, which we will not repeat here, in favour of the semi-empirical expression, verified by experience, $Su = \alpha.p^*_{LM}/(\pi+2)$, ($\alpha$ rheological coefficient of Ménard).

The coefficient α varies from 0.25 to 1 in soils, the lower limit tending to rise between 0.3 and 0.35 in HSSR (Hard Soils & Soft Rocks); we can write in this case $\alpha = 2/3 \pm 1/3$.

On average, therefore, there is a linear relationship: $\sigma_c = 0.26(\pm 0.13).p^*_{LM}$, assuming that HSSR exhibit plastic behaviour, and that the relationship varies with the confining stress.

As for the modulus, the theoretical comparison is more difficult to make, but we can investigate a correlation through the ratios E_{M}/p^*_{LM} and E/σ_c.

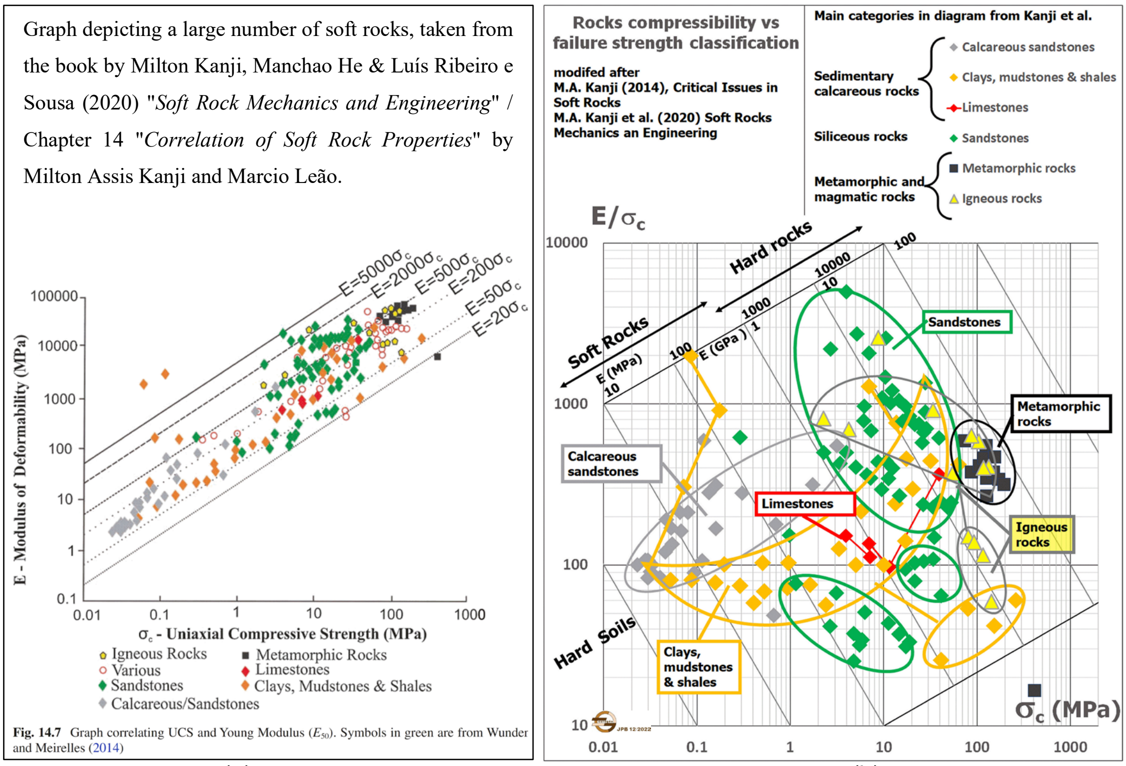
[1] in the rest of the text, the simple writing E, or explicitly E_{UCT}, will be used for E_{50} or E_d.

2 CLASSIFICATION BY THE RESULTS E & σ_C OF UNIAXIAL COMPRESSION TEST

2.1 Soft rocks data from M.A. KANJI et al.

M.A. Kanji (2013) has published a large amount of data on soft rocks to characterize the correlations between different parameters, recently included in Kanji & Leão (2020). Figure 14.7, shown here as Figure 2(a), indicates a linear correlation between σ_c and E_{50} with values of E/σ_c ranging from 20 to 5000. Therefore, rather than a correlation, it should be thought that this ratio shows a range of values which allows a classification according to the nature of the rocks and their state of alteration. We can use the same data, in a logarithmic plot: E/σ_c f (σ_c) as shown in Figure 2(b).

From Figure 2(b) some types of rocks are well grouped, and the combined value of the 2 axes characterizes them well into groups, such as metamorphic rocks or igneous rocks; others, such as sandstones, present a greater dispersion or groupings by distinct zones, groups which may correspond to different degrees of cementation. Some authors, such as Lama & Vutukuri (1978) have used the E/σ_c ratio as a criterion for classifying rocks, without however using this ratio as a graph axis.



(a) (b)

Figure 2. Presentation of data [σc | E] in a diagram [σc | E/σc].

2.2 Rock data from other sources

The works cited were devoted to soft rocks. Within the literature there is a large set of data of σ_c on harder rocks, which are not always accompanied by the measurement of E. These two parameters are found from data tables from various publications, including: Hoek's manual (1997-2023), US Manual on Subsurface Investigations (2010), Sanchez Rodriguez & al. (2011), Manchao He (2014), Gudmundsson (2015), Palmström & Singh (2001).

These data make it possible to complete the classification Figure 2 for hard rocks. The limits of the resulting diagram in Figure 3 are fixed to integrate the majority of the quoted data; by comparison and to set the upper limit (E maximum and σ_c maximum), some benchmarks are shown, given by very hard materials, natural: quartz, diamond, or manufactured: concrete, steel, bronze, glass, industrial ceramics (https references).

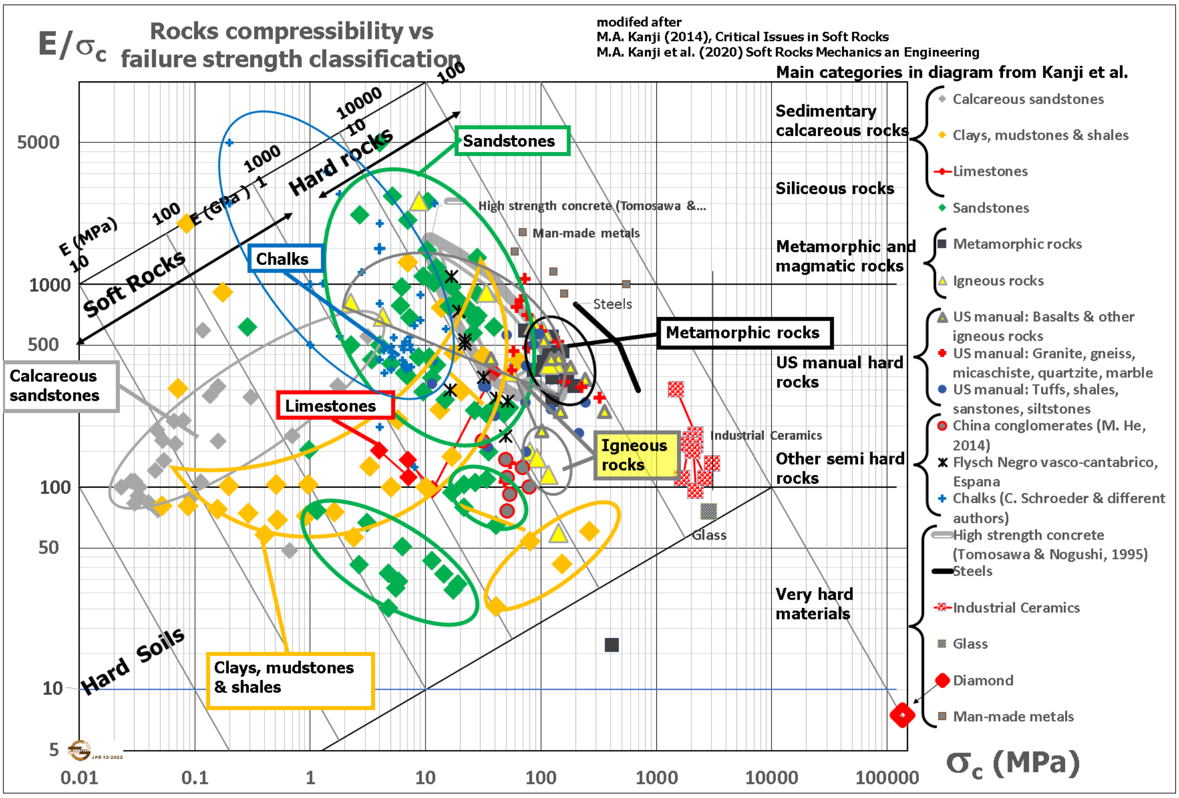


Figure 3. Set of different rocks and materials in the diagram [σc | E/σc].

2.3 *Proposed rock classification from UCT data*

From the range of information compiled in the above references, we can see a way of classifying rocks from their deformation and failure characteristics measured in a uniaxial compression test. The maximum strength σ_c in the US Manual data is from a test on basalt of the John Day Formation in Oregon (USA), with σ_c=350MPa, originally cited by Goodman (1989); it is a relatively recent igneous rock, from an eruption during the Cenozoic (Oligocene) era. What is rather expected, statistically, is that the maxima in σ_c or E come from very old multimetamorphic rocks (Kanji, 2014).

In fact, one of the metamorphic rocks from Kanji & Leão (2020) is given for σ_c=433MPa, but with a fairly low modulus E=6900MPa, so it appears very isolated from the other rocks near the bottom of the diagram (fig. 3). As maximum values of E and σ_c, we found data on a taconite from Minnesota, an iron ore quartzite from the Precambrian basement of North America, with σ_c =400MPa with a higher modulus E=100GPa. It is the most "Guinnessable" natural rock we have. In our own data, examined in Section 3, the maxima are also quartzites, one in the Precambrian series of French Guiana, a metaquartzite in the basement of South America (σ_c =235MPa / E=84GPa), the another in the "Armorican sandstone" of the Ordovician in the Cotentin peninsula (σ_c =406MPa / E=77.7GPa).

Chalks constitute a very particular type of formation and may occupy an atypic position in the diagram (E/σ_c always rather high), according to their stratigraphy, original lithology and degree of weathering (Verbrugge & Schroeder, 2018).

An interpretation of these results is presented in figure 4, and indicates materials formed at high temperature, and metallic or non-metallic single crystals (Zn, Cu, Fe, W, Si and C). They trace an absolute boundary for matter E<1000GPa and E/σ_c <5000, while crystalline rocks remain bounded by E<100GPa and σ_c <500MPa.

Within the limits of this classification, one can propose that the trend ①: "moduli and limit uniaxial strength increasing simultaneously" marks a parallel gradation of the degree of cementation and consequently of the decreasing porosity; and for the trend ②: "increasing E/σ_c ratio with decreasing σ_c strength", one hypothesis is that of a gradation between a clearly brittle behaviour of the test at failure and a behaviour tending to be more ductile.

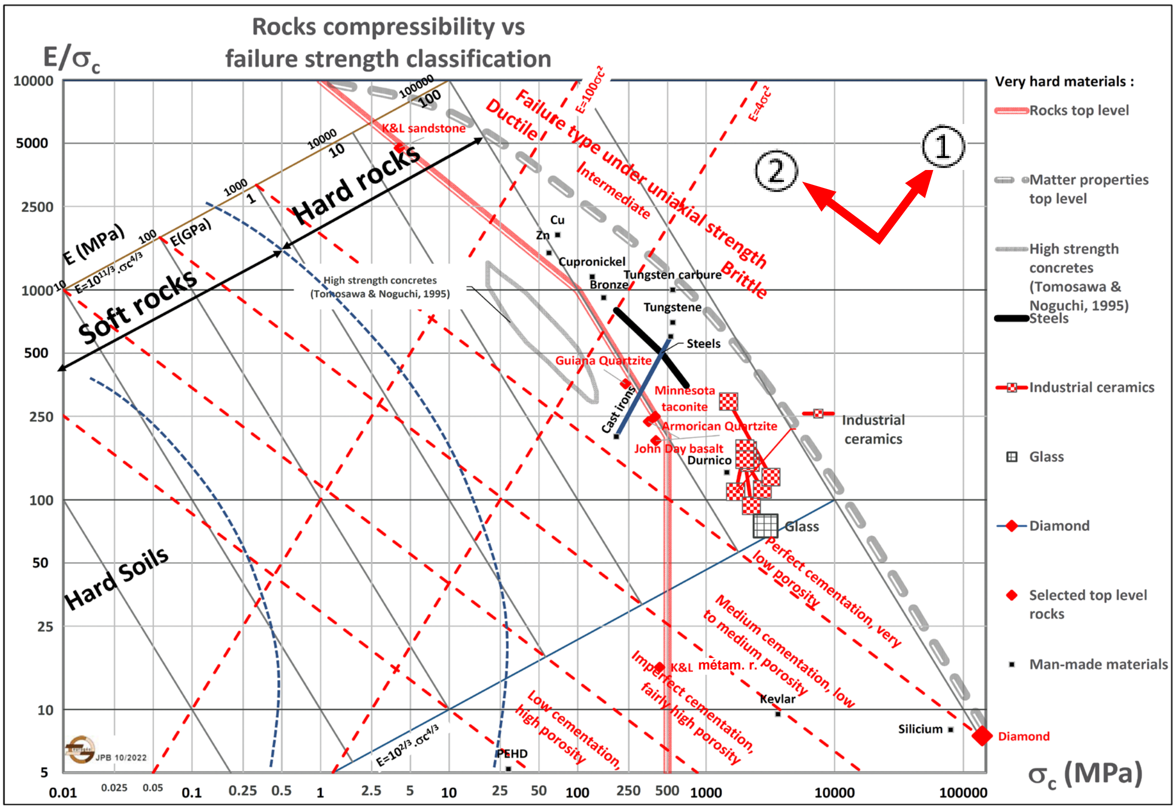


Figure 4. Proposed rock behaviour from UCT data.

3 PRESSUREMETER TESTS VERSUS CORE TESTS.

In some sites, it has been possible to carry out both types of tests on the same rock mass, as initiated by L. Ménard (1965). In situ tests were generally carried out up to 20 to 30 MPa using devices such as Mazier (Télémac), Hyperpac (Apagéo-Géomatech) or HPD (CamInSitu) dilatometers. We were then able to see how the relative behavior, on both the [σ_c | E/σ_c] diagram and the Pressiorama [p^*_{LM} | E_M/ p^*_{LM}] (Baud & al., 2014, 2015, 2021), is related to the type of rock, the structure and fracturing of the rock mass and the resulting drillability.

There is insufficient space to present here these comparisons, which focused on the following rocks, on which the pressuremeter tests reached between 10 and 30 MPa: Coarse limestone from the Paris Basin at Gouvieux (Oise) (Arscop, 2020) / Medio-Liassic sandstone from Lorraine at Vandœuvre-lès-Nancy (Meurthe-et-Moselle) / Miocene molasses from the Rhodanian furrow at Charmes-sur-Rhône (Drôme) / Urgonian alpine reef limestone at Etrembières (Haute-Savoie) / Fammenian sandstone from the Ardennes at Maubeuge (Nord) / Armorican Ordovician quartzitic sandstone at La Hague (Manche) / Precambrian metaquartzite on the Comté River from French Guiana / Precambrian rocks at Menai Bridge (Wales) / Jurassic limestones at Andelnans (Terr.-de-Belfort). Several of these data will be revisited in more detail in a later publication.

One of the conclusions is that the relations between E_M and E_{UCT}, and between p^*_{LM} and σc, rarely correspond to the theoretical expectation from Figure 1 & paragraph §1.2:

- For moduli, correlations are often under the form $E_{UCT}=K.E_M^a$, with $0.2<a<1$, giving $5<E_{UCT}/E_M<100$. (see Fig.5). This fairly wide range of ratios is obviously related to the degree of fracturing of the rock mass. Fracturing density produces a deformability measured by the in-situ test, much higher than that of the UCT on a rock sample free of any fracture, at least without any visible discontinuity. This therefore may constitute a kind of measure or index of this fracturing.

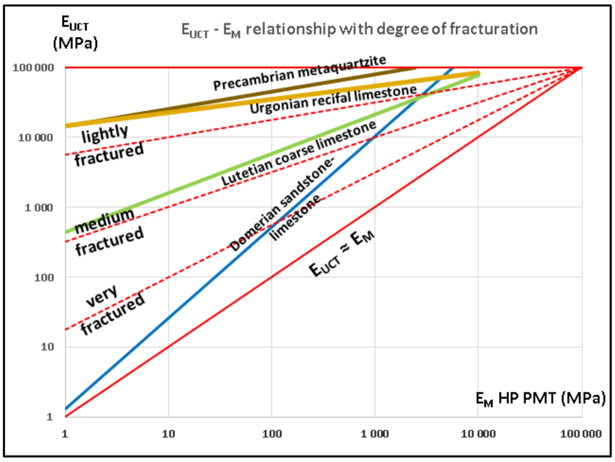


Figure 5. Correlations E_{UCT} vs E_M for 4 site cases.

- For failure parameter, typically $0.8<p^*_{LM}/\sigma c<3$. In this case, the ratio is rather limited and indicates the fact that failure in cylindrical expansion is influenced by the confining stresses, unlike failure of a sample in uniaxial compression without applied lateral stress.

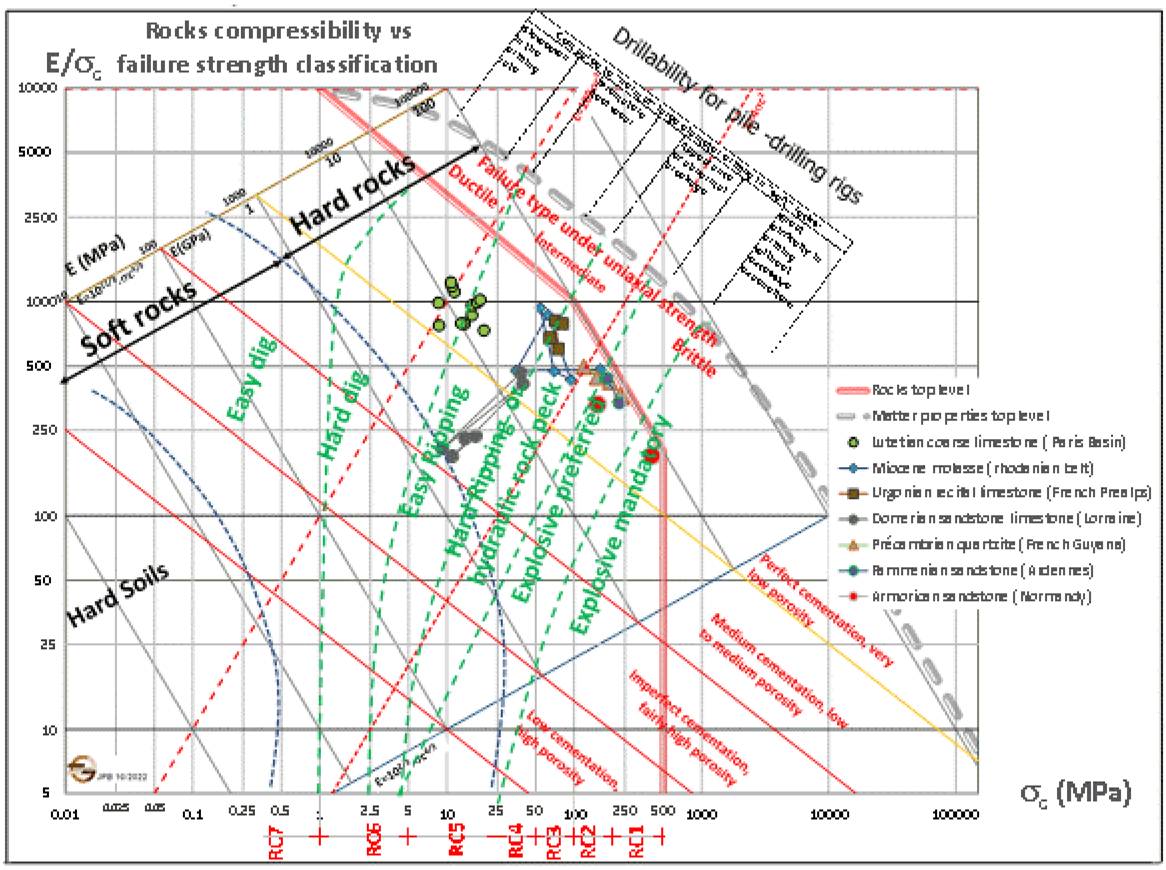


Figure 6. Drillability chart from [σc | E/σc].

Another conclusion from the σc | E/σc diagram is a prognosis of drilling capacity for pile drilling rigs, as shown in Figure 6, based on observed difficulties (slowdown in drilling, wear and breakage of drilling tools), where the drilling capacity in the vocabulary of earthworks is correlated with the findings of Waltham (1994, 2009).

ACKNOWLEDGEMENTS

The authors wish to acknowledge the companies that have made it possible to collect relevant data for the drillability of rocks, in particular from the point of view of pile drilling rigs: Botte Fondations (Vandœuvre, Etrembières, Charmes-sur-Rhône, Maubeuge, Andelnans), Franki Fondations Fayat (The Hague), Bouygues TP France (French Guiana). Respectful thanks to Prof. emeritus Christian Schroeder for his comments and advices during a proofreading of this text.

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