Determination of soil deformation modulus *E* based on the results of the Ménard pressuremeter tests

Détermination du module de déformation du sol *E* à partir des résultats des essais pressiométriques Ménard

Marek Tarnawski[1#], *Tomasz* Tarnawski[1]

[1]*Geoprojekt Szczecin, ul. Tartaczna 9, 70-893 Szczecin, Poland*
[#]*Corresponding author: m.tarnawski@geoprojekt.szczecin.pl*

ABSTRACT

Pressuremeter tests offer the benefit of rapidly obtaining parameters that characterize the compressibility and capacity of soil directly, eliminating the reliance on often questionable correlations. However, the initial segment of the pressuremeter curve is affected by measurement errors due to soil structure relaxation or disturbance during drilling, which leads to an underestimation of the deformation modulus derived from the test results. Acknowledging this issue, Louis Ménard, the inventor of the pressuremeter, referred to this parameter as the "pressuremeter modulus“ (E_M) and devised a unique method for calculating settlement. This method has gained traction globally, particularly in Ménard's native France and other French-speaking nations. In contrast, Polish standards and educational programs tend to favor alternative methods for calculating the settlement of building subsoil, primarily using the deformation modulus *E* or oedometer modulus. Consequently, the pressuremeter serves as a supplementary "special test“ to other geotechnical investigations. Ironically, the number of labor-intensive and costly laboratory compressibility tests is frequently inadequate. Considering this, research has been conducted to explore the potential for determining the deformation modulus *E* through an innovative method that transforms the results of pressuremeter tests. This article outlines the findings from the initial phase of this research, which involved developing an extension of the "Presjometr“ program, with the results proving to be promising.

RESUME

L'avantage des essais pressiométriques est l'acquisition rapide des paramètres décrivant directement la compressibilité et la capacité du sol, sans avoir recours à des corrélations souvent discutables. Mais la première partie de la courbe pressiométrique est encombrée d'erreurs de mesure résultant de la relaxation ou de la perturbation de la structure du sol lors du forage. Cela sous-estime la valeur du module de déformation obtenu à partir des résultats d'essai. Conscient de cela, Louis Ménard, l'inventeur du pressiomètre, a appelé ce paramètre "module pressiométrique" (E_M) et a développé une méthode spéciale de calcul du tassement. Cette méthode a gagné en popularité dans le monde entier, mais principalement en France, la patrie de Ménard, et dans les pays francophones. Les normes polonaises par exemple (ainsi que les programmes d'études) préfèrent d'autres méthodes de calcul de la souplesse du sous-sol des bâtiments. Leur paramètre de base est le module de déformation *E* ou module oedométrique. Le pressiomètre en tant qu'"essai spécial" ne fait donc que compléter d'autres études géotechniques. Paradoxalement, le nombre d'essais de compressibilité en laboratoire, longs et coûteux, est souvent insuffisant. Compte tenu de ce qui précède, une étude a été menée pour établir les possibilités de détermination du module de déformation *E* à l'aide d'une méthode innovante de transformation des résultats des essais pressiométriques. L'article décrit les résultats de la première étape de cette étude, qui a consisté à développer une extension du programme « Presjometr ». Les résultats décrits ci-dessous se sont révélés prometteurs.
.

Keywords: pressuremeter parameters, pressuremeter vs. deformation modulus.

1. Introduction to settlement issues

When soil is subjected to load, forces are transmitted between individual grains, which can result in soil compaction or particle movement, potentially leading to slippage if the tangential force surpasses the shear resistance. By compressing soil that cannot expand laterally, despite local intergranular slippage, overall slippage within the soil mass cannot occur. When the soil can expand laterally, slippage will happen when a force exceeding the soil's shear strength is applied [Lambe and Whitman, 1977]. The oedometer test exemplifies the first process, while the pressuremeter test reflects the second. The next load step (typically double the previous one) of oedometer test is added after settlement stabilization. During the pressuremeter test, loading is applied with equal pressure steps, every minute. These differences result from the fact, that in the oedometer the soil sample is subject to consolidation and the pressuremeter probe penetrating the soil is first counteracted by the horizontal geostatic stress and then, in a changing manner, by the mechanical properties of the soil.

The intricate relationship between additional stress and soil deformation is influenced by several factors, including the characteristics of the soil being loaded, such as its strength, compressibility, granulation related to permeability, and water saturation. Another crucial aspect is the ratio of the load magnitude to the soil's strength and the rate at which they increase. Time also plays a significant role. Soil exhibits properties of an elastic, viscous, and plastic medium [Glazer, 1985], and its response to loading is characterized by rheological models[1]. However, challenges in acquiring parameters' values for these models hinder their practical use. A common approach to the analysis of settlement under a certain, but not excessive, load is to divide it into three phases (Fig. 1).

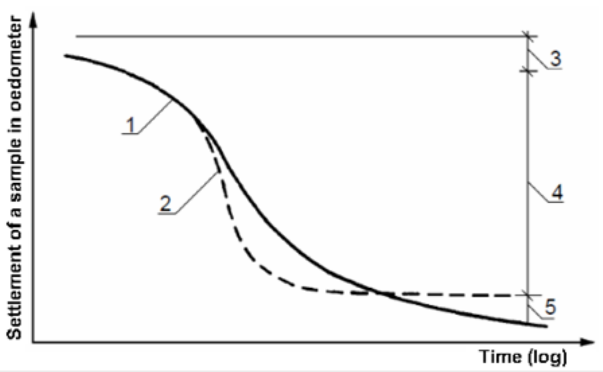

Figure 1. Settlement of loaded soil [Biernatowski et al., 1987] 1. consolidation curve in the oedometer test, 2. settlement curve according to the classical Terzaghi approach, 3. instant settlement, 4. primary consolidation, 5. secondary compressibility.

Immediate settlement occurs in all soils, being of crucial importance in coarse-grained ones (sands, gravels), especially those that are unsaturated. In fine-grained soils (silts, clays) and organic soils, it is noticeable at a low moisture content S_r. If, as is usually the case, the pores of the fine-grained soil are completely (or almost completely; $S_r > 0.9$) filled with water, elastic settlement plays a minor role. Water takes over almost the entire increase in load. Consolidation consists in the outflow of water from the pores to places with lower pressure. The clear and rapid deformation observed in the oedometer test after the first, slight load is applied, results mainly from the restoration of natural structure of the sample after its relaxation or disturbance during collection, transport and assembly to oedometer ring. The situation is similar at the beginning of the pressuremeter test. The last stage indicated in Fig. 1 is secondary consolidation: a rheological phenomenon in the soil skeleton called its creep.

In the analysis of soil settlement under load, it is essential to incorporate a parameter known as modulus into our calculations. The modulus represents the ratio of stress (the force applied to the surface) to the resulting deformation (either compression or extension) of the material due to this stress. This definition applies to solid materials exhibiting elastic characteristics, meaning their deformation is initially proportional to the applied stress, and they revert to their original shape once the stress is removed. These concepts were articulated as early as the 17th and 18th centuries, notably through Hooke's law and Young's modulus.

Even materials that appear elastic at first glance, such as polymers and metals, have an elastic limit beyond which they start to display plastic characteristics under excessive stress. In a fragmented, three-phase medium like soil, elastic and plastic behaviors coexist. Consequently, the conventional Young's modulus is not suitable for calculating settlement. Instead, the modulus of primary deformation E_o is used for soils. Additionally, the oedometer modulus E_{oed} and the pressuremeter modulus E_M can also be applied. These moduli should be determined within the load range where elastic deformations and the primary consolidation process are significant. This is illustrated in Figure 1, which shows the reaction of the soil under a certain load. To determine the modulus, it is necessary to apply increased stress, which leads to deformation (see Fig. 2). Analyzing the load-strain curves presented in Fig. 2, it is evident that the slope variability of curve 1 (oedometric) is markedly different from the other two curves. The slope decreases beyond zone A, ultimately nearing a constant level. This indicates that the oedometer modulus, derived from successive load increments and strain measurements, is on the rise. The peak values are a result of testing artificially consolidated soil under increasing loads. The upward curvature of curves 2 and 3 signifies the transition between the zones of pseudo-elastic strains B and plastic strains C.

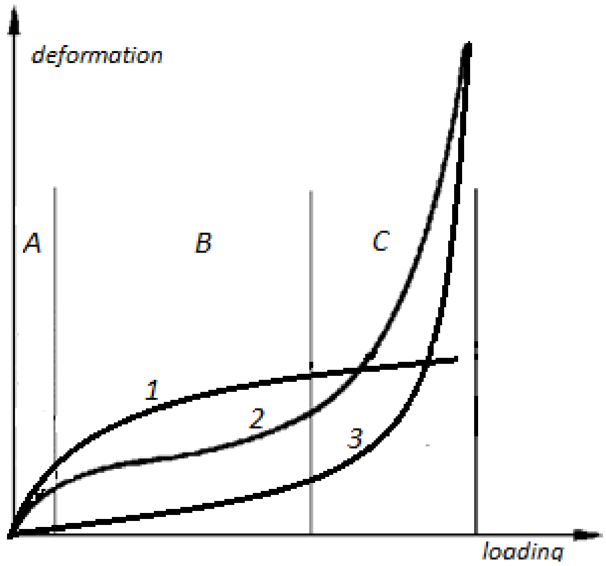

Figure 2. Load-strain curves. 1 – from oedometer test, i.e. without the possibility of lateral expansion, 2 – from pressuremeter test, 3 – reaction (deformation) of soil in the deviatoric stress field, after [Ménard and Rousseau, 1962]. A – zone of elastic deformations or restoration of the natural state of the sample or the wall of the hole, B – zone of microplastic deformations, also called pseudo-elastic, i.e. consolidation zone, C – zone of (large) plastic deformations (does not apply to curve 1). Source: own study.

[1] Rheology is a branch of physics that deals with the deformation ("flow"; *panta rhei*) of matter over time.

The oedometer (curve 1), pressuremeter (curve 2) and deformation (curve 3) moduli should be identified in zone B. Theoretical curve 3 may reflect the actual conditions at the ground surface. At greater depths, such as at the foundation level, settlement should not occur until the additional load from the structure surpasses the vertical geostatic stress that has been removed by excavation, disregarding any potential soil relaxation. The shape of curve 3 in zone B suggests that as additional stresses increase, the deformation modulus will gradually decline, leading to a non-linear increase in calculated settlements. The primary deformation modulus E_o is determined by the ratio of the increase in effective stress $\Delta\sigma'_i$ to the unit deformation (compression) of the considered soil layer $\Delta h'_i/h_i$. Therefore $E_o = \Delta\sigma'_i\, h_i / \Delta h'_i$. The formula for the oedometer modulus is the same, but the deformation modulus of a given soil will be smaller than the oedometer modulus due to the horizontal deformations ε_x and ε_y. The numerical ratio of horizontal to vertical deformations, i.e. ε_x (=ε_y) / ε_z is Poisson's ratio ν_o. The horizontal stresses σ'_x and σ'_y are consistently less than the vertical stress σ'_z in a ratio defined by the lateral expansion coefficient (or rest pressure) K_o. The value of K_o (=ν_o / 1 − ν_o) and the relationship between E_o on E_{oed} are derived through a complex reasoning process [Wiłun, 2000], resulting in the equation $E_o = \delta E_{oed}$, where $\delta = (1 + \nu_o)(1 - 2\nu_o) / (1 - \nu_o)$. When using data from curve 2 or 3, we must ensure that the B/C limit is not exceeded during the design phase. Oedometer tests solely enable the calculation of settlement values; therefore, load-bearing capacity parameters must be assessed using an alternative method.

2. Performing and interpreting the results of pressuremeter tests.

The first phase of the pressuremeter test is not a soil test. It focuses, rather, on restoring in situ parameters through the pressure applied by the probe. Recognizing the critical significance of the accuracy of the soil cavity (including diameter and wall integrity) and the proper setup of equipment for reliable test outcomes, the procedures for executing pressuremeter surveys were formalized by the French Ministry of Equipment, Housing, and Transport (MELT) in 1971. Subsequently, the AFNOR NFP NF P 94-110 Ménard pressuremeter test standard was introduced in 1991. Its revision in 1999 laid the groundwork for the establishment of the European standard (EN ISO). The Polish version (designated as PN-EN ISO 22476-4) was released in 2015, The second edition of EN ISO from 2021 is currently applicable, although it has not yet been translated into Polish.

Comprehensive guidelines for the proper execution of pressuremeter tests are available in both standards and textbooks, making it unnecessary to reiterate them here. It is essential to emphasize the significance of evaluating existing data on lithology, hydrogeology, and anticipated soil strength within the designated exploration area. If the research plan encompasses additional field tests, it is recommended to conduct those first. The insights gained from these tests will not only aid in the accurate execution of the pressuremeter tests but also ensure their appropriate placement within the profiles of the pressuremeter boreholes, which is crucial since these tests are point-based rather than continuous. Textbooks and standards outline various technical options for drilling (making cavities) required for pressuremeter tests, which should be meticulously chosen based on the expected soil conditions. For organic or soft and firm, cohesive soils as well as saturated sands, flush drilling is the preferred method, whereas continuous flight auger is commonly utilized for stiff and hard silts or clays and wet sands. The slotted tube technique is suitable for gravels and over-consolidated soils. It is imperative that the drilling equipment is specifically designed for this testing method; using an usual drill with the necessary diameter significantly increases the likelihood of test failure.

Once the water level in the volumetric eyepiece has stabilized, the testing commences with the first pressure increase. The automatic recorder captures the volumes at 1, 15, 30, and 60 seconds, starting from the moment the target pressure is achieved (which can take anywhere from a few seconds to approximately 20 seconds). The first two readings serve as control values, while the third and fourth are essential for interpreting the test results. After 60 seconds, additional pressure increments are applied. Once the final test measurement is recorded, the gas supply is halted, and the pressure is gradually decreased, allowing the water to be expelled from the probe back into the volume meter. The probe's return to its cylindrical shape enables it to be maneuvered freely within the hole.

During the second phase of the test, the soil exhibits proportional deformation in response to the applied loads, while in the third phase, the deformations escalate significantly. The curve created from the data pairs of pressure (p) and volume (V) approaches a specific p value asymptotically. The asymptote of the curve determines the limit pressure of the tested soil, and the nearly linear segment of the pressuremeter curve allows for the determination of a parameter (modulus) that describes the soil's compressibility.

Eighteen years after introducing his invention, Louis Ménard presented his final proposal for interpreting pressuremeter tests and applying their findings to foundation design [Ménard, 1975]. The contemporary standard principles vary only in minor aspects. The core of the interpretation process involved identifying specific points on the pressuremeter curve to facilitate the calculation of compressibility and bearing capacity parameters of the soil being tested.

The maximum stress value is achieved with infinite cavity expansion. L. Ménard got around this impractical scenario by introducing a conventional limit value, termed the "pressuremeter" limit pressure, denoted as p_{LM}. It corresponds to doubling the initial volume of the cavity. The starting point for this "doubling" is the actual (after the phase of reconstruction of the original hole wall) beginning of the test, i.e. volume V_1 (Fig. 3). At this point, the volume of the cavity is the sum of the initial volume of the central measuring chamber V_C (which in typical equipment is 535 cm^3), plus the volume V_1. The doubling of this volume is calculated using the corrected[2] reading on the pressuremeter's volumetric scale as $V_L = V_C + 2V_1$. This limit volume and consequently the corresponding limit pressure p_{LM}[3] is interpolated when the final, corrected volume obtained during the test exceeds V_L If the test is terminated earlier, the extrapolation is performed using two standard methods: the inverse curve (points p, $1/V$) and the hyperbolic fitting. The standard recommends adopting the extrapolation result that yields the lower mean error as calculated by the specified method.

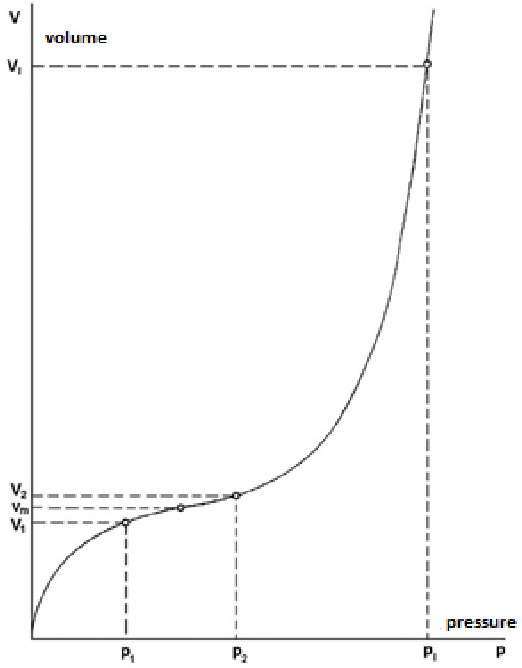

Figure 3. Characteristic points of the pressuremeter curve used to determine the limit pressure p_{LM} and the pressuremeter modulus E_M; from [Tarnawski, 2007]. Explanations in the text.

The pseudo-elastic deformation phase is utilized to determine the pressuremeter modulus of the soil being tested. The protective chambers facilitate a horizontal deformation direction of the pressuremeter probe's measuring chamber, which is perpendicular to the borehole wall. Consequently, the shear deformation modulus can be defined as $G = V\ \Delta p/\Delta V$. This can be converted into the deformation modulus $E_0 = 2(1+\nu)G$ by assuming [Gibson and Anderson, 1961] the volume V_m[4] as V, the Poisson's ratio as a constant, conventional value of $\nu = 0.33$, and calculating the parameters Δp and ΔV from the coordinates of the start and end points of the nearly linear section of the curve: p_1, V_1, p_2 and V_2 (Fig. 3). Ménard's formula for the pressuremeter modulus[5] E_M is expressed as:

$$E_M = 2.66(V_c + v_m)\frac{p_2 - p_1}{V_2 - V_1}. \quad (1)$$

The identification of the starting and ending points of the "modulus zone" initiates with the calculation of the slope[6] m_i for the segments of the curve's second phase (B in Fig. 2). The lowest value of m_i is m_E. The modulus zone includes those subsequent segments that exhibit a slope that is less than or equal to β times m_E. The method for calculating the β coefficient is specified in the EN ISO 22476-4 standard. Previously, the number "6" was included in the complex formula for determining β. In the ISO standard, it was changed to $2\delta V$. Here, δV represents the "tolerance for V", which was initially set as 3 cm^3. Although it may appear similar, the outcome derived is merely a preliminary estimate for identifying the coordinates of the beginning and end of the pseudoelastic range. If the modulus zone derived in this manner contains an insufficient number of sections (as per the standard n < 3), then the δV tolerance range should be increased, thereby expanding the modulus zone, which results in a lower (more conservative) value of the pressuremeter modulus.

By noting the volume measurements on the volumetric scale at the midpoint of the applied pressure duration, specifically after thirty seconds (V_{30}) and at the conclusion of this period (V_{60}), and analyzing differences between subsequent V_{60} - V_{30} values, we can observe certain patterns. Typically, the second difference is smaller than the first, followed by several subsequent differences that are similar and very minimal (close to zero). This suggests that the deformations tend to stabilize after approximately 30 seconds, which aligns with the expectation for pseudo-elastic deformations. Around the midpoint of the test, the differences begin to rise gradually, but irregularly. The curve that connect the points representing the differences V_{60} - V_{30} for subsequent pressure values p is the creep curve. This curve is illustrated below the pressuremeter curve and is utilized to ascertain the creep pressure p_f. The ISO standard has simplified this process by reducing it to drawing two lines with points (p, $\Delta V60/30$). One line incorporates points from the "second group" with smaller values, while the other includes points from the "third group" with progressively larger differences. The x-coordinate at which these two lines intersect is regarded as the initial value of p_f.

3. Advantages and disadvantages of the pressuremeter technique.

Only the end of the initial phase of the pressuremeter test signifies the start of the actual testing process. The assumption that the onset of the rectilinear segment of the curve corresponds to the original horizontal stress p_o did not hold. The value p_o is usually hidden in its rectilinear, second segment of the curve. Consequently, it only

[2] The term "corrected" refers to a value that differs from what was documented during the field test, as it has been adjusted to reflect the results of equipment calibration.
[3] In the past, the symbol p_l was used, as in Fig. 3.
[4] The V_m value represents the present volume of the cavity, determined by adding the initial measuring chamber volume V_C to the mid-point value between V_1 and V_2 on the pressuremeter curve referred to "small" v_m (refer to Fig 3).
[5] Derived in a similar way to the deformation modulus E_o, yet distinct from it and typically lower, which is why it is referred to as "pressuremeter".
[6] Difference in volume relative to pressure difference.

partially represents the pseudo-elastic behavior of the soil being influenced by three overlapping factors: the counteracting relaxation, the diminishing deformations of the disturbed soil zone as pressure increases, and the original deformations of the soil above the p_o threshold (Fig. 4). If we consider the impact of the first two factors as *de facto* volume losses, similar to those corrected during calibration, adjusting the curve by their full values would result in a downward shift, causing it to intersect the p-axis at the p_o point. This would lead to a reduction in the slope at the curve's beginning. The resulting graph (5) would resemble that produced by the self-drilling pressuremeter commonly used in Great Britain [Clarke, 1995]. If the volume losses associated with the disturbance of the borehole wall are not fully accounted for before reaching p_o (curve 4-4a), they will influence the curve's trajectory in the second phase, leading to a decrease in the pressuremeter modulus value. As a result, the slope of the nearly rectilinear phase of curve 1 is steeper than those of curves 3 and (the more) 5.

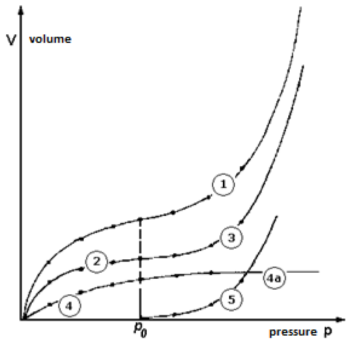

Figure 4. Factors influencing the shape of a typical Ménard pressuremeter curve (1): volume losses used to counteract the relaxation (2) and those resulting from the compression of the disturbed soil ring (4). Curve (2-3) will be obtained if the walls of the borehole are not disturbed at all. The horizontal section of the volume loss graph due to the disturbance of the borehole walls is marked as (4a), which means that this phenomenon has no effect on the shape of the final phase of the curve. Curve (5) will be obtained after subtracting the effect of the relaxation (2) and the disturbance of the borehole wall (4-4a) from curve (1) [Tarnawski, 2004].

The Ménard pressuremeter test cannot yield accurate deformation modulus values. Typically, these values are lower than those derived from alternative methods such as oedometer, triaxial apparatus, or rigid plate [Shields and Bauer 1975, Baguelin et al. 1984]. An effective empirical approach [Ménard and Rousseau, 1962, Ménard 1975] involved the application of a correction (rheological) factor α, which effectively increases the modulus value.

The trajectories of the final sections of the curves deemed accurate exhibit significant similarity. It is important to note that to achieve the same value of the pressuremeter limit pressure p_{LM}, these curves do not converge at a single point defining p_{LM}; instead, they run almost vertically and remain relatively parallel one to another, as V_1 which is a part of the formula for calculating the pressuremeter limit pressure, varies for each curve. The absence of major concerns regarding the validity of the obtained p_{LM} values is not unexpected, since in the third phase of the test, the effects of soil relaxation or wall failure would only manifest if the test cavity was completely improperly prepared.

The method for determining the pressuremeter creep pressure p_f is now defined in the updated ISO standard in a lengthy sentence. To enhance the importance of this parameter, the author conducted a more thorough analysis of the determination method [Tarnawski and Ura, 2015, Tarnawski, 2016]. When we take a closer look at the arrangement of points (p, $\Delta V_{60/30}$) from the third group of results (connected by a diagonal line), we will notice that the $\Delta V_{60/30}$ values usually increase irregularly, but not chaotically. It is generally observed that the last one, two, or three differences tend to rise more sharply compared to the preceding ones. Analyzing the cause of this phenomenon may involve examining the creep process that occurs after surpassing critical loads, where plastic deformations become crucial. Two or three phases can be distinguished within it. In the latter case, these would be: the initial "decaying" creep, followed by secondary "constant rate" or "non-decaying" creep, and finally, third-order deformations, during which the strain rate escalates until failure (Soga, 2005; Fig. 5).

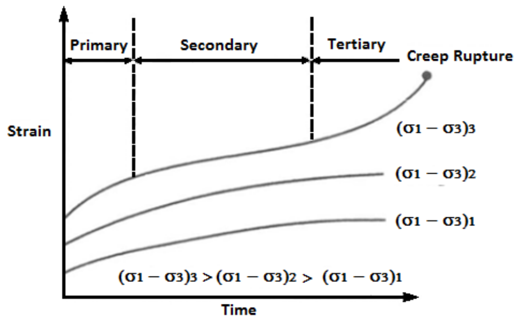

Figure 5. Increasing the deviatoric stress results in progression through successive creep phases: primary, secondary and – through third-order deformations (tertiary) – to Creep rupture [Soga, 2005].

The pressuremeter test is short, yet the creep phenomenon is evident. Consequently, it is probable that the initial and final segments of the oblique line will converge during the inflection phase of the first and third order secondary deformation graph, specifically in the "persisting" creep phase.[7]

Considering the above, the authors suggested that when graphically assessing the pressuremeter creep pressure p_f, only the less step portion of the diagonal straight line should be considered (Fig. 6)[8]. Thanks to this, the p_f value is rationally approximated to the end of

[7] The authors find the second phase of creep shorter than presented in Fig. 5. If this observation holds true, it would imply that numerous structures would undergo (persistent creep), and not just a select few, such as the famous Leaning Tower of Pisa, which has been slowly tilting for centuries before Prof. Jamiolkowski reacted.

[8] A similar illustration is displayed as Fig. D.4 beneath the “long sentence” about it, in the aforementioned, updated (in ‘2021) ISO standard. This is presented there without any explanation or reference to our concept, which was introduced in 2015 at ISP7 in Tunisia.

the pseudo-elastic zone p_2. Then, its conventional calculation ($2\delta V = 6$ in the formula for β) suffices. This method was incorporated into the computer program PRESJOMETR 2.0 by the authors years ago [Tarnawski and Tarnawski, 2005]. Routine testing has consistently validated this approach. If the p_f value remains noticeably above p_2, particularly beyond the subsequent test point, and the E_M/p_{LM} ratio is surprisingly high, it is reasonable to extend the modulus zone following standard guidelines. However, if it includes two[9] or three intervals (pressure jumps) or more, generally no adjustments are necessary or recommended.

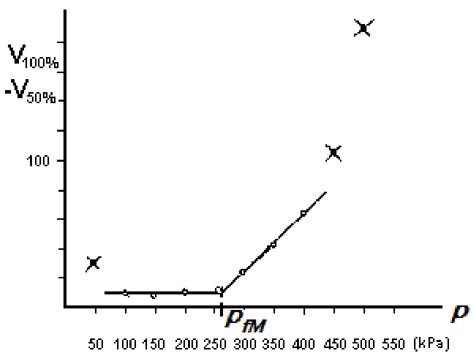

Figure 6. Creep pressure p_f determination scheme according to the authors [Tarnawski, 2016].

Laboratory tests for assessing mechanical properties of soils face several limitations, including a typically small and statistically uncertain number of samples, as well as scale effect due to the limited dimensions of the samples tested. These issues often lead to significant variability in the values of the angle of internal friction (φ) and cohesion (c) derived from tests on an uniform geotechnical layer. Generally, a lower ϕ value is associated with higher c values and *vice versa*, reflecting the local variability of the soil. The use of a cross-shaped rotary probe to assess shear resistance is an option, which, however, fails in soils with a high strength. Evaluating the strength of coarse-grained soils seems to be easier, as it is characterized by a single parameter - the angle of internal friction ϕ. Nonetheless, challenges in sampling necessitate the derivation of correlations from results obtained through other testing methods, particularly probing. Numerous correlations exist, and their outcomes can vary significantly. Given the aforementioned challenges and uncertainties in determining the classical strength parameters of construction soils, the importance of the Menard limit stress p_{LM} should be appreciated.

Similarly, the second strength parameter, known as creep pressure p_f, when determined carefully as described, serves as a clear indicator of the stress level at which the subsoil beneath the structure will start to experience disproportionate settlement in response to increased load. Unlike the so-called permissible loads, this parameter holds genuine physical significance.

The first half of the pressuremeter curve is unfortunately affected by measurement inaccuracies due to soil structure relaxation and/or disturbance during the drilling process. These losses lead to a reduction in the deformation modulus derived from the test results. The Ménard geotechnical calculation method, which utilizes the rheological coefficient α has become popular globally, particularly in Ménard's homeland, France, and in the French-speaking countries. This is largely due to the fact that, for example, Polish or European standards, along with academic programs, tend to favor alternative methods for assessing the compressibility of a building's subsoil, focusing on parameters such as the deformation modulus E, or the oedometer modulus. Therefore in Poland, "Geoprojekt Szczecin" stands out as the sole company that has consistently provided Ménard pressuremeter testing services since the 1970s. The expertise gained has inspired the authors to explore an expanded interpretation of these tests to facilitate the determination of the deformation modulus E, which will be discussed in the remainder of this article.

4. The concept of determining the deformation modulus from the results of the pressuremeter test.

The repeatability of pressuremeter test results in various soils encouraged the creation and analysis of such data sets, as well as correlations of pressuremeter parameters with results of other tests [e.g. Cassan, 2005, Tarnawski 2007, Retamosa, 2013]. The so-called soil classes were established based on the values of the pressuremeter limit pressure p_{LM} and the ratio of the pressuremeter modulus E_M to p_{LM} [Reiffsteck et al., 2013]. A notable method for classifying soils and rocks is the graph $\log(p'_L)/\log(E_M/p'_L)$ called "Pressiorama", which enhances the precision of determining the rheological coefficient α, crucial for predicting settlement [Baud, 2005, Baud and Gambin, 2013]. Recently, clustering analysis [Młynarek et al., 2005]) has gained traction for similar objectives, such as grouping and distinguishing objects based on calculated "distances" between them. In analyzing pressuremeter test results, values such as pressuremeter limit pressure p_{LM}, the modulus from the unloading-loading cycle E_e and the E_e/p_{LM} ratio were examined [Monnet et al., 2015]. The substitution of the pressuremeter modulus with the unloading-loading cycle modulus E_e was due to its perceived stability, being less affected by insufficient drilling quality. A detailed comparison of the secondary modulus E_e and the (primary) pressuremeter modulus E_M shows that the former is typically at least double the latter. This discrepancy indicates that volume losses during testing, caused by unavoidable borehole wall disturbances in the first two phases of the test [Tarnawski 2003 and 2004] lead to an underestimation of the pressuremeter modulus, which the authors identify as a significant concern.

The exploration of effective techniques for extrapolating pressuremeter curves during the early stages of this research methodology led to the development of the inverse curve method, which

[9] The standard suggestion that the number of pressure steps in the modulus zone must be $n \geq 3$ may be excessive when the total number of pressure steps is <10. However, it cannot be limited to one increment.

employs a six-parameter equation. This equation produces a graph consisting of two hyperbolas with opposing concavities linked by a straight line [Van Wambeke and D'Hemricourt, 1975, 1978]. Researchers agree that the hyperbolic method accurately reflects the experimental test points, but it is important to note that the first hyperbola merely attempts to account for the ground unloading in the borehole and the damage to the test cavity wall before the beginning of the test. Consequently, a recommendation was made to exclude these artificial effects from the curve by initiating it from V_1 [Baud and Gambin, 2005]. It refers to the classical elastic-plastic model. However (see curves 2 and 3 in Fig. 2, or curve 5 in Fig. 4) during the phase called elastic the ground does not respond in accordance with Hooke's law. The deformations, although perhaps not distinctly, do not maintain a proportional relationship with the stresses. This behavior differentiates the soil, characterized as an elastic-plastic body (with a predominance of plasticity), from that of a rigid body. [Tarnawski, 2020]

Since no soil can be considered a perfectly elastic material under light loads, nor perfectly plastic under heavy loads, the authors believe that modeling the pressuremeter test with a single hyperbola most accurately reflects the true behavior of loaded soil. Moreover, it is a mathematically simpler solution compared to previous proposals. The authors suggest employing a hyperbolic formula in the following manner:

$$V(p) = V_{p0} + \frac{A}{p_{\infty} - p}, \qquad (2)$$

preferring the model with the fewest practical parameters to estimate from data points. This choice is based on the fact that regression-based parameter estimation tends to distribute measurement errors across the calculated values. The greater the ratio of data points to parameters, the more likely it is that these errors will balance out. Having at our disposal only a handful of data points (see below), we see the merit in reducing the parameter space. Otherwise (i.e. with higher number of parameters needed to describe the curve), the hyperbola could become overly susceptible to measurement errors. At the same time, such formula suits our needs, in the sense that it allows to control the base volume, the vertical asymptote ("limit pressure") and "curvature" ("mode of passing from elastic to plastic characteristic) of the hyperbolic curve.

The parameters are derived from the data points that characterize the right side of the pressuremeter curve, although the selection of the initial value (the 'cut-off') remains an area for further investigation. The estimation of hyperbola parameters is conducted through multiple linear regression utilizing transformed ('linearized') $\{p_i, V_i\}$ data points, as:

$$[A_2, A_1, A_0] = \text{curvefit}([p_i V_i,\ p_i,\ 1]) \qquad (3)$$

Then, the actual parameters $[p_{\infty}, A, V_{p0}]$ are computed from the above linear regression results as:

$$p_{\infty} = \frac{1}{A_2}; \quad V_{p0} = -A_1 p_{\infty}; \quad A = p_{\infty}(A_0 - V_{p0}) \qquad (4)$$

At this point, for the purpose of this preliminary study, it was decided to perform the curve-fitting based on all points (at and) beyond the midpoint of the Menard modulus zone, i.e. for pressures $p \geq ½ (p_1+p_2)$. This selection is advantageous as it minimizes the impact of errors present at the beginning of the curve. However, this method restricts the number of data points available for parameter estimation, leading to an additional arbitrary criterion to exclude tests with fewer than five usable data points. Further studies will show whether this initial assumptions, of only three hyperbola parameters and of skipping the left part of the curve (burdened with above-mentioned errors resulting from damage to the hole wall), will not prove to be too conservative.

5. The first effects of introducing the single hyperbola method

The initial tests of the method proposed above were carried out in February 2025 utilizing results from recent routine works by the company "Geoprojekt Szczecin": in Poznań (central Poland) and Świnoujście (northwestern corner of the country). The verification of the accuracy of the obtained E modulus values involved comparing the E_M/E ratios with the recommended rheological coefficient α for the respective soils. A certain problem was the selection of the pressure range Δp for which the E values were calculated. Initially, two ranges were assumed: from p_0 to p_0 + 100 kPa, with the resulting values termed E_{max} and from p_0 to the "pseudo_p_2" point, which was positioned as far to the right from p_0 as p_2 was from p_1 in the standard test, referred to as E_{min}. (Table 1). The 'Kind of Soil' column in the table includes the details provided in parentheses in the list beneath the table.

Table 1. Pressuremeter vs deformation modulus

Kind of Soil	E_M/E_{max}	E_M/E_{min}	α
MSa; ^{fg}Q$_p$	0,52 (0,47-0,64)	0,76 (0,66-0,92)	0,33 – 0,5
sisaCl; ^{g}Q$_p$ n. cons.	0,53 (0,43-0,60)	0,75 (0,59-0,86)	0,5 – 0,66
sisaCl; ^{g}Q$_p$ over-cons.	0,69 (0,61-0,76)	0,91 (0,82-1,03)	0,66 - 1
Cl; ^{m}Pl n. cons.	0,50 (0,40-0,57)	0,73 (0,67-0,78)	0,66
Cl; ^{m}Pl over-cons.	0,62 (0,55-0,84)	0,94 (0,87-1,35)	1
siSa; ^{m}Ol	0,51 (0,34-0,58)	0,79 (0,64-0,89)	0,33 – 0,66
Cl; ^{m}Ol n. cons	0,58 (0,39-079)	0,85 (0,67-0,98)	0,66

The table presents the results of calculations carried out according to the principles described above. A total of 50 results of tests of various soils were taken into account. They were:

- fluvioglacal, Pleistocene medium sands (MSa; ^{fg}Q$_p$),
- glacial, Pleistocene boulder clays (sisaCl; ^{g}Q$_p$) normally consolidated (n.cons.) and over-consolidated (over-cons.),

- maritime, Pliocene clays (Cl; ^{m}Pl) normally consolidated and over-consolidated,
- maritime, Oligocene silty sands (siSa; ^{m}Ol) and
- maritime, Oligocene clays (Cl ; ^{m}Ol) normally consolidated.

The findings from initial studies support a positive evaluation of the potential of the newly proposed method for interpreting the pressuremeter test, which allows for determining the soil deformation modulus E using the Ménard pressuremeter. Consequently, the Ménard pressuremeter could evolve into a versatile instrument that equips designers with comprehensive data necessary for accurately assessing the permissible load on a building's foundation (derived from the p_f value) and for estimating anticipated settlements using the well-established parameter, the soil deformation modulus E. The moduli may be defined arbitrarily based on the concept of the direct foundation project for a specific building, such as the "E_{max}" modulus for minor load applications or "E_{min}", which considers the resistance of the entire pseudo-elastic deformation zone of the soil, as is the case with E_M,

A closer analysis of the data presented in Table 1 does not allow us to avoid the problems that remain to be solved. When we compare the E_M/E_{min} ratio with the rheological coefficient α, we observe both similarities (notably in tills and Pliocene clays), and discrepancies (such as lower α in sands). The reasons for this state of affairs may vary. To clarify this and refine the methodology for determining the soil deformation modulus E using the Ménard pressuremeter, it is essential to conduct specialized comparative studies, which the authors' team is presently undertaking.

6. Conclusions

Soil deformations resulting from gradually applied additional loads can be depicted by a hyperbolic curve (for instance, curve 3 in Fig. 2). Initially, these deformations are elastic (proportional to the increasing load), and later become pseudo-elastic, where deformations rise slightly with each additional load increment. Next, once the creep pressure threshold is passed, the deformations transition to a plastic state, escalating rapidly until the soil structure fails. The pressuremeter curve, characterized by two hyperbolas with opposing concavities linked by a straight line (as seen in curve 2 in Fig. 2), accurately reflects the behavior of undisturbed soil only during its second phase, on the right side of the figure. This is due to the relaxation and disturbance of the soil prior to testing, leading to what are known as "volume losses". Consequently, the pressuremeter modulus is typically lower than the actual deformation modulus E. Naturally, Louis Ménard was aware of this when he developed his method for applying pressuremeter test results to foundation design, particularly in calculating building settlements, and introduced rheological coefficients that in effect increase the modulus values. He employed statistical methods for this purpose.

Unfortunately, we are unable to indicate clearly the error associated with the results of a particular test. Therefore, we propose a different approach: utilizing only the second segment of the pressuremeter curve, which is free from measurement errors, and appropriately extrapolating this hyperbola to the left. The resulting modulus values, which increase through this process (as discussed in the previous chapter), give us reason to be optimistic about determining the deformation modulus E using the results from the Ménard pressuremeter test.

We are in the process of developing the scope of comparative studies utilizing alternative methods to assess the deformation modulus. At this point, we believe that the most suitable comparative tests will include:

- laboratory measurement of the deformation modulus in a triaxial compression apparatus and
- CPT probing.

The triaxial apparatus facilitates the design of tests that closely align with both the primary stresses present in the ground and the stresses generated by the pressuremeter probe. If it were possible to find a correlation with the parameters obtained from CPT probing, it would be a step that would make this popular and efficient equipment even more universal.

Upon completion and analysis of these (or possibly some others) studies, we will refine the method described above. We plan to present it then to the global community of pressuremeter testing specialists.

References

Baguelin F., J.F. Jézéquel, D.H. Shields 1984. "Badania presjometryczne a fundamentowanie" "The Pressuremeter and Foundation Engineering" (in [Polish]). Wyd Geol, Warszawa,.

Baud J-P. 2005 "Analyse des résultats pressiométriques Ménard dans un diagramme spectral [log(p_{LM}), log(E_M/p_{LM})® et utilization des regroupements statistiques dans la modelisation d'un site". ISP5 – Int Symp 50 years of pressuremeters, Marne-la-Vallée, Vol. 1, 167–174.

Baud J.-P., M. Gambin 2013. "Détermination du coefficient rhéologique a de Ménard dans le diagramme Pressiorama®". 18th Conf on Soil Mech and Geot Eng ISP6, Paris, 4-2; 1–4.

Biernatowski K., E. Dembicki, K. Dzierżawski, W. Wolski 1987 "Fundamentowanie. Projektowanie i wykonawstwo" "Foundation. Design and Construction" (in [Polish]). Arkady, Warszawa.

Cassan M. 2005. "Le pressiométre et la résistance au cisaillement des sols – Cas particulier des Argiles saturées". ISP5 – Int Symp 50 years of pressuremeters, Marne-la-Vallée, Vol. 1, 699–714.

Clarke B.G. 1995. "Pressuremeters in Geotechnical Design". Blackie Academic & Professional, London.

EN ISO 22476-4:2021 "Geotechnical investigation and testing - Field Testing - Part 4: Prebored pressuremeter test by Ménard procedure" CEN European Standard

Gibson R.E., W.F. Anderson 1961 "In situ measurement of soil properties with the pressuremeter" Civil Eng and Public Work Rev, Vol. 56 № 658, 615–618.

Glazer Z. 1985 "Mechanika gruntów" "Soil Mechanics" (in [Polish]) Wyd Geol, Warszawa.

Lambe T. W., R.V. Whitman 1977 "Mechanika gruntów" "Soil Mehanics" (in [Polish]) Arkady, Warszawa.

Ménard L. 1975 "Interpretation and Application of Pressuremeter Test Results to Foundation Design" Sols Soils № 26.

Ménard L., J. Rousseau 1962 "L'évaluation des tassements, tendances nouvelles" Sols Soils № 1, Paris.

Młynarek Z., J. Wierzbicki, W. Wołyński 2005 "Use of cluster method for in situ tests" Studia Geot et Mech, Vol. XXVII 3-4, 15–27.

Monnet J., J. Hughes, J. Dafni 2015 ”The use of cluster analysis for identification of soil layer – The Grays Harbor Pontoon construction project” ISP7 - Pressio. Int Symp 60 years of Pressuremeters; Tunisian Assoc of Soil Mech, Hammamet, Tunisie, 249–260.

Reiffsteck P., A. Martin, T. Perini 2013 ”Application et validation d'abaque pour la classification des sols àpartir deziaks résultats pressiométriques” 18th Conf on Soil Mech and Geot Eng ISP6, Paris, 4-6; 1–7

Retamosa M. 2013 ”Undrained Shear Strength of Over-consolidated Clays Based on Creep Pressure Results from Pressuremeter Tests” 18th Conf on Soil Mech and Geot Eng ISP6, Paris, 3-5; 1–5.

Shields D., G. Bauer 1975 ”Determination of the Modulus of Deformation of Sensitive Clay using Laboratory and In Situ Tests” Proc. ASCE Spec Con. on In Situ Measurement of Soil Properties, Raleigh, North Carolina, vol. I, 395–421.

Soga K. 2005 ”Time Effects Observed in Granular Materials” The COE Workshop on Evaluation of Mech Beh of Granular Materials, Lecture 3, Hokkaido University, Sapporo.

Tarnawski M. 2020 ”Badania presjometryczne” ”Pressuremeter tests” (in [Polish]) in: „Badanie podłoża budowli. Metody polowe” “Examination of the Building Substructure. Field Methods” (in [Polish]), M. Tarnawski ed, 360-414. PWN, Warszawa.

Tarnawski M. 2016: “The right way to define pressuremeter creep pressure“ Inżynieria Morska i Geot, № 5, 287–295.

Tarnawski M. 2007 “Zastosowanie presjometru w badaniach gruntu“ “Application of pressuremeter in soil testing“ (in [Polish]), PWN, Warszawa,.

Tarnawski M. 2004 “The Perfect Ménard Pressuremeter Curve“ Arch of Hydro-Engineering and Env. Mechanics; Polish Academy of Sciences, Gdańsk, Vol. LI №. 4, 387–402.

Tarnawski M. 2003 “Shapes of Ménard Pressuremeter Curves“ Proc. 13th Reg. African Conf. of Soil Mech. and Geot Eng.: “The Involvement of Geot Eng in Infrastructure Dev in Africa”. Marrakech, Morocco, 183–190.

Tarnawski M., T. Tarnawski 2005 “PRESJOMETR 2.0: a comfortable and prospective tool for pressuremeter test interpretation” ISP5 Int Symp 50 years of pressuremeters, Marne-la-Vallée, Vol. 1, 369–376,.

Tarnawski M., M. Ura 2015 ”Towards soil profile from pressuremeter data” ISP7 - Pressio. Int Symp 60 years of Pressuremeters; Tunisian Assoc of Soil Mech, Hammamet, Tunisie, 281–288.

Van Wambeke A., J. d'Hemricourt 1975 ”Courbes pressiométriques inverses: méthode d'interpretation de l'essai pressiométrique” Sols Soils № 28, Paris,

Van Wambeke A., J. d'Hemricourt 1978 ”Evolution dans l'interpretation de l'essai pressiométrique” Proc. 3rd Int. Congr. IAEG, sect. IV, Vol 1, , 91-98.

Wiłun Z. 2000 ”Zarys geotechniki” ”The Outline of Geotechnics” (in [Polish]) Wyd Komunikacji i Łączności, Warszawa;, wyd. I-IV