Pressuremeter tests and the hardening soil model for deep excavations in the Athens basin formations

Essais pressiométriques et modèle de durcissement des sols pour les fouilles profondes dans les formations du bassin d'Athènes

Nikos Gerolymos[1#], *Ilias Michalis*[2#], *Nikos Manos*[3#], *Apostolis Ritsos*[4#]

[1]*Professor of Geotechnical Engineering, N.T.U.A., Greece, gerolymos@gmail.com*
[2]*Dr Civil Engineer, Senior Tunnel and Geot. Eng. Consultant, Greece, imichalis@avax.gr*
[3]*Civil Engineer N.T.U.A, Edafomichaniki S.A., Greece, nmanos@edafomichaniki.gr*
[4]*Civil Engineer N.T.U.A, Edafomichaniki S.A., Greece, aritsos@edafomichaniki.gr*

ABSTRACT

Pressuremeter tests conducted within selected geological formations of the Athens basin have been evaluated geotechnically and mathematical expressions are proposed to determine the geotechnical parameters, which are required for design purposes. Pressuremeter proves to be an effective testing method for obtaining valuable information about the strength characteristics of the geological formations of the Athens basin. The results were proved instrumental for calibrating critical geotechnical parameters of the Hardening Soil model, which was adopted in numerous excavation designs in the aforesaid complex geological subsurface conditions, by offering realistic predictions.

RESUME

Les résultats des essais pressiométriques réalisés dans des formations géologiques sélectionnées du bassin d'Athènes ont été évaluées géotechniquement et des expressions mathématiques sont proposées pour déterminer les paramètres géotechniques nécessaires pour des raisons d'études. Le pressiomètre s'avère être une méthode d'essai efficace pour obtenir des informations précieuses sur les caractéristiques mécaniques de résistance des formations géologiques du bassin d'Athènes. Les résultats se sont avérés utiles pour calibrer les paramètres géotechniques critiques du modèle d'analyse du type Hardening - Soil, qui a été adopté dans de nombreuses analyses et etudes d'excavation dans les conditions géologiques complexes de subsurface susmentionnées, en offrant des prédictions réalistes.

Keywords: Athens basin formations, Hardening Soil Model, Menard Pressuremeter, Pressiorama®

1. Introduction

The present work focuses on the evaluation of pressuremeter (PMT) tests, conducted at various locations within the formations of the Athens Basin, where deep excavations and temporary support systems were implemented. These excavations reached depths of up to 45 m. Although the support structures are temporary, they were designed as permanent structures to account for the imposed loads. This approach was adopted due to the proximity of the excavations to historical and residential buildings, as well as the significant seismicity of the region.

The pressuremeter testing equipment and the methodology employed were appropriate for assessing the in-situ response and the properties of geological formations, ranging from soil-like materials to those classified as hard soils or weak rocks. However, in complex geological and geotechnical formations—where pressuremeter theory must be applied with careful interpretation—the reliable performance of these tests requires significant experience to ensure the accuracy of the derived parameters and conclusions.

This study primarily focuses on pressuremeter tests conducted on cohesive, compact, and cemented formations within the spectrum of weak rocks, ranging from extremely fractured to slightly fractured, and of fair to good quality. These formations predominantly belong to the Athenian Schist substratum.

According to the Geological Strength Index (GSI), they range from extremely poor conditions ($GSI < 20$) to fair-to-good conditions, with a maximum $GSI = 65$.

Finite Element Method (FEM) analysis is commonly used to predict ground deformations, settlements, and lateral displacements in serviceability limit state design, which requires the modelling of the elasto-plastic, nonlinear behaviour of geomaterials. To support such analyses, a detailed calibration methodology was developed by establishing correlations between pressuremeter-derived geotechnical properties and the key parameters of the Hardening Soil Model (HSM).

The FE software packages **Plaxis** and **DeepEx** were employed in the analysis of deep excavations, incorporating the HSM to effectively simulate the hardening behavior of the geomaterials and address the limitations of the traditional elasto-perfectly plastic models.

For the preparation of this work, several previous bibliographic references, test results and numerical simulations were considered.

Concerning the performance and the evaluation of the pressuremeter test results, the basic bibliographic reference includes the scientific work, in several articles and references, of Louis MENARD, Michel GAMBIN, Jean-Pierre BAUD, Jean-Louis BRIAUD, Roger FRANK Philippe REIFFSTECK, Serge VARAKSIN, and of F. BAGUELIN, J.F. JEZEQUEL, D.H. SHIELDS.

The GSI, (Geological Strength Index), is being widely used in projects in Greece. Its main references can be found in works by Evert HOEK (since 1994), E.T. BROWN, Pavlos MARINOS, Vassilis MARINOS.

Finally, the Hardening Soil Model was initially proposed by T. SCHANZ and P.A. VERMEER (1999).

2. Geotechnical Investigations

The performed site investigations included the execution of multiple sampling boreholes, pressuremeter boreholes, and comprehensive geological and geotechnical evaluations based on both in-situ and laboratory testing. A total of twenty-one (21) sampling boreholes and seven (7) pressuremeter boreholes were drilled at seven (7) different locations across Athens. The sampling boreholes were positioned close to the pressuremeter boreholes, ensuring accurate correlation of stratigraphy between them.

Based primarily on GSI classification, twenty-five (25) distinct Geotechnical Units (GUs) were initially identified. In the seven pressuremeter boreholes, eighty-five (85) tests were performed. Among these, fifty (50) tests had direct correlations with 164 selected samples used for HSM and GSI assessments, covering twenty (20) different Technical–Geological Units.

This article provides selective descriptions, focusing on the most representative geotechnical units:

- **GU 5**: Athenian Schist, Upper Unit (GSI=18–57)
- **GU 6**: Athenian Schist, Lower Unit (GSI=14–47)

To establish well-defined lower and upper bounds for pressuremeter values:

- **GU 2A** and **GU 2B** (predominantly soil-type formations) were selected for the lower bound
- **GU 7** and **GU 8** (good rock formations, GSI = 42–51) were selected for the upper bound.

The Ménard pressuremeter modulus (E_M) was measured and correlated with both the HSM parameters and the GSI values across all the examined cases. The pressuremeter results proved instrumental in calibrating geotechnical parameters for implementing the Hardening Soil Model, enabling realistic predictions for excavation design in the complex subsurface conditions of the Athens Basin.

This paper presents a methodology for correlating key parameters of the Hardening Soil (HS) model with commonly used mechanical properties of intact rock and rock mass. The proposed correlations are applicable across a wide range of rock mass qualities, characterized by a Geological Strength Index (GSI) of ≥10 and an unconfined compressive strength (σ_{ci}) of the intact rock greater than 1.2 MPa.

While the HS model was originally developed for soil-like materials, its application to rock-like materials in this study represents an innovative first-time use.

3. Geological Structure

The Alpine substratum of the Athenian Basin comprises the "Athenian Schist," a clastic formation with characteristics typical of a flysch-type sequence, locally overlain by Neogene and Quaternary sedimentary formations.

The Athenian Schist is a flyschoid formation consisting of schists and carbonate rocks (e.g., marbles), with alternating layers of shales, meta-siltstones, meta-claystones, and meta-sandstones, intercalated with limestones and conglomerates. Its permeability and mechanical behavior vary significantly, ranging from hard soils to soft or competent rock, depending on the degree of weathering. The prefix “meta -” denotes the metamorphic nature of these formations, resulting from tectonic activity and prolonged exposure to high pressures and temperatures.

The various geological and geotechnical formations within the Athenian Basin, where pressuremeter tests were conducted, are classified into distinct Technical–Geological Units (GUs), based on geological investigations carried out at multiple locations.

From the results of the rock mass classification, the weighted average of the GSI limits (range) was calculated for each GU encountered in the sampling boreholes.

Each Technical-Geological Units (GU) is an engineering geological formation, which can be structured by one or more technical-geological formations, which are structured by one or more rocks. The GSI and the pressuremeter tests, as well as a variety of laboratory tests, contribute to the separation of ground layers, in the individual GU.

In the present work, greater emphasis is placed on the Geological Units (GUs) of the Upper and Lower Units of the Athenian Schist.

In total, the main distinct GUs are:

GU1. Man made deposits. This soil type formation includes: Man made & Artificial Deposits.

GU2. Fluvio-torrential Deposits & Alluvial Fan Deposits: Soil type formations, are the Fine-Grained and Coarse-Grained Soils & Clay of low plasticity, Clayey to Sandy Gravels. Hard soil to rock type formations, are the Claystone, Siltstone, Breccia.

GU3. Crest Limestones: Limestone, Karst Limestone.

GU4. Sandstone - Marl Series: Weathering mantle, Claystone / Siltstone, calcareous Claystone / Siltstone, Sandstone, calcareous sandstone, Breccia, calcareous breccia, Claystone / Siltstone - calcareous Claystone / Siltstone with sandstone intercalations, Claystone / siltstone and limestone alternations, Sandstone - calcareous sandstone with claystone / siltstone - calcareous claystone / siltstone intercalations, Claystone / siltstone and sandstone alternations, Calcareous marl, Marly limestone, clastic / intraclastic marly limestone, Thin-bedded marly limestone, Karstic marly limestone, Limestone, and shear zones.

GU5. Athenian Schist, Upper Unit: Weathering mantle. Meta-Siltstone, Meta-Sandstone, Calcareous metasandstone meta-sandstone, Metasiltstone and metasandstone alternations, Schist (chlorite quartzitic, chlorite epidote, calcareous chlorite), Phyllite, calcareous phyllite, Limestone, Karstic limestone, Limestone and schist (chlorite quartzitic, chlorite epidote, calcareous chlorite) alternations, Phyllite and limestone alternations, Shear zones.

GU6. Athenian Schist, Lower Unit: Shale, Meta-Siltstone, Calcareous metasiltstone, Meta-Sandstone, Shale with metasandstone intercalations, Metasiltstone with shale intercalations, Metasiltstone and metasandstone alternations, Chlorite schist, Limestone, Shear zones.

GU7. Alepovouni Unit: Marble / Dolomitic marble / Dolomite, Schist (calcareous mica quartzite), Marble / Dolomitic marble / Dolomite with schist / sandstone intercalations, Karstic marble / Karstic dolomitic marble.

GU8. Ultrabasic rocks: Serpentinite, Listwanite (carbonated serpentinite), Karstic listwanite (karstic carbonated serpentinite).

GU9. Igneous rocks: Greenstone / Metadiabase.

Groundwater is present either as a continuous aquifer or as localized circulation within more permeable formations. In both cases, it appears at a shallow depth below the ground surface. The measured permeability of the ground ranges from k=10-5 to 10-7 m/sec.

In Fig. 1. typical borehole samples are presented.

Figure 1. Typical Rock type formations, at high measured values (EDAFOMICHANIKI S.A.).
1a. Athenian Schist, Upper Unit. GU 5.5 GSI=33
1b. Athenian Schist, Lower Unit GU 6.6 GSI=28
1c. Ultrabasic Rocks GU 8.3 GSI=50

4. Soil Constitutive Models

In the case of FEM numerical modelling, ground conditions must be appropriately classified, particularly because the stress–strain relationships of ground formations are inherently nonlinear. For such Geological Units (GUs) and rock-type formations, modelling should account for stress changes due to small elasto-plastic deformations that occur even before reaching the design state. These responses can also be evaluated in the laboratory using a triaxial testing apparatus. For completeness, a brief description of several geotechnical constitutive models—ranging from linear elastic to the Hardening Soil Model (HSM)—is provided below.

The **elastic model** assumes a linear relationship between stress and strain. This simplification is typically valid for rapid analytical solutions and serves as a first-order estimate of deformation under service loads that are well below failure conditions, based on a limit equilibrium approach.

The **elastic–perfectly plastic model**, and in particular the **Mohr–Coulomb model**, represent soil behavior up to the onset of failure. These models are commonly used for limit equilibrium analysis and provide a basic framework for initial design stages.

However, most ground formations exhibit **nonlinear stress–strain behavior**, with stiffness decreasing rapidly from an initially high value. These materials often retain some residual strength and exhibit **plastic deformations**, which persist even after partial or full unloading. From soils to massive rock formations, volume changes may occur, or the material may enter a state of continuous shearing without further stress change.

The **Hardening Soil Model (HSM)** is a natural extension of perfectly plastic models and is better suited for more accurate calculations and realistic predictions of ground deformations. Compared to the Mohr–Coulomb model, the HSM uses different constitutive equations and incorporates a **stiffer unloading/reloading modulus**. Additionally, it defines the physical and mechanical properties of subsurface layers using a broader set of parameters, allowing for a more refined representation of ground response under loading and earth pressures. The **presence of groundwater** is a critical loading factor, and its influence is modeled with greater accuracy in HSM, providing a more realistic simulation of in-situ conditions during construction.

Pressuremeter tests reflect the stiffness–strain response at large strains, typically within the shear strain range of $\varepsilon = 10^{-1}$ to 10^{-2}. The self-boring pressuremeter offers improved performance, extending to a broader range of shear strains, from $\varepsilon = 10^{-1}$ to 10^{-4}.

In terms of limiting ground movements and displacements for deep cut-and-cover excavations in residential areas of Athens, strict criteria are typically applied. The most commonly accepted limits are:

- *Vertical settlements*: less than 30–35 mm
- *Lateral (horizontal) displacements*: less than 0.10%–0.15% of the excavation depth (ΔH)

The selection of appropriate computational models and geotechnical parameters is therefore essential, aiming not only to ensure excavation safety but also to provide realistic estimates of ground movements, which in turn inform the design of adequate support measures.

5. Pressuremeter Ménard PMT

5.1. Performance of Pressuremeter tests

Pressuremeter tests were conducted at various depths within these geological formations using a Ménard-type pressuremeter (APAGEO SEGELM, AX/BX).

Key pressuremeter parameters—including the pressuremeter modulus (**E_M**), limit pressure (**P_{LM}**), initial pressure (**P_{oM}**), creep pressure (**P_f**), net limit pressure (**P^*_{LM}**), and the ratio **E_M/P^*_{LM}**—were derived from the test data. The maximum applied pressure did not exceed 10 MPa.

In several tests, the Ménard net limit pressure (**P^*_{LM}**) could not be directly measured and was instead estimated numerically using either the pressuremeter creep pressure (**P_f**) or the maximum pressure achieved during the test. The shape of the pressuremeter expansion curve enabled extrapolation of the limit pressure where necessary.

Pressuremeter-derived parameters were also correlated with laboratory test results, supporting the selection of critical geotechnical design parameters such as the elastic modulus (**E_s**), effective friction angle (**φ'**), shear strength, coefficient of earth pressure at rest (**K_0**), and mobilized shear stress (**τ_f**).

5.2.Pressuremeter parameters

The pressuremeter parameters **E_M** and **P^*_{LM}** for each Technical–Geological Unit (GU), at table 3 are summarized (the table 3 is given at the last page of this article).

- In soil-type formations and cemented hard soil formations, **E_M** ranges from 6 to 645 MPa.
- In GU 5, **E_M** ranges from 17 to 1,392 MPa.
- In GU 6, **E_M** ranges from 51 to 824 MPa.
- In GUs 7 and 8 (representing fair to good quality rock masses), **E_M** reaches up to 5,540 MPa.

In Fig. 2. typical pressuremeter curves are presented, that corresponds to the upper limits of the measured pressuremeter parameters.

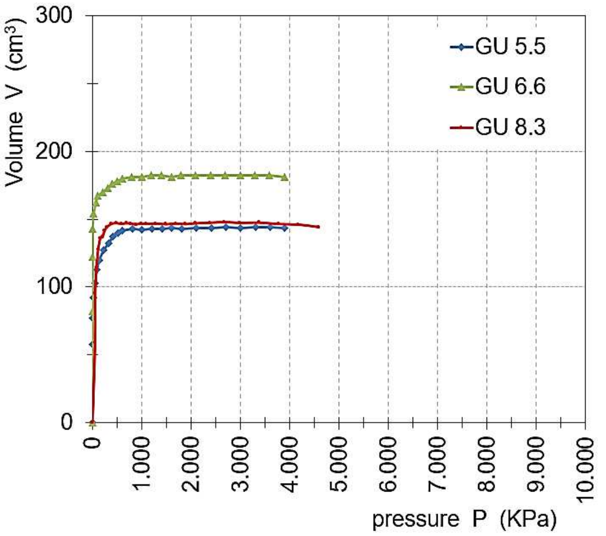

Figure 2. Pressuremeter cuves of Rock type formations with high measured values (EDAFOMICHANIKI S.A.).

2a. Athenian Schist, Upper Unit. GU 5.5 GSI=33, E_M=1.4GPa P^*_{LM}=1,70 P_f=6.5MPa $E_{50.REF}$=0,8GPa

2b. Athenian Schist, Lower Unit GU 6.6 GSI=28, E_M=0,7GPa P^*_{LM}=1,70 P_f=6,4MPa $E_{50.REF}$=0,8GPa

2c. Ultrabasic Rocks GU 8.3 GS1=50, E_M=1.12GPa P^*_{LM}=1,70 P_f=7,7MPa $E_{50.REF}$=4,6GPa

In Fig. 3., the **Pressiorama®**, (Baud J. P. & Gambin M.) is used, for presenting the range of the pressuremeter parameters for the Athens basin Technical–Geological Units GUs 2, 5, 6, 7, 8.

The focus of this paper is on presenting a methodology for correlating the pressuremeter modulus (E_M) with key Hardening Soil Model (HSM) parameters.

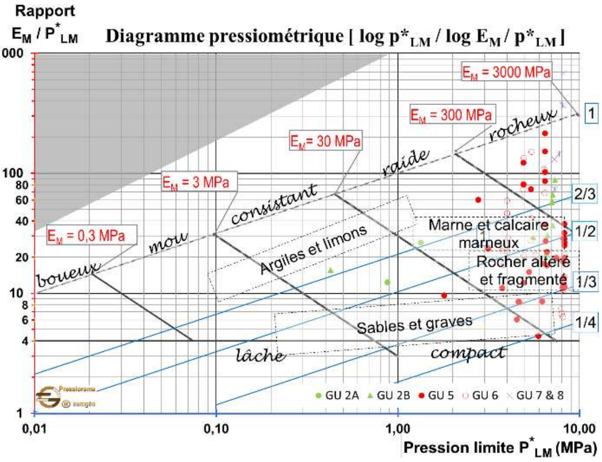

Figure 3. The Pressiorama® of GU2, GU5, GU6, GU7, GU8.

5.3.Pressuremeter Modulus E_M

The rheological factor "α" is an empirical parameter used to describe the soil's resistance to deformation. It is a key factor in interpreting the stress-strain behaviour of the soil under the applied pressure.

The value of "α" provides insight into how the soil deforms under the influence of the pressure applied by the pressuremeter probe.

For Soil type geo materials, peat, clay, alluvium, sand, sand and gravel, the empirical value "α" is defining usually, by using the ratio: E_M/P^*_{LM}.

The value "α" according to SOLS SOILS, Ménard & Rousseau (1962 and 1975), Baguelin F. - Jézéquel J. F. - Shields D.H. (1978), for soil geomaterials and for the settlement calculations, is ranging: 1/4, 1/3, 1/2, 2/3, 1.

For settlement calculations, the pressuremeter modulus E_M has been empirically related to the modulus of compressibility, which is equivalent to the effective Young's modulus, or the "static" modulus of soil (Lebranc, 1982):

$$E_s = E_{YOUNG} = E_{OED} = \frac{E_M}{\alpha} \tag{1}$$

In the Athens basin, for soil type geomaterials and at a waste disposal site, those proposed values of "α" have been confirmed (Ritsos et al 2005, 2013, 2023), in relation also to performed laboratory tests and approaches by using and other parameters, such as the number N at the Standard Penetration test.

After the XV ECSMGE Conference in Athens (2011), the Pressiorama®, (Baud & Gambin) is a valid presentation diagram for the pressuremeter parameters as well as to choose the empirical value "α".

At table 1, and for shallow foundations settlement calculations, recently expressed the proposed value "α" is given, according to NF P94-261, Baud & Gambin (2013).

Table 1. The rheological factor α. (NF P94-261 for shallow foundations. Baud J. P., Gambin M. 2013).

	E_{YOUNG} / E_M			
overconsolidated	3	3	4.5	dense
n. consolidated	4.5	4.5	6	loose
	Clay, Silt	Sand	Gravels	

For Rock type geo materials, at table 2, the proposed values of "α" according to SOLS SOILS, Ménard & Rousseau (1962 and 1975), Baguelin F. - Jézéquel J. F. - Shields D.H. (1978), are given.

Table 2. The rheological factor α for Rock.

	Extremely fractured	**Other**	**Slightly fractured or extremely weathered**
α	1/3	1/2	2/3

It has also been noted that the range $0.8 < \alpha < 1$ can be achieved in OCR cohesive soils and in rock type masses.

However, in harder formations, ranging from hard soils to weak rocks, and particularly for FEM analysis and HSM approaches, the pressuremeter modulus (E_M) cannot be directly used, as it differs from the elastic modulus (E), which is a fundamental soil parameter.

In Fig. 4, the typical ground stiffness-strain response is given, according to Atkinson & Sallfors (1991), Maier (1993) and Reiffsteck (2002). The deformation rate within the measurement range of the pressuremeter test (strain between 1% and 10%) is significantly higher than that of a retaining wall (strain approximately 0.01%). Additionally, the pressuremeter modulus is purely deviatoric, and the deformation measured with the probe is horizontal.

For rock type formations the primary loading modulus needs to be set, that is lower than the elastic unloading-reloading modulus E_{ur}, when considering a triaxial stress approach, or when considering a purely deviatoric stress path.

In the data base of the GU5, GU6, GU7, GU8, a better estimate arises, by using the Eq. 2, that is according to Ménard & Rousseau (1962) and Baud & Gambin (2013)

$$E_{YOUNG} = \frac{E_M}{\alpha^2} \tag{2}$$

According to Baud & Gambin et al (2013, 2018, 2021), by using the Eq. (3) and Eq. (4), the rheological factor "α", can be calculated from the pressuremeter parameters (with $k_E \approx 3$ to 5, $m = 0.5$ and $n = 2$):

$$a = \frac{\left({}^{E_M}\!/_{P^*_{LM}}\right)^{\left(\frac{1}{n}\right)}}{k_E\left({}^{P^*_{LM}}\!/_{P_0}\right)^{\left(\frac{m}{n}\right)}} \tag{3}$$

$$k_{\mathrm{E}} = \frac{\pi+2}{\left(\frac{\ln \mathrm{E_M}}{\mathrm{P^*_{LM}}}\right)^{\left(\frac{1}{3}\right)}} \tag{4}$$

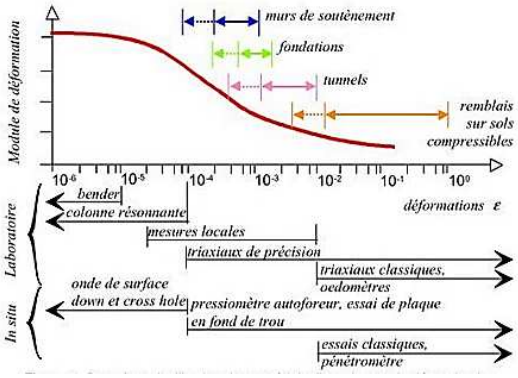

Figure 4. Characteristic stiffness-strain responce. Reduction of shear modulus with level of strain.

6. Hardening Soil Model HSM

6.1. Theoretical background

The Hardening Soil Model (HSM) is an advanced constitutive model implemented in PLAXIS for simulating the nonlinear, stress-dependent behavior of soils. Unlike traditional linear-elastic or perfectly plastic models, the HSM accounts for strain hardening, capturing key geotechnical phenomena such as stress-dependent stiffness, plastic yielding, and preconsolidation effects.

In contrast to the Mohr-Coulomb (MC) model, which assumes constant stiffness, the HSM incorporates three different stiffness parameters:

- Secant stiffness in primary loading (E_{50})–governs the stress-strain relationship in drained triaxial conditions.
- Unloading-reloading stiffness (E_{ur})–controls elastic behavior during unloading and reloading cycles.
- Oedometer stiffness (E_{oed}) – defines soil response under one-dimensional compression.

The model uses a hyperbolic stress-strain relation, as shown at Fig. 5. to represent soil deformation under triaxial loading, ensuring a more realistic description of soil behavior under different stress conditions. The HSM is particularly suitable for soft to medium-dense soils and is widely applicable in deep excavations, tunneling, embankments, and foundation design.

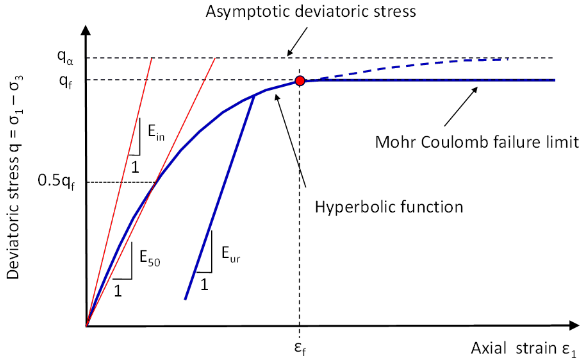

Figure 5. Hyperbolic stress-strain relation in primary loading for a standard drained triaxial test.

Despite its frequent use in practice, its application is limited to soil-like materials, while rock mass behaviour falls outside its range of applicability.

In this paper, a calibration procedure is proposed, focusing on correlating key HS model parameters, such as E^{ref}_{50}, R_f and m with widely used mechanical properties of the intact rock and rock mass (e.g.: σ_{ci}, σ_{cm}, E_{rm}, GSI).

The developed correlations are valid for a wide range of rock mass qualities, characterized by a geological strength index of GSI ≥ 10 and unconfined compressive strength of the intact rock $\sigma_{ci} \geq 1.20$ MPa.

The proposed calibration procedure is defined by the following equations:

$$R_f = \begin{cases} 1-\frac{1}{4}\left(\frac{\sigma_{cm}}{E_0}\right)\left(\frac{E_i}{\sigma_{ci}}\right)^{1.2} & \text{if} \quad \frac{\sigma_{cm}}{\sigma_{ci}} < \frac{E_{rm}}{E_i} \\ 1-\frac{1}{4}\left(\frac{E_{rm}}{E_0}\right)\left(\frac{E_i}{\sigma_{ci}}\right)^{0.2} & \text{otherwise} \end{cases} \tag{5}$$

$$E_{50} = \begin{cases} \frac{1}{8}\sigma_{cm}\left(\frac{2-R_f}{1-R_f}\right)\left(\frac{E_i}{\sigma_{ci}}\right)^{1.2} & \text{if} \quad \frac{\sigma_{cm}}{\sigma_{ci}} < \frac{E_{rm}}{E_i} \\ \frac{1}{8}\sigma_{ci}\left(\frac{2-R_f}{1-R_f}\right)\left(\frac{E_{rm}}{E_i}\right)\left(\frac{E_i}{\sigma_{ci}}\right)^{1.2} & \text{otherwise} \end{cases} \tag{6}$$

$$m = \begin{cases} \frac{1}{4}e^{-0.017GSI} & \text{GSI} \geq 20 \\ 0.5 & \text{GSI} < 20 \end{cases} \tag{7}$$

$$E^{ref}_{50} = \frac{E_{50}}{\left[\frac{c\cos\varphi + \sigma_3\left(z=z_{ref}\right)\sin\varphi}{c\cos\varphi + p_\alpha \sin\varphi}\right]^m} \tag{8}$$

$$E^{ref}_{oed} = E^{ref}_{50}\left(\frac{c\cos\varphi + p_\alpha \sin\varphi}{c\cos\varphi + \frac{p\alpha}{K_0^{NC}}\sin\varphi}\right)^m \tag{9}$$

$$E^{ref}_{un} = 3E^{ref}_{50} \tag{10}$$

6.2.Correlation between PMT and HSM

Having defined the effective Young's modulus as a function of the pressuremeter modulus, and to connect the secant stiffness in primary loading (E_{50}) with the pressuremeter modulus, E_{50} should also be expressed as a function of the effective Young's modulus.

Assuming that the effective Young's modulus, or, equivalently, the static modulus of soil corresponds to an axial deformation of $\varepsilon_1 = 0.1\%$, in the context of HSM the static modulus can be defined as:

$$E_s = \frac{q}{0.001} = \frac{1}{\frac{1}{2E_{50}} + \frac{0.001R_f}{q_f(\varphi,c)}} \tag{11}$$

For typical values of cohesion c and friction angle φ, the ratio E_s / E_{50} ranges from 1 to 2, with rock-like materials approaching 2.

6.3.Correlation between PMT, HSM and GSI

In Geotechnical Engineering, it is essential for multiple methodologies to converge in order to ensure reliable results.

Based on pressuremeter measurements and the classification of ground formations into distinct Technical–Geological Units (GUs)—primarily using the Geological Strength Index (GSI) system—various correlation diagrams of key parameters were developed, for the mainly described to this article GU's 5, 6, 7, 8.

At table 4 (the table 4 is given at the last page of this article) the range of GSI and the relative range of the $E_{50.ref}$ for each one of the referred Technical–Geological Units (GUs), are summarized.

In Fig. 6. the correlation of the Pressuremeter modulus E_M versus to GSI is given.

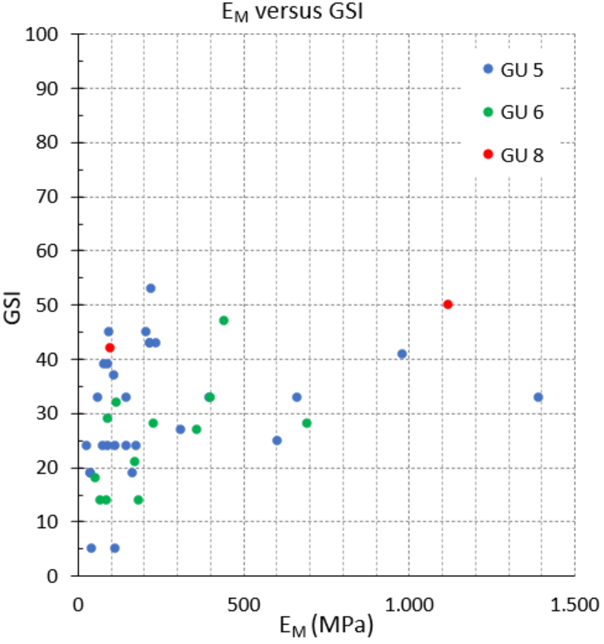

Figure 6. E_M versus GSI, at GU 5, GU 6, GU 8

In Fig. 7. a clear correlation is observed between the reference stiffness modulus $\mathbf{E^{ref}_{50}}$ and the **GSI** values.

In Fig. 8. the correlation of the Pressuremeter modulus E_M versus to E^{ref}_{50} is given.

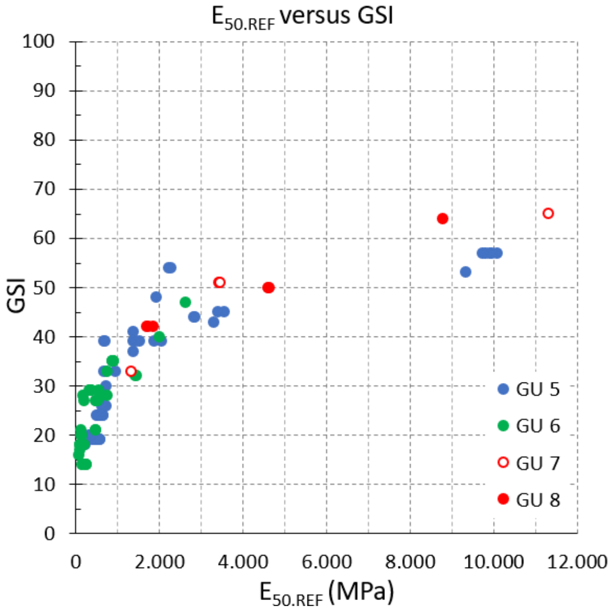

Figure 7. $E_{50.REF}$ versus GSI, at GU 5, GU 6, GU 7, GU 8

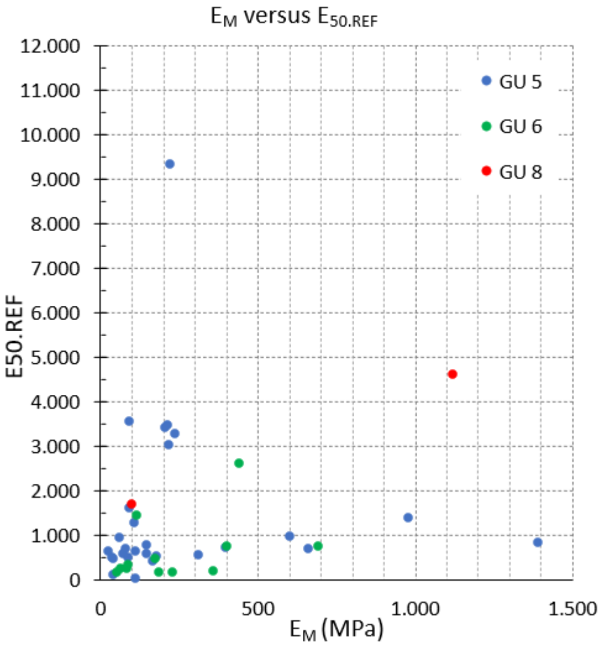

Figure 8. E_M versus $E_{50.REF}$, at GU 5, GU 6, GU 8

Furthermore, in Fig. 9. the characteristic difference in strength between the upper units (**GU5**) and lower units (**GU6**) of the Athenian Schist, are illustrated, while the highest values correspond to the stronger rock formations of **GU7** and **GU8**.

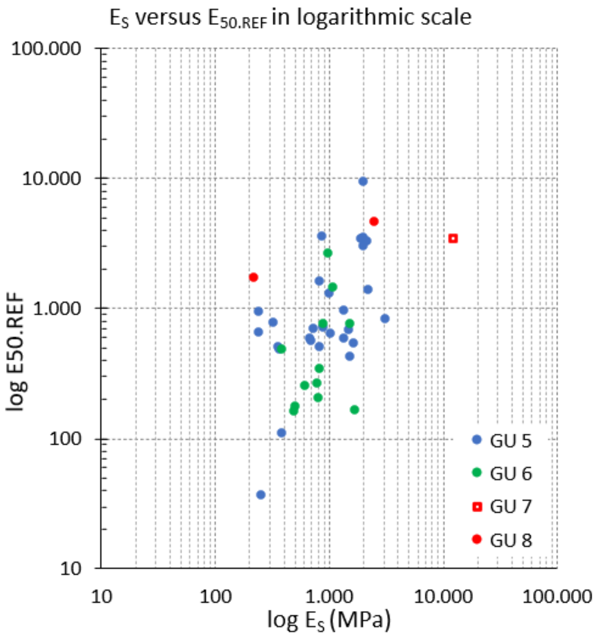

Figure 9. E_S versus $E_{50.REF}$, at GU 5, GU 6, GU 7, GU 8 (in logarithmic scale).

7. Conclusions

A methodology was presented for correlating the parameters of the Hardening Soil Model with widely used rock mass mechanical properties (e.g.: σ_{ci}, σ_{cm}, E_{rm}, GSI).

The results are presented in the form of analytical expressions that can be readily used in the analysis of geotechnical engineering problems involving excavation in rock masses. The range of applicability is limited to rock masses with GSI>10 and an U.C.S. compressive strength of the intact rock greater than 1.20 MPa.

For well-characterized soil and ground formations, where both physical and mechanical properties are well established, pressuremeter testing procedures can be applied with confidence.

A critical factor in this process is the classification of rock mass quality, based on its structural characteristics and condition.

Acknowledgements

To Michel GAMBIN.

References

Baguelin F. – Jézéquel J. F. – Shields D.H. (1978) The Pressuremeter and foundation engineering, Trans Tech publications, Germany.

Baud J.-P. & Gambin M. 2011. “Classification des sols et des roches à partir d’essais d’expansion cylindrique en haute pression”. XV ECSMGE, Athens.

Baud J.P. , Gambin M. 2013. “Détermination du coefficient rhéologique α de Ménard dans le diagramme Pressiorama®”. ISP 6, Paris.

Baud J.P. 2021. “Soil and Rock Classification from Pressuremeter Data. Recent Developments and Applications”. 6th International Conference on Geotechnical and Geophysical Site Characterization. Budapest, Hungary.

Briaud J.-L. Ménard Lecture. (2013). “The pressuremeter test: Expanding its use”. 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris.

Gambin M. 2010. “Les théories et leur évolution face à la réalité en Géotechnique”. Conférence Coulomb, Paris.

Gambin M., Rousseau J. (1988) Interpretation and application of pressuremeter test results to Foundation design, General Memorandum, sols soils No 26-1975 - Revised 1988.

Marinos P., Marinos V., Hoek E., 2007. “The Geological Strength Index (GSI): A characterization tool for assessing engineering properties of rock masses”. Underground works under special conditions. ISRM Workshop W1, Madrid, Spain. Publisher: Taylor and Francis. p.p. 13-21.

Menard, L. (1975): "The Menard Pressuremeter: Interpretation and Application of the Pressuremeter Test Results to Foundations Design", Sols-Soils, No. 26, Paris, France.

Reiffsteck Ph. 2002. “Nouvelles technologies d'essai en mecanique des sols. Etat de l'art”. Paramètres de calcul géotechniques, presses de l'ENPC / LCPC, Paris, pp. 201-243.

Ritsos A., Migiros G., Kollios A., Kolovaris E., Paris 2005. “Evaluation of Pressuremeter tests performed within the formations of Athens basin”. ISP5 – Pressio 2005, International Symposium 50 years of pressuremeter.

Ritsos A., Basdekis A., Gambin M., Paris 2013. “Pressiorama – Application of Ménard Pressuremeter in Several Geological Formations Encountered in Greece”. ISP6, 18th International Conference on Soil Mechanics and Geotechnical Engineering.

Ritsos A. “The Ménard Pressuremeter (MPM). Application in Greece”. Athens 2023. Invited speaker, 9th Hellenic Conference on Geotechnical Engineering.

Varaksin S. Vice Chairman TC211 Scientific Advisor. 2015. “The Menard Pressuremeter: history, equipment, new developments, installation procedures, design rules and methods”.

Table 3. Measured range of parameters from PMT

G.U.	Geological Units	E_M	P^*_{LM}
		MPa	MPa
	Fluvio-torrential Deposits		
2A.2	Coarse Grained Soils	11 - 137	0.8 – 6.6
	Alluvial Fan Deposits	**overall: 6 - 645**	**overall: 0.4 – 7.3**
2B.3	Claystone Siltstone	163 - 645	5.5 – 7.4
2B.4	Breccia	203 - 417	6.7 – 7.1
GU 5	**Athenian Schist, Upper Unit**	**overall: 17 – 1392**	**overall: 1.8 – 8.3 >10**
5.1	Weathering mantle	41 - 114	3.7 – 6.4
5.2	Meta-Siltstone	17 - 602	1.8 – 8.3
5.3	Meta-Sandstone	80 - 980	5.3 – 8.2
5.5	Metasiltstone and metasandstone alternations	146 – 1392	5.4 – 7.5
5.6	Schist	109 - 263	8.1 - >10.0
5.10	Limestone and schist alternations	94 - 236	8.3 - >10.0
GU 6	**Athenian Schist, Lower Unit**	**overall: 51 - 824**	**overall: 3.7 – 8.4**
6.3	Meta-Sandstone	117 - 442	4.0 – 6.5
6.4	Shale with metasandstone intercalations	53 - 185	3.7 – 8.0
6.5	Metasiltstone with shale intercalations	89 - 360	3.7 – 5.4
6.6	Metasiltstone and metasandstone alternations	51 - 824	5.0 – 8.4
GU 7	Alepovouni Unit		
7.3	Marble, Dolomite, Schist	2900 – 5540	8.1 - >10.0
GU 8	Ultrabasic rocks	overall: 100 - 1120	overall: 3.7 – 7.7 >10
8.3	Karstic listwanite	540 – 1120	7.2 – 7.7

Table 4. Calculated range of HCM parameters

G.U.	Geological Units	GSI	$E_{50.ref}$
			MPa
	Fluvio-torrential Deposits		overall: 12-79
2A.2	Coarse Grained Soils		43 - 79
	Alluvial Fan Deposits		**overall: 50-500**
2B.3	Claystone Siltstone		420
2B.4	Breccia		420 - 500
	Athenian Schist, Upper Unit	**overall: 18 - 57**	**overall: 37-10000**
5.1	Weathering mantle		37 - 130
5.2	Meta-Siltstone	19 - 33	410 - 1000
5.3	Meta-Sandstone	39 - 41	680 - 1540
5.5	Metasiltstone and metasandstone alternations	26 - 54	630 - 2300
5.6	Schist	37 - 48	1300 - 2000
5.10	Limestone and schist alternations	43 - 45	3300 - 3600
	Athenian Schist, Lower Unit	**overall: 14 - 47**	**overall: 75-2600**
6.3	Meta-Sandstone	32 - 47	1400 - 2600
6.4	Shale with metasandstone intercalations	14 - 29	160 - 260
6.5	Metasiltstone with shale intercalations	17 - 29	100 - 370
6.6	Metasiltstone and metasandstone alternations	21 - 33	480 - 760
	Alepovouni Unit	overall: 33 - 65	overall: 1330 - 11330
7.3	Marble, Dolomite, Schist	51	3450
	Ultrabasic rocks	overall: 42 - 64	overall: 1700-8800
8.3	Karstic listwanite	42-50	4600