Interpretation of Menard's Pressuremeter Tests in sandy soils through analytical and numerical methods

Interprétation des essais pressiométriques de Ménard dans les sols sableux par des méthodes analytiques et numériques

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ABSTRACT

Constitutive models play a fundamental role in numerical geotechnical analysis, enabling the consideration of complex soil and rock behavior. Within the generalized plasticity framework, the elastic strain increment is determined by employing the generalized Hooke's Law, independently of the constitutive model applied. Therefore, accurately estimating the elastic modulus is essential to enhance the predictive capabilities of any constitutive model. Among field tests, the Menard Pressuremeter Test (MPT) has been recognized for over 70 years as one of the most reliable methods for determining in-situ soil deformability and strength parameters. This paper reviews the analytical and numerical methods available for interpreting MPT results in sandy soils, with a focus on estimating the soil deposit's elastic modulus under loading and reloading conditions. It also presents the methodology for determining key strength parameters such as the internal friction angle and undrained shear strength. The study includes the interpretation of 46 MPTs performed in Mexico City, establishing correlations between the elastic modulus, NSPT blow count, and other conventional geotechnical parameters. Additionally, the influence of fine content on soil behavior is analyzed. Results show that sandy soils with fines content up to 30% continue to behave as granular materials. This finding has significant implications for assessing the long-term stability of geotechnical structures, as it directly impacts both shear strength and deformation characteristics. Finally, key recommendations are provided for interpreting MPT data in geotechnical site characterization, contributing to the development of more accurate constitutive models and enhancing the reliability of numerical simulations in geotechnical design.

RÉSUMÉ

Les modèles constitutifs jouent un rôle fondamental dans l'analyse géotechnique numérique, car ils permettent de prendre en compte le comportement complexe des sols et des roches. Dans le cadre de la plasticité généralisée, l'augmentation de la déformation élastique est déterminée en utilisant la loi de Hooke généralisée, indépendamment du modèle constitutif appliqué. Par conséquent, il est essentiel d'estimer avec précision le module d'élasticité pour améliorer les capacités prédictives de tout modèle constitutif. Parmi les essais de terrain, l'essai pressiométrique de Ménard (EPM) est reconnu depuis plus de 70 ans comme l'une des méthodes les plus fiables pour déterminer les paramètres de déformabilité et de résistance du sol in situ. Cet article passe en revue les méthodes analytiques et numériques disponibles pour l'interprétation des résultats de l'essai pressiométrique de Ménard dans les sols sableux, en mettant l'accent sur l'estimation du module d'élasticité du dépôt de sol dans des conditions de chargement et de rechargement. Elle présente également la méthodologie permettant de déterminer les paramètres de résistance clés tels que l'angle de frottement interne et la résistance au cisaillement non drainé. L'étude comprend l'interprétation de 46 TPM réalisés à Mexico, établissant des corrélations entre le module élastique, le nombre de coups de NSPT et d'autres paramètres géotechniques conventionnels. En outre, l'influence de la teneur en fines sur le comportement du sol est analysée. Les résultats montrent que les sols sableux contenant jusqu'à 30 % de fines continuent à se comporter comme des matériaux granulaires. Cette constatation a des implications significatives pour l'évaluation de la stabilité à long terme des structures géotechniques, car elle a un impact direct sur la résistance au cisaillement et les caractéristiques de déformation. Enfin, des recommandations clés sont fournies pour l'interprétation des données MPT dans la caractérisation des sites géotechniques, contribuant au développement de modèles constitutifs plus précis et améliorant la fiabilité des simulations numériques dans la conception géotechnique.

Keywords: Menard's pressuremeter tests, sandy soils, Standard Penetration Test, Geotechnical testing, Numerical modelling

1. Introduction

In geotechnical engineering, assessing soil mechanical properties is essential for efficient and safe design, especially when numerical modelling is involved in the analysis. Among these properties, the elastic modulus, the reloading elastic modulus, the friction angle and the undrained shear strength are key to predicting soil response to applied stresses; such parameters have proven to be especially useful in analyses involving saturated and unsaturated soils for different geotechnical applications, from design of foundations to slope stability. Nowadays, estimating soil properties through laboratory and in-situ testing is common. However, recovering undisturbed samples of sandy soils remains a challenge, making in-situ testing indispensable for soil characterization purposes.

The Menard Pressuremeter Test (MPT) has been recognized for seven decades as a reliable technique for in situ soil characterization. This is mainly because a stress-strain curve can be obtained from the soil to interpret elastic and strength parameters mathematically. The pioneering work of Menard in 1963 and Baguelin, Jezequel and Shields in 1978 laid the foundations of this method (Menard, 1963; Baguelin, Jezequel, & Shields, 1978). It is particularly suitable for obtaining more representative estimates of the in-situ state with fewer disturbances. It takes advantage of determining the elastic modulus using loading and unloading cycles, as demonstrated by its application in laboratory conditions (Schnaid, 2009); this is particularly relevant in projects where the accuracy of the elastic response of the soil is critical for the structural design (Robertson, 1986).

This study focuses on interpreting 46 MPT tests performed at a project site in Chapultepec Park, Mexico City. The results are compared with data from standard penetration tests (SPT) and laboratory tests, establishing correlations between elastic modulus and NSPT counts. This comparison allows for evaluating the efficiency of the MPT and provides a more accurate basis for geotechnical computational models, offering a more reliable tool for site characterization and integration of predictive models in geotechnical engineering.

1.1. Site location

The project site corresponds to a causeway that connects the two most significant sections of Chapultepec Park in Mexico City. The site is at the Sierra de las Cruces, part of the Trans-Mexican Volcanic Belt. The Sierra de las Cruces consists of different stratovolcanoes and is the western geological boundary between the basins of Mexico and Toluca. At the project site prevails a series of knolls (fan-like) made of intercalation of pyroclastic and epiclastic deposits (Arce et al., 2019).

In addition, several geological faults cross the region, predominantly normal and oblique-slip faults associated with the extensional tectonic regime of the Trans-Mexican Volcanic Belt (Arce et al., 2019; Siebe et al., 2005). These faults generate discontinuities within the subsoil, leading to zones of weakened mechanical behavior, increased permeability, and potential planes of failure (Ovando-Shelley et al., 2007; Lozano-García and Ortega-Guerrero, 1994). From a geotechnical standpoint, these fault zones contribute to anisotropy in the soil mass, irregular settlement patterns, and an elevated risk of slope instability and ground collapse (Juárez-Camarena et al., 2016; GCDMX, 2023), all of which must be carefully considered in geotechnical design and construction.

The approximate coordinates of the Project Site are Latitude: 19.42° N and Longitude: -99.19° W. The project site is shown in Figure 1.

Figure 1. Location of the project site in Mexico City

1.2. Geotechnical site conditions

The Sierra de las Cruces is primarily composed of Quaternary volcanic rocks, such as pyroclastic deposits and lava flows (Siebe et al., 2005), which have shaped the region's topography and alluvial deposit distribution. According to Lozano-García and Ortega-Guerrero (1994), the alluvial deposits in the western half of the Valley of Mexico Metropolitan Area are the result of the erosion of volcanic formations and are composed of sand, silt, and gravel carried by water currents. The composition of these deposits varies depending on the slope of the terrain, the amount of vegetation, and the amount of rainfall.

According to the geotechnical zoning of the Mexico City Building Code (GCDMX, 2023) and Juárez-Camarena et al. (2016), the project site is located inside Zone I, which is called Hills. This zone is formed by hard soils that were deposited outside the lake area, but sandy deposits in a relatively loose state can also be found. In this area, cavities and rocks, sand mines, caves, and uncontrolled landfills are common. The project site location inside the geotechnical zoning of the Mexico City Building Code (2023) is shown in Figure 2.

The soils in the area are highly heterogeneous geotechnically; volcanic ash makes up the highest layers, which give way to sandy-silty alluvial deposits with interbedded gravel layers (Ovando-Shelley et al., 2007). Although alluvial deposits can exhibit variable bearing capacity, volcanic-derived sandy soils typically offer good shear strength. In contrast, some fine-grained deposits exhibit moderate to high plasticity, which affects compressibility and settlement potential (Santoyo et al., 2005).

Most of the soils examined in this study were categorized as sandy soils, which include silty sands, sandy silts, and sandy lean clays. All these soils had an average sand content of over 45% and a fine content of over 24%. Medium-dense to very-dense circumstances were indicated by the Standard Penetration Test (SPT) N-values, which varied from 26 to 70. With an average value of 8%, the fine fraction's plasticity index was continuously below 13%, indicating low plasticity throughout all samples. Table 1 summarizes the geotechnical characteristics derived from the 46 pressuremeter experiments that were examined.

Finally, it is noteworthy to mention that despite the geotechnical zoning indicating the prevalence of hard soils, different landslide events have been recorded in the region in the last 25 years, even though it is commonly recommended to protect slopes against weathering by applying shotcrete, as illustrated in Figure 3. These events suggest that the materials behave predominantly as granular soils despite the content of fines being higher than 24 %.

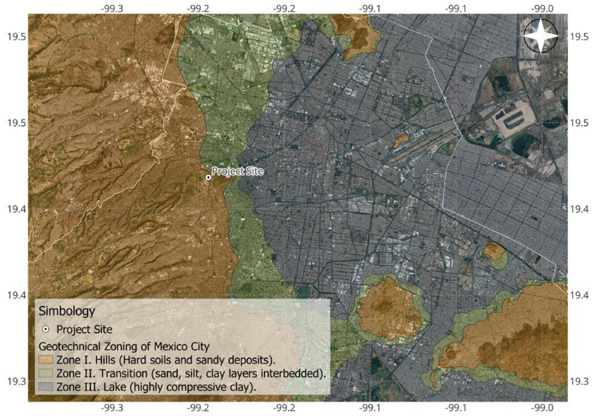

Figure 2. Location of the project site (GCDMX, 2023)

Figure 3. Santa Fe slope landslide, located near to the project site (Photo by Ximena Mejia, Excelsior, 2015)

1.3. Pressuremeter test

The Menard Pressuremeter Test (MPT) was designed by Louis Menard in 1963 (Menard, 1963; Baguelin, Jezequel & Shields, 1978) and nowadays has become an in-situ technique widely used to characterize the mechanical properties of soils and rocks. It allows the estimation of strength and deformability parameters by the interpretation of the basic variables that are obtained by the mathematical and graphical analysis from the stress-strain relationship. The basic pressuremeter variables obtained are the Pressuremeter Modulus (E_m), the limit pressure (p_L) and the pressuremeter creep pressure (p_f). Based on the pressuremeter modulus, it is possible to estimate the elastic soil modulus (E). The undrained strength (S_u) and the internal friction angle (ϕ) can be analytically obtained from the stress-strain relationship and the cavity expansion theory. These three soil parameters are commonly applied in the assessment of the soil response in solving analytically and numerically geotechnical-related problems.

The MPT is performed by applying radial pressure by means of a pressuremeter probe on the wall of a borehole (cylindrical cavity), which radially expands until it reaches the yielding of the soil. Thus, it develops a stress-strain curve that can define the initial stiffness of the soil and the evolution of plastic strains that are related to the strength of soil parameters. The pressuremeter modulus (E_m) is derived from the slope of the stress-strain curve in the pseudo-elastic range, reflecting the rigidity of the soil under loading conditions. During a MPT, it is also possible to perform unload-reload cycles, allowing the estimation of the unloading-reloading pressuremeter modulus (E_{rm}), which is especially important in geotechnical problems such as cyclic loading, excavations and tunnelling.

The limit pressure (p_L) is considered the pressure the soil can withstand before plastic failure. The undrained strength (S_u) of the soil can be estimated based on empirical relations between the p_L and the initial effective pressure of the soil (σ') or by applying equations derived from the expansion of cavity theory. The internal friction angle (ϕ) corresponding to granular soils is often determined by empirical relations that include the E_m and the p_L (Schnaid, 2009), however it can be estimated by Hughes et al. (1977) analytical formulation.

Despite the analytical formulations proposed for the interpretation of the stress-strain curve obtained from MPT, the effectiveness, accuracy, and reliability in estimating soil parameters highly depend on the borehole quality. Therefore, to ensure the quality of the pressuremeter data, the 46 pressuremeter tests analysed in this research were performed in accordance with international standards ISO 22476-4 and ASTM D4719. The tests were carried out using a Menard-type tri-cellular pressuremeter probe with a high-pressure cloth membrane of 100 bar capacity, which consists of a central water-inflated measuring cell and two gas-inflated guard cells located at each end of the measuring zone. All three cells are balanced to the same pressure, ensuring uniform cylindrical radial expansion along the measuring section. The probe is covered by a high-pressure cloth membrane that allows controlled deformation during the test. Figure 4 shows the performance of a pressuremeter test at the project site.

Figure 4. Performance of a pressuremeter test at site project

Table 1. Summary of properties and results of the 46 analysed pressuremeter tests

No.	Depth [m]	USCS	%F	IP%	Nspt	p_L [MPa]	p_L/p_f	E_m [MPa]	E_{rm} [MPa]	ϕ [°]	S_u [MPa]
MPT1	22.9	ML	63	8	46	2.21	2.33	71.5	-	41	-
MPT2	22.9	CL	65	13	48	2.21	2.34	84.3	-	-	0.338
MPT3	22.5	ML	63	7	50	2.44	2.22	94.7	-	45	-
MPT4	21.5	CL-ML	58	7	52	2.43	2.86	97.9	-	40	-
MPT5	22.5	CL	68	10	53	2.21	2.92	93.5	-	43	-
MPT6	22.9	CL	62	13	52	2.59	2.53	102.0	-	36	-
MPT7	23	CL	68	8	41	2.13	2.69	50.4	-	62	-
MPT8	13	CL	54	9	46	3.73	2.21	74.1	-	-	0.817
MPT9	21.5	ML	68	9	46	2.36	2.39	70.6	100.3	-	0.375
MPT10	14.5	ML	57	9	36	1.98	2.80	47.3	111.7	-	0.377
MPT11	21.7	ML	70	7	52	2.36	2.43	78.4	-	-	0.420
MPT12	15.4	SM	27	8	32	2.39	2.50	43.4	127.0	50	-
MPT13	19	ML	81	9	41	2.27	2.00	63.9	128.2	-	0.442
MPT14	13.9	ML	58	8	40	2.30	2.82	51.7	108.3	-	0.491
MPT15	20	ML	66	6	33	1.57	2.72	46.0	99.7	-	0.328
MPT16	14.2	SM	29	8	46	2.36	2.47	72.2	118.7	49	-
MPT17	19.5	ML	79	8	55	1.88	2.45	96.5	136.1	-	0.303
MPT18	12.7	SM	38	-	35	2.28	2.94	47.6	107.2	-	0.549
MPT19	19	ML	76	7	47	2.36	2.32	66.5	152.6	-	0.46
MPT20	19	SM	32	9	60	1.79	2.30	101.6	160.6	-	0.272
MPT21	18	ML	62	4	44	2.13	2.91	59.4	116.1	-	0.496
MPT22	28.5	ML	71	7	70	5.47	1.84	123.1	210.7	-	0.147
MPT23	14.2	SM	26	-	34	2.07	2.48	48.4	95.1	50	-
MPT24	23	CL	56	11	46	2.32	2.76	65	125.6	-	0.443
MPT25	14.2	SM	30	-	39	2.29	2.76	48.3	109.0	-	0.464
MPT26	23	ML	71	10	31	1.70	2.08	47.7	107.6	-	0.32
MPT27	17	SM	32	-	27	2.24	2.97	32.5	103.3	-	0.498
MPT28	29	SC-SM	24	6	65	4.01	2.69	107.1	158.1	55	-
MPT29	13	SM	29	-	30	2.03	2.41	36.5	91.0	-	0.495
MPT30	23.5	SM	30	9	69	3.99	2.76	128.2	214.1	50	-
MPT31	19.5	ML	74	8	43	2.34	2.29	62.6	127.9	-	0.513
MPT32	14.6	SM	28	7	50	2.34	2.77	74.6	108.6	49	-
MPT33	6.5	ML	79	7	26	2.44	2.47	42.6	96.7	-	0.594
MPT34	6.5	ML	79	9	34	2.35	2.33	42.3	98.8	-	0.555
MPT35	15	CL	68	9	59	2.46	2.00	101.7	134.7	-	0.424
MPT36	15	ML	66	7	46	1.87	1.80	70.1	111.0	-	0.347
MPT37	6.5	SM	34	12	43	2.24	2.19	47.3	97.5	-	0.473
MPT38	15	ML	62	8	49	1.88	1.78	61.5	105.2	34	-
MPT39	6.5	SM	48		46	2.01	2.40	59.7	101.1	41	-
MPT40	15	CL	66	8	53	2.01	2.11	74.5	121.9	-	0.354
MPT41	12.5	SM	25	8	38	1.51	2.23	46.9	88.2	44	-
MPT42	21	ML	45	11	70	4.30	2.89	131.7	-	-	0.831
MPT43	6.5	SM	24	7	28	1.71	2.43	23.1	67.3	-	0.44
MPT44	15	ML	66	8	51	2.32	1.91	74.2	110.6	-	0.439
MPT45	13.5	SM	43	10	45	2.67	2.47	46.8	85.2	46	-
MPT46	19.5	ML	67	7	63	3.57	1.88	99.1	174.7	-	0.85

%F: Fine content
IP%: Plasticity index
E_m: Pressuremeter Modulus (from MPT)
E_{rm}: Unloading-reloading pressuremeter modulus (MPT)
S_u: Undrained Shear Strength (from MPT)
UCSC: Unified Soil Classification System
Nspt: Blow count
p_L: Limit pressure (from MPT)
p_L/p_f: Ratio of limit pressure to creep pressure
ϕ: Friction Angle (from MPT)

2. Methodology of interpretation

The methodology used to interpret the pressuremeter test data is based on cavity expansion theory, which provides a framework for estimating fundamental geotechnical parameters such as the shear pressuremeter modulus (*G*), the undrained shear strength (S_u) for fine-grained soils, and the internal friction angle (ϕ) and dilation angle (ψ) for granular soils. This theory is built on the assumption that the soil behaves as a homogeneous, isotropic, and continuous medium, which simplifies the complex stress–strain behaviour encountered in natural soils. However, the formations investigated in this study—pyroclastic deposits—are known for their high heterogeneity, as evidenced by the variable sand content reported in Table 1. Although this suggests that the assumptions of cavity expansion theory are not strictly met, applying this framework still offers a useful and practical way to assess how the soil behaves under loading. It also helps identify whether the mechanical response is mainly governed by frictional mechanisms, typical of granular soils, or by undrained shear strength, as in fine-grained soils. Thus, while idealized, the assumption of homogeneity in the mechanical soil response is helpful for interpreting pressuremeter results in these variable volcanic soils.

2.1. Framework of Cavity Expansion Theory

The problem of expanding a cylindrical cavity has been solved under the assumption that the soil is a homogeneous, isotropic, and continuous medium, with the pressuremeter probe idealized as an infinitely long cylinder with axial symmetry, where the displacements occur radially during expansion. If it is also accepted that the soil follows Hooke's law and behaves elastically, the fundamental equilibrium equation in cylindrical coordinates is governed by Equation 1.

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \qquad (1)$$

The cavity strain, which is the equivalent circumferential strain in the cavity wall, is calculated according to Equation 2, which is applicable for small deformations.

$$\varepsilon_c = \frac{a - a_0}{a_0} \qquad (2)$$

2.2. Elastic Soil Interpretation

The fundamental relationship that describes how the applied cavity pressure relates to the shear modulus and the strain at the cavity wall is described by Equation (3), which is also valid when the cavity expands in an elastic material that follows Hooke's law.

$$p - \sigma_{h0} = 2G\varepsilon_c \qquad (3)$$

Based on Equation (3) and considering the initial horizontal stress (σ_{h0}) along with the corresponding cavity strain (ε_c), the shear modulus (*G*) can be calculated by measuring the increase in pressure relative to the increase in the radial displacement during a pressuremeter test, as expressed by Equation (4). Notably, Mair and Wood (1987) pointed out that the cavity expansion phenomenon is a shearing process instead of a compressive one; therefore, the shear modulus can be derived directly from pressuremeter data.

$$\mathrm{G} = \frac{1}{2}\frac{dp}{d\varepsilon_c} \qquad (4)$$

The Pressuremeter Modulus (E_m) was calculated as a function of the shear modulus (*G*) and the Poisson's ratio (ν), as expressed by Equation (5). A value of 0.3 was considered for Poisson's ratio (ν) to interpret the pressuremeter tests.

$$E_m = 2G(1 + \nu) \qquad (5)$$

Since a slotted tube was not used during testing, the Menard Pressuremeter Modulus (E_m) was estimated using the formulation proposed by Lamé, which is applicable within the elastic portion of the pressure–volume curve. This method accounts for the average incremental volume and the slope corresponding to the pseudo-elastic range of the test. The expression used is shown in Equation (6), and is defined as:

$$E_m = 2(1 + \nu)\left[V_0 + \left(\frac{V_B - V_A}{2}\right)\right]\frac{dp}{dV} \qquad (6)$$

Where V_0 represents the initial volume of the measuring cell, V_A and V_B are the volumes recorded at two consecutive points within the elastic range, ν is the Poisson's ratio (assumed to be 0.3), and dp/dV is the slope of the pressure–volume curve in that range. This formulation provides an operational stiffness that, although potentially affected by early plastic deformations, offers a practical estimation of the in-situ soil deformability under elastic conditions.

2.3. Undrained analysis

Under undrained conditions, such as those that prevail in fine-grained soils, the pressuremeter expansion phenomenon can be analyzed using Equation (7). This expression stems from the assumption that fine-grained soils behave as an elastic-perfectly plastic material, following the Tresca failure criterion.

$$p = \sigma_{h0} + Su\left[1 + ln\left(\frac{G}{S_u}\right) + ln\left(\frac{\Delta v}{v}\right)\right] \qquad (7)$$

The term σ_{h0} represents the in situ horizontal stress before testing, while S_u represents the undrained shear strength of the soil. The logarithmic terms account for the effect of stiffness and the volumetric strain, which captures how the cavity expands under applied pressure. This equation is crucial for estimating the undrained shear strength, a key parameter in foundation and slope stability analysis.

2.4. Drained Analysis

For drained conditions, typically observed in sandy soils, the pressure-expansion relationship is given by Equation (8), that represents the methodology developed by Hughes et al. (1977) to determine the angle of internal friction (ϕ) and the angle of dilatation (φ) from pressuremeter test data.

$$ln(p - u_0) = S\, ln(\varepsilon_c) + A \quad (8)$$

Equation (8) expresses the relationship between the effective pressure (p-u_0) and the logarithm of the cavity strain (ε_c). Constant A represents an intercept that depends on the soil properties, while S represents the slope of the expansion curve in logarithmic space.

Slope S is related to the friction angle (ϕ) and the angle of dilatation (φ) by Equation (9). This equation is derived from the assumption that the soil around the cylindrical cavity deforms under axial symmetry and plane strain and accounts for the effects of the volume changes that occur in granular soils under shearing loading.

$$S = \frac{(1 + \sin\varphi)\sin\phi'}{1 + \sin\phi'} \quad (9)$$

During the interpretation of the pressuremeter data, the empirical relationship between friction angle (ϕ) and the angle of dilatation (φ) established by Bolton (1986) was taken into consideration. This relationship relates the peak friction angle to the critical state friction angle and the dilation angle as represented by Equation (10).

$$\varphi \approx \phi' - 30° \quad (10)$$

The analytical interpretation of Menard Pressuremeter Tests (MPT), commonly based on cavity expansion theory, assumes idealized material behaviors, where soils are classified as either purely cohesive or purely frictional (Hughes et al., 1977; Mair & Wood, 1987). While these assumptions simplify the derivation of key parameters such as the undrained shear strength (Su) and the internal friction angle (ϕ), they do not capture the complex behavior of natural soils, particularly those exhibiting transitional characteristics due to variable fines content or heterogeneity (Ovando-Shelley et al., 2007; Arce et al., 2019). For fully cohesive soils, cavity expansion solutions under undrained conditions may oversimplify the stress-strain response by neglecting strain-softening and anisotropy effects (Schnaid, 2009). Conversely, for fully frictional soils like clean sands, the analytical approach may ignore critical phenomena such as particle crushing, fabric changes, and localized strain zones, which influence strength and deformation characteristics (Bolton, 1986; Santoyo et al., 2005). These limitations underscore the need for complementary methods.

To overcome these analytical constraints, numerical modelling is employed, enabling a more detailed simulation of soil behavior under complex loading conditions (Smith & Griffiths, 2015). Numerical models, such as the Mohr-Coulomb and Hardening Soil Models, allow for the incorporation of non-linear stress-strain relationships, stress-path dependency, and cyclic loading effects. In this study, numerical analyses were used to validate the parameters derived from MPT results and to improve the understanding of soil behavior where fines content, cementation, or stratification deviated from the idealized assumptions of classical cavity expansion theory.

2.5. Interpretation of K_0

The interpretation of the coefficient of earth pressure at rest (K_0) was carried out using the pressuremeter test data and Equation (11). The initial horizontal stress σ'_{h0} was estimated from the field curves by identifying the point where the pseudo-elastic phase begins, corresponding to the onset of cavity expansion. Since no groundwater was encountered during testing, total stresses were used for this estimation; therefore, the vertical stress σ'_{v0} was calculated as the product of the depth of the test and the unit weight of the soil, taken as 17 kN/m³ in all cases.

$$K_0 = \frac{\sigma'_{h0}}{\sigma'_{v0}} \quad (11)$$

To evaluate the stress history of the soil, the overconsolidation ratio (OCR) was estimated based on the K_0 values obtained from the pressuremeter tests. For this, the empirical equation proposed by Mayne and Kulhawy (1982) was used:

$$K_0 = (1 - \sin\phi')\mathrm{OCR}^{\sin\phi'} \quad (12)$$

In this case, a friction angle ϕ'=45° was assumed for soils exhibiting predominantly frictional behavior. This value was derived from the pressuremeter data (Table 1), where 35% of the tests analyzed showed mechanical behavior typical of granular soils with negligible cohesion. Additionally, the observed shear strength and stiffness ratios are consistent with frictional response at this friction angle.

The OCR, in turn, was calculated in by using Equation (12). The resulting OCR ≈ 2 confirms a lightly overconsolidated soil, consistent with the geological history of the volcanic deposits at the site.

2.6. Numerical Modelling

The strength and deformability parameters derived from the analytical equations were verified by numerical simulations using a Mohr-Coulomb model, considering whether the soils behave either as a purely cohesive or a purely granular soil to assess the soil response as a function of its fine content. By analyzing the soil's response through loading, unloading, and reloading cycles, these simulations enabled direct comparison with the measurements of the in situ pressuremeter test, indicating either the undrained shear strength or the friction angle was more suitable for describing the mechanical response of the sandy soil.

Numerical simulations were conducted using FLAC2D in an axisymmetric configuration to complement the analytical interpretation of the Menard Pressuremeter Test (MPT). The model simulated cylindrical cavity expansion under drained conditions, employing a Mohr-Coulomb constitutive model to represent soil behavior, a Poisson's ratio equal to 0.3 was assumed in all the simulations. Key soil parameters, such as internal friction angle, dilation angle, and cohesion (undrained shear strength), were assigned based on the analytical interpretation of the in-situ test results.

The simulation considered three phases loading, unloading, and reloading replicating the conditions observed in the field tests. Boundary conditions and initial stress states were applied to reflect the in-situ stress conditions.

This numerical approach allowed validation of the parameters obtained from analytical methods and provided deeper insight into complex soil responses not fully addressed by traditional cavity expansion theory (Smith & Griffiths, 2015).

The analytical equations utilized to interpret pressuremeter tests are efficient, accurate, and dependable for practical geotechnical applications, as graphically demonstrated by the close relationship between the numerical simulations and the field results in Figures 5 and 6.

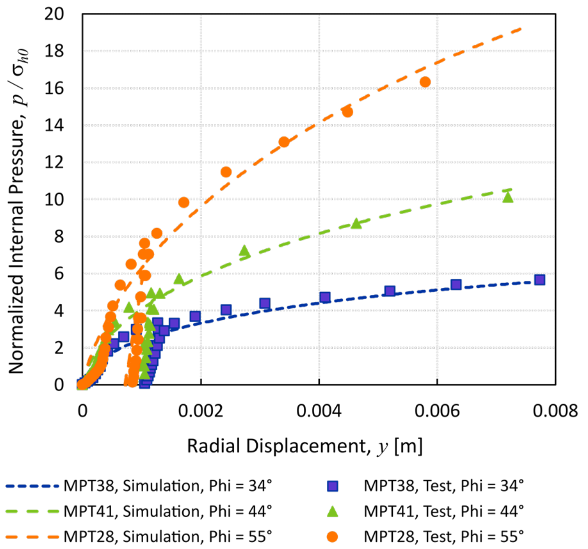

Figure 5. Comparison between performed tests and numerical simulations for materials with frictional behavior

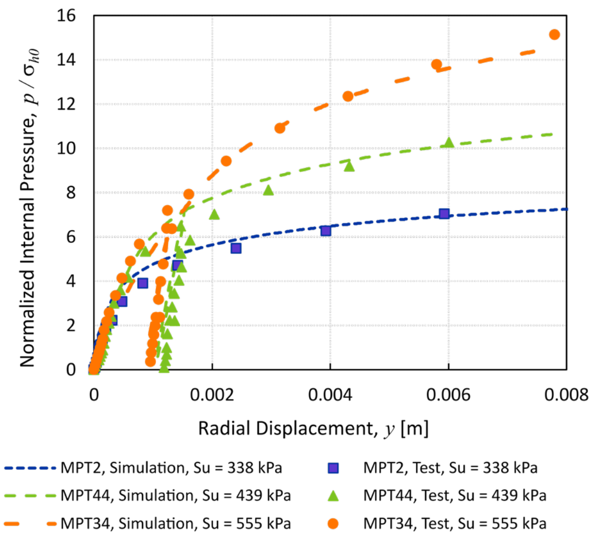

Figure 6. Comparison between performed tests and numerical simulations for those materials with undrained behavior

3. Results

The summary of results obtained from the analysis of the 46 pressuremeter tests on sandy soils in the project site is shown in Table 1, where the limit pressure (p_L), the ratio of limit pressure to creep pressure (p_L/p_f), the pressuremeter modulus (E_m), the friction angle (ϕ), and the undrained shear strength (S_u) are presented.

The results obtained from the pressuremeter data allowed to establish a correlation between the SPT-N value and the pressuremeter modulus (E_m) that is expressed by Equation (10), where the Pearson correlation coefficient is also indicated. Equation (10) was estimated by implementing a potential trend to prevent the E_m value from becoming negative for small SPT-N values. Equation (10) aims to estimate the deformability modulus of the sandy materials that prevail in the western side of Mexico City. The correlation established is graphically presented in Figure 7. It is worthy to say that the elastic modulus considered in the numerical simulations was the Pressuremeter Modulus,

$$E_m = 0.2936 * N_{SPT}{}^{1.4226} \qquad (10)$$

$$R^2 = 0.8961$$

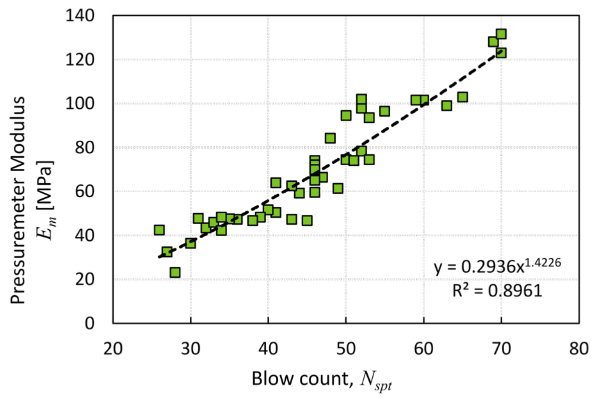

Figure 7. Correlation found between the Blow Count (Nspt) and Pressuremeter Modulus (Em).

The mechanical behavior of 35% of the 46 pressuremeter tests was described by a friction angle and cero cohesion, indicating that granular behavior prevails even though the fine content ranges from 24% to an average value of 46%. The histogram of the friction angle derived from the pressuremeter data is presented in Figure 8, showing that a representative friction angle for the sandy soils presented at the project site ranges from 40 to 49° when a frictional behavior prevails.

Figure 9 presents the histogram of the ratio of pressuremeter modulus to undrained shear strength that was obtained in those pressuremeter tests where undrained behavior prevails; an average value of 102 was obtained for the site project soils, which is common in moderately consolidated cohesive soils. The soils associated with such mechanical response have higher than 54% fine content.

On the other hand, the obtained ratio of reloading pressuremeter modulus to pressuremeter modulus is presented in Figure 10, a representative ratio value for the soil of the site project ranges from 1.5 to 2.5.

Finally, the relationship between depth and in situ horizontal stress is shown in Figure 11. This is essential to assess the soil's stress condition and determine the effective horizontal stress distribution (σ_{h0}), based on which it is possible to compute of the Over Consolidation Ratio (OCR), which offers details on the soil's stress history and reaction to new loads, depends on the precise estimate of σ_{h0}. Assuming an average volumetric weight of the soil equal to 17 kN/m³, and considering a depth of 25 m, it is possible to estimate the average effective horizontal stress equal to 0.2 MPa. If a friction angle of 45° is also considered, the OCR might be estimated to be equal to 2, indicating a lightly overconsolidated soil and the K_0 equal to 0.47.

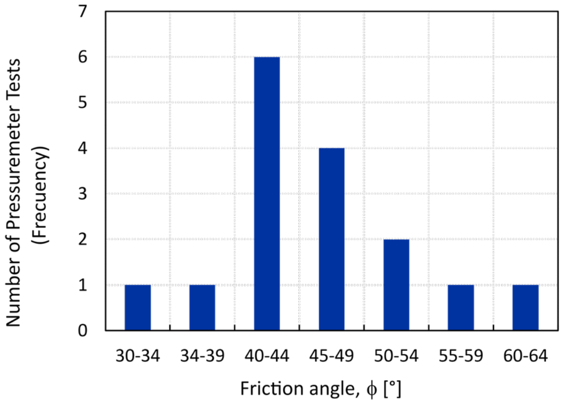

Figure 8. Histogram of the interpreted friction angle

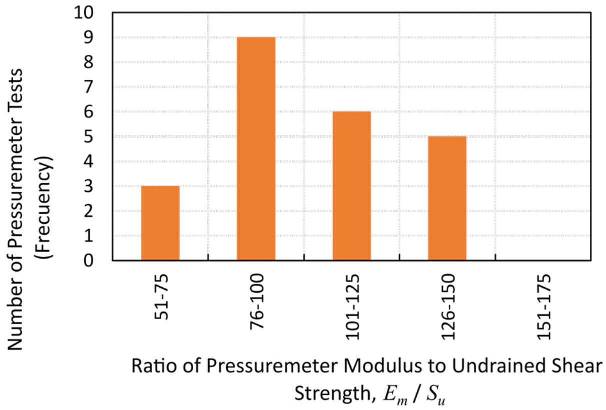

Figure 9. Histogram of ratio of pressuremeter test Modulus to Undrained Shear Strength

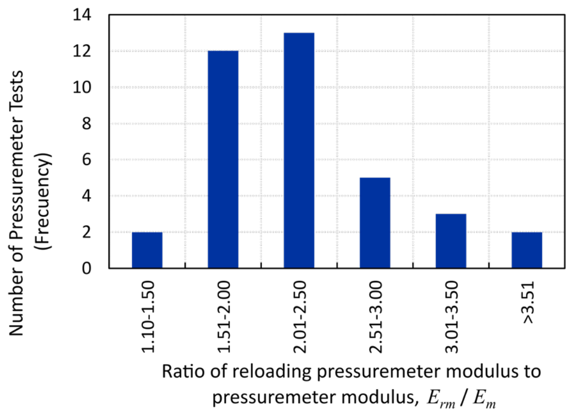

Figure 10. Histogram of ratio of reloading pressuremeter modulus to pressuremeter modulus

4. Conclusions

Based on the results of the 46 pressuremeter test analysis, the following conclusions are presented:

- The results obtained from the pressuremeter data allowed to establish a correlation between the SPT-N value and the pressuremeter modulus (E_m) that is expressed by $E_m = 0.2936 * N_{SPT}{}^{1.4226}$.
- The sandy materials of the site with fine content below 45% mostly behave like purely frictional soils, where the angle of internal friction primarily controls the shear strength. Therefore, it is advised that a friction angle near 45° be considered in geotechnical designs where long-term behaviour is crucial, such as in deep excavation and slope stability studies.
- The sandy materials of the site with fine content above 54% mostly behave like purely cohesive soils, where the undrained shear strenght primarily controls the shear strength. The average ratio of pressuremeter modulus to undrained shear strength that was obtained is 102, which is common in moderately consolidated cohesive soils. Therefore, it is advised that this parameter be considered in geotechnical designs where short-term behaviour is crucial, such as shallow and deep foundations.
- The average ratio of limit pressure to creep pressure is equal to 2.4; and the ratio of reloading pressuremeter modulus to pressuremeter modulus is equal to 2.3.
- The OCR estimated based on the in-situ horizontal effective stress obtained from the pressuremeter tests is equal to 2, indicating a lightly overconsolidated soil.
- The estimated average K_0 is equal to 0.47 based on the pressuremeter tests
- Despite the formations investigated in this study—pyroclastic deposits—being known for their high heterogeneity, the assumption of homogeneity in the mechanical soil response remains applicable.

Overall, the site's soils exhibit a distinct behavioral shift: over a particular fines content limits, the material shifts from cohesive to frictional behavior, with the latter being more prevalent in low-fines content sands. This distinction is essential for choosing the right parameters based on the type of soil and geotechnical study.

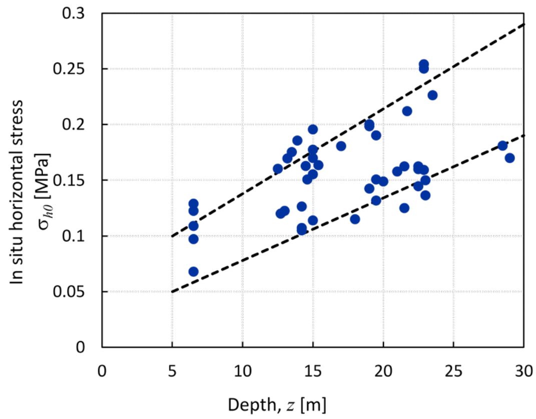

Figure 11. In situ horizontal stress versus depth relationship obtained from the pressuremeter tests

References

Arce, J. L., Layer, P. W., Macías, J. L., Morales-Casique, E., García-Palomo, A., Jiménez-Domínguez, F. J., Vásquez-Serrano, A. 2019. "Geology and stratigraphy of the Mexico Basin (Mexico City), central Trans-Mexican Volcanic Belt", J.

Maps, 15(2), pp. 320-332, 2019. https://doi.org/10.1080/17445647.2019.1593251

ASTM. "D4719-87, Standard Test Methods for Prebored Pressuremeter Testing in Soils", ASTM International, West Conshohocken, PA, United States, 1994.

Baguelin, F., Jezequel, J., Shields, D. H. "The Pressuremeter and Foundation Engineering", Trans Tech Publications, Clausthal-Zellerfeld, Germany, 1978. https://doi.org/10.1139/t79-069

Bolton, M. D. 1986. "Strength and dilatancy of sands", Géotechnique, 36(1), pp. 65-78, 1986. http://doi.org/10.1680/geot.1986.36.1.65

Excélsior, "Documentaron los riesgos de deslaves en Santa Fe" (in Spanish), [online] Available at: www.excelsior.com.mx/comunidad/2015/11/06/1055466, accessed: 18/03/2025.

Gobierno de la Ciudad de México (GCDMX). "Reglamento de construcciones para el Distrito Federal", Gaceta Oficial de la Ciudad de México, Mexico City, Mexico, 2023.

Hughes, J. M. O., Wroth, C. P., Windle, D. 1977. "Pressuremeter tests in sands", Géotechnique, 27(4), pp. 455-477.

ISO. "22476-4:2012, Geotechnical investigation and testing – Field testing – Part 4: Ménard pressuremeter test", International Organization for Standardization, Geneva, Switzerland, 2012.

Itasca Consulting Group, Inc. "FLAC2D, (Version Demo 9.2.18)", Available at: https://www.itascacg.com/software/flac2d, accessed: 18/March/2025.

Lozano-García, S., Ortega-Guerrero, B. 1994. "Late Quaternary environmental changes of the central part of the Basin of Mexico: Correlation between Texcoco and Chalco basins", Rev. Palaeobot. Palynol., 80(1), pp. 35-50. https://doi.org/10.1016/S0034-6667(97)00046-8

Mair, R. J., Wood, D. M. "Pressuremeter Testing: Methods and Interpretations", 1st ed., CIRIA Butterworths, London, United Kingdom, 1987.

Menard, L. 1963. "The Menard Pressuremeter and its Applications", Sol. Soils Found., 5(2), pp. 45-61.

Ovando-Shelley, E., Orozco, R., Santoyo, E. 2007. "Geotechnical properties of lacustrine soils in the Valley of Mexico", Soils Found., 47(5), pp. 901-914.

Ovando-Shelley, E., Romo, M. P., Santoyo, E. 2013. "Geotechnical characteristics of the subsoil of Mexico City", J. Geotech. Geoenviron. Eng., 139(9), pp. 1515-1529.

Robertson, P. K. 1986. "In-Situ Testing for Site Characterization", In: Proceedings of the In-Situ Testing Symposium, ASCE, New York, United States, 1986, pp. 112-123.

Santoyo, E., García-Flores, L. F., Santoyo, S. 2005. "Evaluation of the geotechnical behavior of volcanic soils from the Basin of Mexico", Eng. Geol., 78(3-4), pp. 269-286.

Sousa Coutinho, R. M. (2017). Characterization of Materials Behaviour by the Pressuremeter Test (master's thesis). Instituto Superior Técnico, Universidade de Lisboa.

Schnaid, F. "In Situ Testing in Geomechanics: The Main Tests", 1st ed., Taylor & Francis Group, London, United Kingdom, 2009. https://doi.org/10.1201/9781482266054

Siebe, C., Rodríguez-Lara, V., Schaaf, P., Abouchami, W. 2005. "Geochemistry and Sr-Nd-Pb isotopic composition of recent (< 200 years old) scoriae from the Valley of Mexico: Insights into the sources of Quaternary arc magmas", Geofís. Int., 44(3), pp. 259-282.

Smith, I. M., Griffiths, D. V. "Programming the Finite Element Method", 5th ed., Wiley, Chichester, United Kingdom, 2015. https://doi.org/10.1002/9781119189237