Ménard Pressuremeter Test and Dynamic Compaction

Le pressiomètre Ménard et le compactage dynamique

Stéphane Brûlé[1#]

[1]*Menard, Orsay, France*
[#]*Corresponding author: stephane.brule@menard-mail.com*

ABSTRACT

The intellectual legacy of Louis Ménard continues to be honoured, notably through the French National Project ARSCOP. With the renewed development of dynamic compaction in several regions around the world, the importance of geotechnical characterization using the Ménard pressuremeter in situ test, both before and after ground improvement, remains one of the method's key strengths. In France, dynamic compaction has also seen a resurgence, particularly for applications on brownfield sites, due to its ability to drastically reduce the carbon footprint on construction sites when compared to concrete-based solutions. Within the ARSCOP framework, which is based on soil strain ranges, it is particularly relevant to examine how this approach contributes to soil densification objectives, aligning with the broader goal of achieving high performance with reduced energy input.

RESUMÉ

L'héritage intellectuel de Louis Menard continue à être exalté, notamment par le Projet National français ARSCOP. Avec le redéveloppement du compactage dynamique dans plusieurs régions du Monde, l'importance de la caractérisation géotechnique par le pressiomètre Ménard avant et après travaux reste l'un des points forts du procédé. En France, le compactage dynamique a aussi connu un regain d'intérêt pour son application sur les friches industrielles et sa capacité à diminuer drastiquement l'empreinte carbone sur chantier, comparativement à des solutions faisant appel au béton.
Dans la démarche ARSCOP basé sur les gammes de déformations des sols, il est intéressant d'examiner la portée que celà peut avoir sur un objectif de densification des sols, toujours dans la démarche d'obtenir la performance avec moins d'énergie.

Keywords: ISP8; paper template; formatting; September 2025.

1. Introduction

The Ménard pressuremeter, patented in 1955 (Ménard, 1955) by Louis François Auguste Ménard, has made a major contribution to the control of soil improvement works using the dynamic compaction technique.

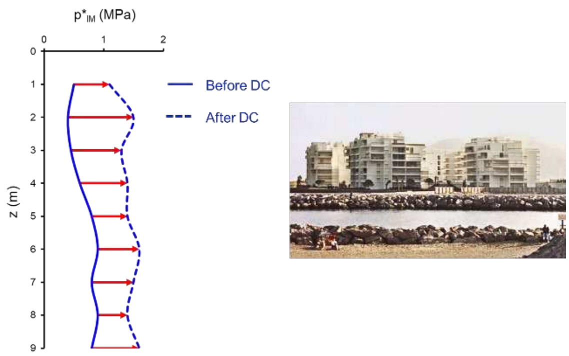

Figure 1. In 1969, the first application of dynamic compaction was carried out for a sea-side resort on reclaimed land at Mandelieu-La-Napoule (France). The graph on the left shows the improvement of the pressuremeter limit pressure p^*_{lM} before and after dynamic compaction (DC). From Ménard (Ménard, 1975).

As early as 1969 in France, at Mandelieu-La-Napoule, 8 to 10 m of reclaimed land was densified for a sea-side resort (Ménard, 1975). Improvement of the initial pressuremeter limit pressure p^*_{lM} ranged from 50 to 120% (Fig. 1). When Louis Ménard developed the modern version of dynamic compaction, there was still a drive to increase equipment capabilities, with the aim of potentially improving up to 40 meters of soil.

In the current use of dynamic compaction, particularly for construction sites located in urban areas, it is essential to control the amplitude of the waves generated by the impacts, as well as the magnitude of settlement of the working platform during compaction. In fact, excessive settlement implies the need to import costly materials to restore the project level. At the same time, the goal is to achieve significant performance without delivering unnecessary «over-compaction». The current trend is to better characterize the soil in order to improve it, if necessary, only to the extent required. This is accompanied by a shift toward using smaller equipment in certain applications. The overall approach is also part of an effort to reduce the carbon footprint of ground improvement projects.

It is worth noting that for companies such as MENARD, soil characterization prior to ground improvement works is still a key part of the concept. Moreover, the ongoing international development of MENARD company, particularly in the United States and Canada, reinforces the enduring relevance of foundational tools as the PMT, in geotechnical practice.

The aim of this article is to revisit the approach to soil characterization using the pressuremeter test (PMT), to present a case study of high-energy compaction on a former industrial site, and to discuss the contribution of the ARSCOP project in redefining soil improvement requirements.

2. ARSCOP Project

2.1. Pressurmeter test: tools and methods in constant evolution

Since the creation of the pressuremeter (Ménard, 1957; Baguelin et al., 1973; Gambin et Frank, 1982; Gambin, 1990. Briaud, 2013), both the equipment and the testing protocol have evolved (Arsonnet et al., 2005; Burlon et al., 2016), and the drilling and testing methodology has been standardized since 1990 through successive versions of the NF P94-110 standard, ultimately leading to the European standard NF EN ISO 22476-4 (AFNOR, 2015, 2021).

The French national research project ARSCOP (Les Nouvelles Approches de Reconnaissance des Sols et de Conception des Ouvrages géotechniques avec le Pressiomètre - New Approaches to Soil Investigation and Geotechnical Design Using the Pressuremeter), conducted from 2016 to 2024 (ARSCOP, 2024), aimed primarily at preserving the distinctive features of the pressuremeter test—namely, the pressure–volume curve obtained from a small-scale loading test, the stress state, the deformation modulus (E_M), and the strength parameter (p^*_{lM}) while continuing to refine and enhance the testing protocol.

The project was administered by IREX (Institut pour la Recherche appliquée et l'EXpérimentation en génie civil - Institute for Applied Research and Experimentation in Civil Engineering). One of the main results of ARSCOP is to specify the value of the soil deformation modulus $E = kE_M$ as a function of soil type and the range of strain ε under the applied load at the Earth's surface (Fig. 2).

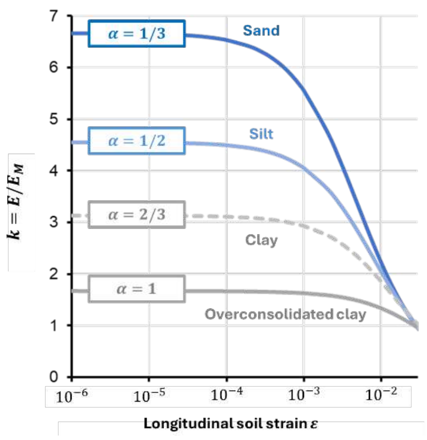

Figure 2. «k-curves», inspired from ARSCOP (ARSCOP, 2023). α is the Menard rheological factor, E is the modulus for the strain range of interest and E_M is the Menard modulus.

In the context of settlement analysis under large-scale loading, Combarieu highlighted, because the method was initially defined for shallow foundations, that « *a formulation of the type $E = \kappa E_M/\alpha$ offers a more suitable framework, with κ being a function of soil type, structural dimensions, and strain level—particularly at very low strains. This approach diminishes the relevance of the Ménard rheological factor α* » (Combarieu, 2006).

The other topics addressed are advanced drilling and self-drilling techniques, specialized equipment and testing protocols to accurately capture soil behaviour in the small-strain range.

In addition, cyclic pressuremeter systems have been engineered to facilitate the investigation of soil liquefaction phenomena. Seismic pressuremeter devices have also been optimized to improve signal quality and enhance measurement efficiency (ARSCOP, 2024).

2.2. A look at the mechanical parameters

The pressuremeter test (PMT) remains a tool that provides, almost instantaneously, soil deformation modulus E and failure parameters for a wide range of materials, from soft soils to firm rocks.

In geotechnical engineering, there are only few in-situ tests that yield a deformation modulus. Many other in-situ geotechnical tests rely on penetration or driving techniques using tools of varying geometry. In such cases, the deformability of the soil mass is assessed based on empirical correlations between the measured physical quantity, which is entirely dependent on the testing technique, and the deformation under static loading.

However, the PMT requires a high level of care, particularly during the drilling phase, as the pressure–volume measurement results are sensitive to the test execution conditions.

Normative documents, professional guidelines and articles by specialists (Baud et al., 2005; Lopes, 2022) have clearly emphasized best practices for conducting the test; nevertheless, the quality of the preliminary borehole, regardless of the length of drilling intervals, which depends on lithology, remains a critical factor. In day-to-day practice, despite its widespread use in french geotechnics, the test still presents several specific challenges.

One of the criticisms, as highlighted by C. Jacquard during his Coulomb Lecture in 2016, is that the true limit pressure of the soil (p^*_{lM}) is rarely, if ever, measured—since the probe is almost never inflated to the point of doubling its volume, due to the risk of it bursting in the borehole. In such situations, extracting the probe with the sheath detached from the rings becomes hazardous, as the now larger-diameter probe can be lost during the removal of the drilling casing (e.g., due to the rupture of one of the rods used to lower the pressuremeter). The entire assembly can then only be extracted by pulling.

As a result, even when the test is well conducted, only a «minimum» value of the limit pressure is often obtained. Extrapolating to determine p^*_{lM} demands a high level of expertise and entails considerable risk in the context of potential legal disputes.

2.3. The art of correlation or intrinsic parameters

In geotechnical engineering, it is not easy to rely on in-situ tests capable of defining mechanical properties that are independent of the geometric characteristics of the testing device, the level of soil deformation, the loading rate, and whether conditions are drained or undrained. This is probably why our profession relies on a remarkable number of correlations between parameters, many of which lack a true scientific basis. Intrinsic mechanical parameters are cohesion, internal friction angle, compressibility, Young's Modulus, Poisson's ratio, swelling coefficient (Reiffsteck et al., 2012).

The PMT is the only in-situ test that offers well-controlled and easily modelled boundary conditions, and that provides information under continuous loading, taking the soil from the small-strain domain to the large-strain domain (Cambou et Bahar, 1993). Several authors discuss the use of PMT results to define intrinsic parameters (Cambou et Bahar, 1993; Monnet, 2013; Lopes, 2022).

The PMT remains an extraordinary tool, but its widespread use doesn't always allow us to exploit its full potential. Boring techniques and the quality of boreholes before testing is often inadequate.

3. Dynamic compaction

3.1. Expansion of application fields

In France, the use of dynamic compaction in urban and peri-urban areas has accelerated since 2008 (Fig. 3). In addition, Rapid Impact Compaction (RIC) quickly adapted to this urban environment (Fig. 4). The special feature is the use of dynamic compaction to limit soil excavation, by means of a high level of densification, and to reduce the removal of material from the site (Brûlé et al., 2010, 2020, 2024).

Figure 3. Dynamic compaction in urban areas.

Figure 4. Rapid impact compaction (R.I.C.) for brownfields.

3.2. Prediction of soil improvement rates

In addition to obtaining higher mechanical parameters, a major challenge now is to estimate the total settlement of the working platform after densification. In fact, customers want a precision of 5 to 10 cm to calculate their backfill budget and stick to it.

Let's look at the information provided by the feedback. We need several pieces of information: the depth of influence of the dynamic compaction, the rate of improvement for mechanical parameters and the induced settlement of the platform.

3.2.1. Depth of influence D

If the magnitude of improvement is closely related to the lithology of the soil to be improved, the depth of improvement D for the impact techniques becomes an important design parameter. Menard and Broise (1975) and Mitchell (1981) provided (Eq. 1) a method to estimate the depth of significant effect of the compaction, D in meter, as a function of the square root of the dynamic compaction energy $E_{DC} = W_{mass}H_{fall}$, expressed in tons-meter, with W_{mass} the mass of the punch pounder, H_{fall}, the maximum height of fall of the pounder.

$$D = \sqrt{E_{DC}} \tag{1}$$

Leonards and co-authors (Leonards et al., 1980) analyse few cases and conclude (Eq. 2):

$$D = \frac{1}{2}\sqrt{E_{DC}} \tag{2}$$

Lukas (Lukas, 1980) suggested (Eq. 3):

$$D = 0.65\ to\ 0.8\sqrt{E_{DC}} \tag{3}$$

Varaksin (Chu et al., 2009) refines the equation Eq. (3) as follows (Eq. 4):

$$D = C\delta\sqrt{E_{DC}} \tag{4}$$

Where: C is the type of drop. Its value is given in Table 1. δ is a correction factor. $\delta = 0.9$ for metastable soils, young fills, or very recent hydraulic fills and $\delta = 0.4 - 0.6$ for sands.

Table 1. Values of C coefficient (In Chu et al., 2009)

Drop Method	Free drop	Rig drop	Mechanical winch
C	1.0	0.89	0.75

Drop Method	Hydraulic winch	Double hydraulic winch	
C	0.64	0.5	

3.2.2. Ratio of improvement

At the thickness D of the improved soil, the ratio $f(z)$ of improvement in densification with respect to depth is given by the following Eq. (5) from Varaksin and Racinais (Varaksin and Racinais, 2009):

$$f(z) = \frac{f_2 - f_1}{D^2}(z - z_{NGL})^2 + f_1 \tag{5}$$

Where z_{NGL} is the natural ground level, f_1 is the maximum improvement ratio observed at the ground surface, and it is dimensionless.

The value of f_1 may be estimated by $f_1 = 0.008\boldsymbol{\varepsilon}_{DC}$ where $\boldsymbol{\varepsilon}_{DC}$ is the energy in tons-meter/m^2; f_2 is the improvement ratio at the maximum depth of influence that can be achieved.

An example of f is shown in Fig.5. This example shows that f reaches a value of 3 near the surface, i.e. an improvement of 300%.

We will discuss in §4, the relevance of this estimate with a real case treated in an urban area by Menard.

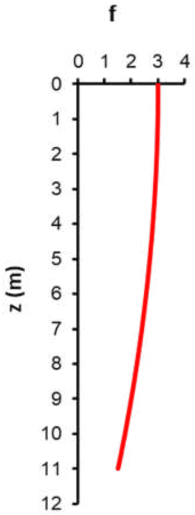

Figure 5. Example with this set of parameters: $C = 0.89$, $\delta = 0.9$, $W_{mass} = 25\,t$, $H_{fall} = 15\,m$, $D = 15.5\,m$, $f_1 = 0.008E_{DC}$, $f_2 = 0.05$.

4. Case study

4.1. Site description

The construction of a new industrial and commercial area in 2009 at the Givors' former glass factory area in France involved heavy dynamic compaction work (Brûlé et al, 2010; Bitri et al., 2013). For the purpose of founding the new buildings, 7–15 m of well-graded gravel backfill lying on geotechnical bedrock, has been densified by dynamic compaction (DC). The groundwater level is about 5-6 m below the surface. In order to assess the quality and depth of ground compaction, pressuremeter test and cone penetration tests are often performed before and after compaction. The test area was quite specific, as the compaction was particularly intense, with the working platform having been lowered by almost 1 m. In the submerged loose granular soils, the probe inside the slotted tube was driven.

4.2. PMT results

4.2.1. Pressurmeter limit pressure

Fig. 6 shows average value curve standard deviation curves for p^*_{lM} ($D = 15.5\,m$, $C = 0.89$, $\delta = 0.9$, $W_{mass} = 25\,t$, $H_{fall} = 15\,m$). Fig. 7 shows the improvement of p^*_{lM} values after compaction (left hand side) on 8 m.

Over the thickness, this represents an average improvement of 3.5 times the initial value of p^*_{lM}. The red curve on the right (Fig. 7) shows the values predicted by the formula of Varaksin and Racinais. The formula appears to be conservative in this case, but the DC carried out was very intense.

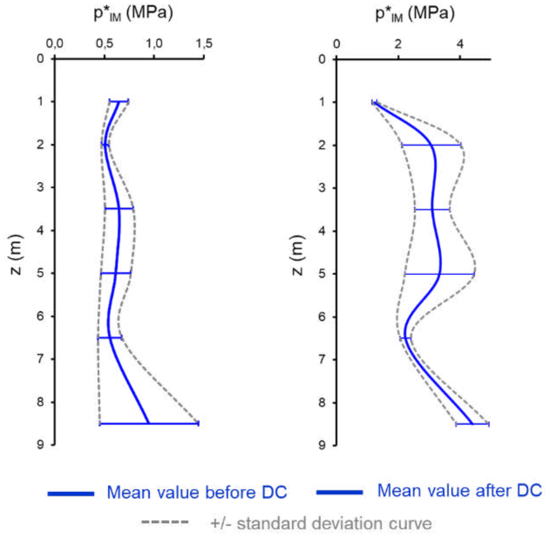

Figure 6. Improvement of p^*_{lM} parameter before and after DC The curves represent the average value of the PMTs conducted at the same depth across all boreholes.

The set of parameters is given in the figure caption (Fig. 7). Fig. 6 shows that the standard deviation varies significantly with depth in both graphs. This variability is partly due to the differing number of measurements at each depth, and it reflects a heterogeneous lithology within the first 8 meters, characterized by fill materials and alluvial deposits, which may include torrential material. The presence of the water table around 5-6 meters likely also influences the PMT results.

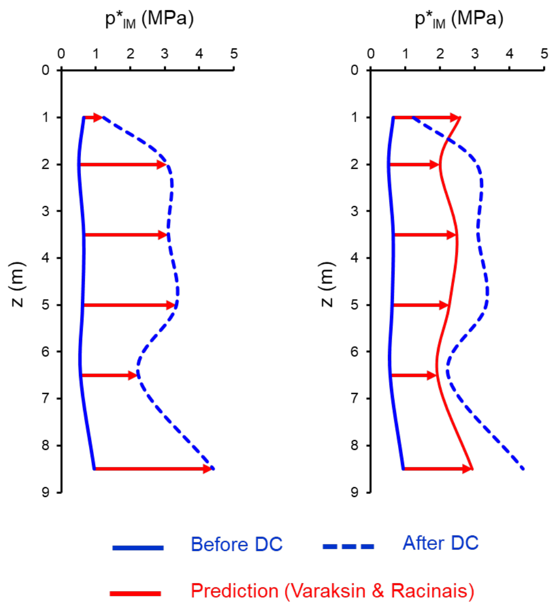

Figure 7. Improvement of p^*_{lM} parameter before and after DC (left) and prediction of the rate of improvement according to the red curve (right). The set of parameters for this worksite: $C = 0.89$, $\delta = 0.9$, $W_{mass} = 25\,t$, $H_{fall} = 15\,m$, $D = 15.5\,m$, $f_1 = 0.008\boldsymbol{\varepsilon}_{DC}$, $f_2 = 0.05$. The curves represent the mean value of the PMTs conducted at the same depth across all boreholes.

4.2.2. Pressuremeter modulus E_M

Over a thickness of 8 m, the average modulus E_M was estimated at 18.4 MPa. The value of the initial modulus was 8.2 MPa (Brûlé et al., 2010). The control tests were carried out approximately three weeks after the dynamic compaction.

The settlement of the initial soil under a vertical distributed load of 100 kPa is 3.3 cm ($\alpha = 1/3$, $E \approx E_M/\alpha$). After densification, the settlement is limited to 1.4 cm.

4.3. CPT results

It can be observed that the tests conducted with the static penetrometer (CPT), at 25 cm intervals, show more significant variations in soil characteristics (Fig. 8). This is, of course, due to the finer sampling interval, but also to a contrasting lithology that the PMT does not capture.

The presence of the water table is also clearly observed, with a temporary lack of improvement around 5 meters depth, likely due to the time required for pressure dissipation following dynamic compaction (aging effect). Soil improvement is not observed at 8 meters in depth, as is in the PMT results.

It remains challenging to compare the results of different in situ tests (Hamidi et al., 2010).

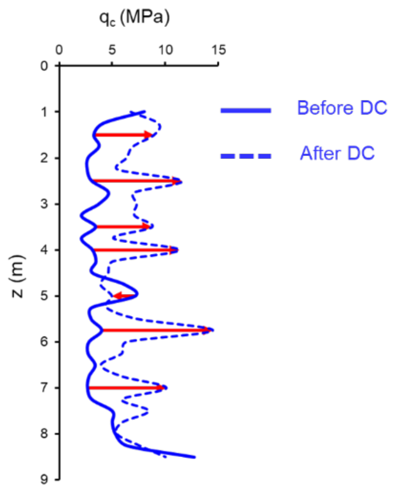

Figure 8. Improvement of cone tip resistance q_c parameter before and after DC. The curves represent the mean value of the CPTs conducted at the same depth across all boreholes.

4.4. MASW results

Using the MASW (Multiple Analysis of Surface Waves) method before and after dynamic compaction, an average improvement of approximately 3% of $V_{s,30}$ (Eq. 6) was observed (Fig. 9). The value of $V_{s,30}$ (parameter from EN 1998 or Eurocode 8; average value of propagation velocity of shear waves in the upper 30 m of the soil profile at shear strain of 10^{-5} or less) is defined as follows, with h_i and v_{si}, respectively the thickness and the velocity of the soil layer i :

$$V_{s,30} = \frac{30}{\sum_{i=1}^{n} \frac{h_i}{v_{si}}} \tag{6}$$

The $V_{s,30}$ parameter can be derived from a profile such as the one shown in Fig. 9 and is used to define soil classes (A to E).

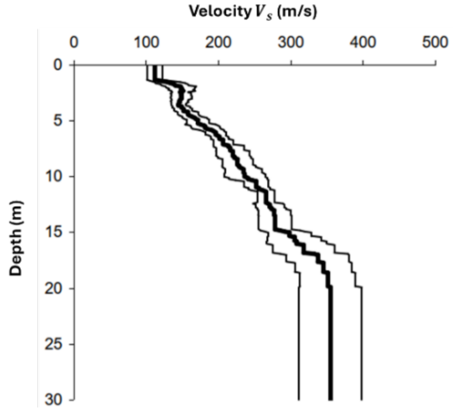

Figure 9. MASW (Multichannel Analysis of Surface Waves). Mean shear wave velocity V_s profile (thick black line) as a function of depth after dynamic compaction (from Brûlé et al., 2010 and 2013 - courtesy of A. Bitri), with standard deviation (two thin curves).

The modest increase is consistent with expectations, as the strain range involved in these techniques lies within the pseudo-linear portion of the curve shown in Fig. 10. The other reason is the significant improvement in velocities in the first few meters below the ground surface, while the average is calculated over a 30 m thickness.

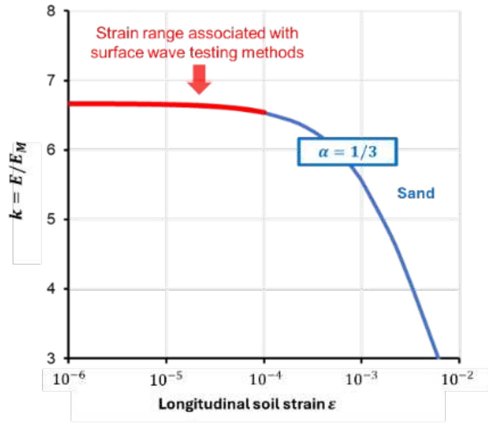

Figure 10. «k-curve» for sand. α is the Menard rheological factor, E is the modulus for the strain range of interest and E_M is the Menard modulus. The red curve illustrates the typical strain range observed in soils when applying surface wave testing methods.

5. Influence of the ARSCOP project on ground densification practices

We now estimate the initial settlement value for the project of the §4, using the ARSCOP relationship $E = kE_M$. u_z and ε_z are respectively defined as the settlement and longitudinal strain of the soil under homogeneous static loading. The indices $bef.DC$ and $aft.DC$ refer to the situation before and after compaction, respectively.

The initial strain is $\varepsilon_{z\ bef.DC} = \Delta h_{soil}/H_{total} = 0.033/8 = 4.1.10^{-3}$. The value of $k_{bef.DC}$ at the first iteration is 3.65. After few iterations, we found $k_{bef.DC} = 4.21$.

$$E_{M\ aft.DC} = \frac{k_{bef.DC} \times E_{M\ bef.DC} \times \frac{u_{z\ bef.DC}}{u_{z\ aft.DC}}}{k_{aft.DC}} \tag{6}$$

$$k_{aft.DC} = f(\varepsilon_{z\,aft.DC} = \frac{u_{z\,aft.DC}}{H_{total}}) = \frac{1}{0.15+30\varepsilon_{z\,aft.DC}} \quad (7)$$

To reduce displacement from 3.3 cm to 1.4 cm, the coefficient k must increase by 16%. This time, the value is directly derived from Eq. (7) then Eq. (6) without iteration.

This result is an increase of the initial modulus E_M from 8.2 MPa to 15.9 MPa (as shown in Eq. (6) and Eq. (7)). Compared to the approach $E \approx E_M/\alpha$, where E_M is 18.4 MPa, the ARSCOP method yields a 16% improvement for the modulus (Fig. 11). Basically, we no longer need to reach 18.4 MPa for the Menard modulus, but 15.9 MPa to achieve the same settlement performance under static loading after dynamic compaction.

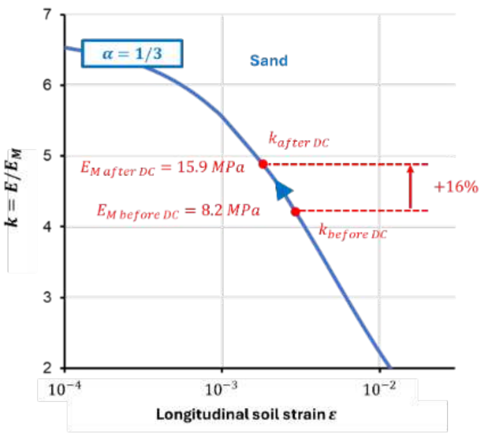

Figure 11. «k-curve» for sand. α is the Menard rheological factor, E is the modulus for the strain range of interest and E_M is the Menard modulus.

6. Conclusions

The ARSCOP national project contributes significantly to improving the quality of soil investigation using the PMT. In the specific case of dynamic compaction, the use of performance prediction curves combined with the «*k- curves»* enables the optimization of the technique's performance.

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