The use of Artificial Intelligence, Finite Element Modelling and Pressuremeter tests for geotechnical characterisation.

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ABSTRACT

Pressuremeter tests have been used for many years but the difficulty for interpreting its results has hindered its wide use. Nowadays, with the rise of fast computers and readily available finite element softwares the problem of interpretation can be overcome. In this manner, the test can be modelled and the soil parameters can be determined by matching the field curve with the finite element model. In this way it is possible to characterise the soil and calibrate a constitutive model in a single process. Even though the procedure appears straightforward, in practice a manual calibration can be time consuming and tedious. In order to expedite this process and obtain a close match, a novel and powerful geotechnical software (DAARWIN) that uses Artificial Intelligence is used. Hence, the calibrated constitutive model can be used to predict many geotechnical problems. The proposed procedure is validated by comparing the modelled deformations against the real deformations measured during construction of an 8.6m deep soldier pile wall, a four- and five-storey building. The constitutive model used is the Hardening Soil Small Strain model and the structures are modelled in PLAXIS 2D and 3D. The error in most cases does not exceed 1mm.

RESUME

Les essais pressiométriques sont utilisés depuis de nombreuses années, mais la difficulté d'interprétation de leurs résultats a freiné leur large adoption. Aujourd'hui, avec l'essor des ordinateurs rapides et des logiciels d'éléments finis facilement accessibles, le problème d'interprétation peut être surmonté. Ainsi, l'essai peut être modélisé et les paramètres du sol peuvent être déterminés en faisant correspondre la courbe de terrain avec le modèle d'éléments finis. De cette manière, il est possible de caractériser le sol et de calibrer un modèle constitutif en un seul processus. Bien que la procédure semble simple, en pratique, une calibration manuelle peut s'avérer fastidieuse et chronophage. Afin d'accélérer ce processus et d'obtenir une correspondance plus précise, un logiciel géotechnique innovant et puissant (DAARWIN), utilisant l'intelligence artificielle, est employé. Ainsi, le modèle constitutif calibré peut être utilisé pour prédire de nombreux problèmes géotechniques. La procédure proposée est validée en comparant les déformations modélisées aux déformations réelles mesurées lors de la construction d'un mur de soutènement en pieux sécants de 8,6 m de profondeur, ainsi que d'un bâtiment de quatre et cinq étages. Le modèle constitutif utilisé est le Hardening Soil Small Strain model, et les structures sont modélisées dans PLAXIS 2D et 3D. Dans la plupart des cas, l'erreur ne dépasse pas 1 mm.

Keywords: artificial intelligence; finite elements; pressuremeter

1. Introduction

A novel method for interpreting high resolution pressuremeters by means of modelling the test with a finite element software (Plaxis) and iteratively back-analysing the soil parameters so as to achieve a minimum difference between the measured and modelled curve with a software that uses Artificial Intelligence (DAARWIN) will be presented in this article. A detailed introduction and context related to the historical developments that have led to this work are given in a companion paper submitted for this Conference (Martinez et al. 2025). A broader explanation related to how DAARWIN works is also given therein. Although DAARWIN was mainly developed for back-analysing monitoring data of geotechnical structures this work will present its application for interpreting pressuremeter data.

In order to assess the applicability of performing a geotechnical characterisation by means of modelling a high resolution pressuremeter curve 3 sites that were developed with different structures were monitored during its construction. The first site consisted of an 8.6m deep soldier pile retaining wall, the second a 4-storey high residential building and the third a 5-storey high residential building. Each site was characterised with either 3 or 4 high resolution pressuremeter test at each representative horizon. The construction process was then modelled in either Plaxis 2D or 3D and the soil properties for each horizon were defined by the referred procedure. It will be shown that the Plaxis finite element model calibrated with the referred procedure matches the monitoring with a maximum error of approximately 1mm, hence demonstrating that the AI-aided FEM pressuremeter interpretation delivers an accurate soil characterisation.

2. Site Descriptions

2.1. Soldier pile wall

The site and monitoring results are described in detail by Jara (2023). The site is located in Concepción, Chile and it is 42m x 43m in plan. It consists of mostly silty sands with one thin silt layer between 10.5m and 11.5m and a thicker silt layer between 14m and 19m. Two CPTu boreholes were pushed. The water table is located at a depth of 5m. One additional borehole was drilled for executing 3 high resolution pressuremeter tests. The cavity was pre-drilled and the 47mm diameter instrument lowered into the hole. Sometimes the pocket did not hold stable and it was necessary to gentle push the instrument into the collapsed ground in order to place it at the desired depth. It has been shown (Hughes & Whittle, 2022) that this insertion disturbance can be dealt with by imposing sufficient deformation during loading and by the unloading of the instrument. The tests were conducted at depths of 2.35m, 5.35m and 10.45m and are shown in figure 2.

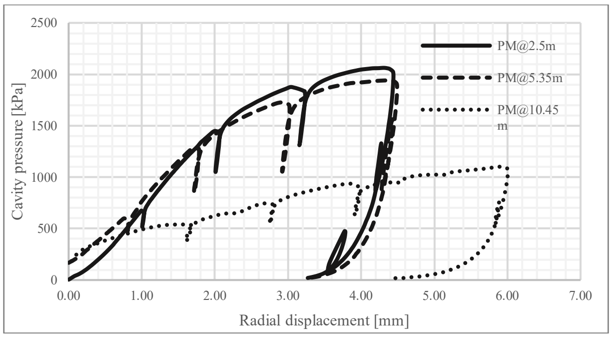

Figure 1. PM Tests at 2.35m, 5.35m and 10.45m.

The soldier pile wall consists of H beams W310x38.7 kg/m with a total length of 10.5m, spaced at 1.6m with 2 rows of grouted anchors 127mm in diameter with 4.5m free length and 13.5m and 10.5m bonded length. The timber lagging consisted of dry pine 75mm thick, see figure 2.

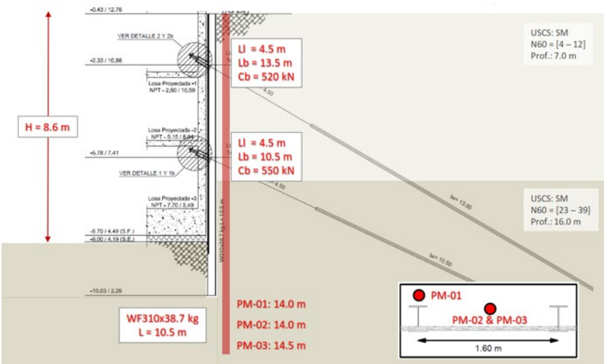

Figure 2. Schematic view of soldier pile wall.

Three inclinometers were installed to a depth of 14m away from the edges to avoid corner effects. A site photograph is shown in figure 3.

Figure 3. Site view.

2.2. Four-storey building

The site and monitoring results are described in detail by Mella (2022). The site is located in Chiguayante, Chile and the footprint size of the building is 26m x 15m. The foundations are spread footings. The structure is founded on a 2m thick compacted sandy fill followed by the natural silty sands. One CPTu borehole was pushed. The water table is located at a depth of 7m. One borehole was drilled for executing 3 high resolution pressuremeter tests, one in the fill at 1.7m and the others in the silty sand at 3.9m and 6m depth (See figure 4).

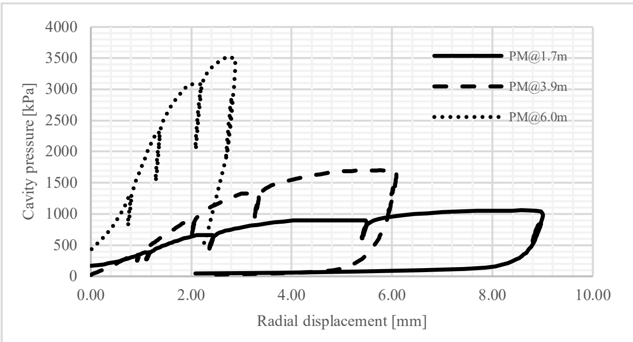

Figure 4. PM Tests at 1.7m, 3.9m and 6.0m.

2.3. Five-storey building

The site is located in San Pedro de la Paz, Chile and the footprint building size is 52m x 32m (Mella, 2022). The foundation type corresponds to a slab foundation. The structure is founded on a 5m deep compacted sandy fill overlying the natural soil which correspond to silty sands. Two CPTu boreholes were pushed. The water table is located at a depth of 2m. One borehole was drilled for executing 4 high resolution pressuremeter tests at 2m, 10.5m, 11m and 14.6m, see figure 5, in which the 2m PM is not shown due to reading errors experienced in that test.

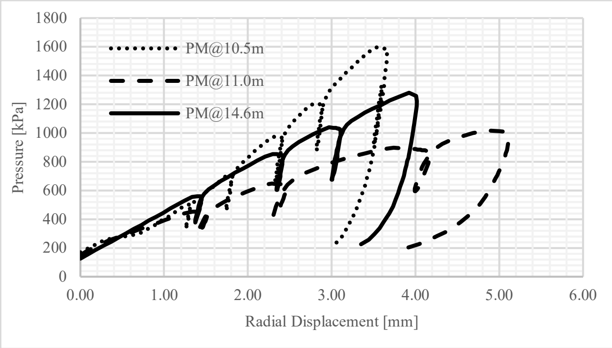

Figure 5. PM Tests at 10.5m, 11.0m and 14.6m.

3. Plaxis Modelling

3.1. Soldier pile wall

A 60m long x 25m deep mesh was used in a plane-strain model in Plaxis 2D to model the wall. The H beam was modelled as a plate element reducing its stiffness in accordance to its spacing. Node to node anchor elements were used to model the anchor free length with an elastic stiffness of 3.5x106 kN/m as embedded beam elements were used to model the anchor grouted length with the following properties:

E (kN/m2) = 2.35E+7
Tskin (kN/m) = 136

The soil horizons were modelled as follows:
H1: From 0 to 7.0m
H2: From 7.0m to 10.5m
H3: From 10.5 to 11.5m
H4: From 11.5 to mesh bottom.

Before running DAARWIN's back-analysis sensitivity analyses were carried out in Plaxis varying the angle of shearing resistance of the different strata to very low and very high values. It was observed that even with as low as 28° no considerable amounts of failure points were obtained behind the wall. It was concluded that the given soldier pile wall scheme was mainly controlled by the stiffness properties rather than the soil strength, although in the constitutive model used stiffness and strength are dependent. Notwithstanding, it was decided to leave the angle of shearing resistance out from DAARWIN's iteratives determination process and fix them at expected values defined by the CPTu results. This a priori definition helps the algorithm focusing the search on the parameters that really control the problem and avoiding multiple optima that are sometimes not physically relatable. This is especially true in the case of the Hardening Soil Small Strain (HSS) model as the stiffness also depends from the angle of shearing resistance so one can obtain the same deformation by combining either a low stiffness with a high strength or a high stiffness with a low strength. The angles of shearing resistance fixed for the DAARWIN model are:

H1: $\phi' = 40°$
H2: $\phi' = 41°$
H3: $\phi' = 28°$
H4: $\phi' = 41°$

The lateral coefficient of at rest pressure K_0 was also fixed at 1.0 for all horizons, as the model used is isotropic and there was no reliable information to use a different value.

The pressuremeters were modelled with a 30m long x 15m deep mesh using an axisymmetric model in Plaxis 2D (figure 6). Simulating the cavity pressure, a uniformly distributed pressure was applied at the test depth at a length of 0.255m which corresponds to the membrane length.

Figure 6. Pressuremeter test model.

In contrast to the wall model, before running DAARWIN, sensitivity analyses showed the dependence on the angle of shearing resistance, hence, in this case, this was a parameter to be determined rather than to be fixed. Hence, for DAARWIN's back analyses, although a high resolution pressuremeter is capable of measuring the shear modulus at small strains G_0 (Jara, 2023), it was preferred to fix this value to avoid the same stiffness-strength dependence problem mentioned above. In this manner, the G_0 values were fixed at the values determined by the downhole carried out during the CPTu tests.

For both type of analyses, wall and pressuremeter, the relationship recommended by Plaxis was adopted:

E50 = 1.25 x Eeod

The relationship between E50 and Eur was left free as it will be seen below.

After having back-analysed the pressuremeter tests a wall model was run using these soil parameters. As it can be seen two pressuremeter test were carried out in horizon 1 (PM@2.35m and PM@5.35m) and the third one in horizon 3 (PM@10.45m). As no pressuremeter tests were available for horizons 2 and 4 the same soil parameters obtained with the wall back-analysis were used for these horizons.

3.2. Four-storey building

A 130m x 70m x 40m deep mesh was used to model the building in Plaxis 3D (figure 7). The spread footing was modelled as a plate element. The transmission of loads to the foundation is through the walls, therefore, the load is modelled linearly distributed along the foundation with a value of q = 17.1 kN/m/storey.

The soil horizons were modelled as follows:

H1: From 0 to 2.0m
H2: From 2.0m to 4.3m
H3: From 4.3 to mesh bottom (40m).

The different soil horizons were characterised by the AI-aided FEM back-analysis of the pressuremeter tests, as described in the previous section.

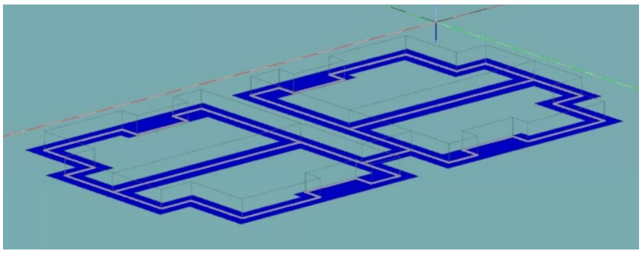

Figure 7. Four-storey building model.

3.3. Five-storey building

A 165m x 160m x 50m deep mesh was used to model the building in Plaxis 3D (figure 8). The slab foundation was modeled as a plate element. As in the previous model, the surcharge is modelled as a linear load with a value of q = 24.4 kN/m/storey.

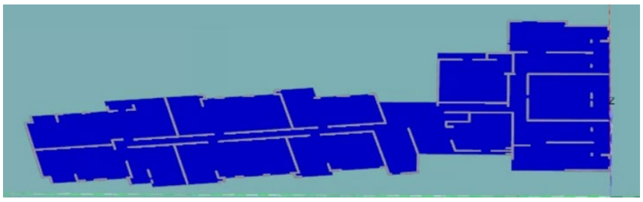

Figure 8. Five-storey building model.

The soil horizons were modelled as follows:

H1: From 0 to 5.0m (PM@1.7m in 4-storey building)
H2: From 5.0m to 9.0m (PM@10.5m)
H3: From 9.0m to 11.0m (PM@11m)
H4: From 11.0 to mesh bottom (PM@14m)

The different soil horizons were characterised by the AI-aided FEM back-analysis of the pressuremeter tests, as described in the previous section. As the 2m PM presented reading problems, for the first horizon the results from the PM at 1.7m from the 4-storey building were used. In this case both horizons correspond to a compacted sandy fill with similar specifications.

4. Results

4.1. Soldier pile wall

Table 1 to 4 show the back-analysed parameters for the wall and the pressuremeter for each horizon. The cells in grey denote the parameters have been left free to be determined by DAARWIN. The cells in white denote the parameters have been fixed to the specified value. The reference pressure for all horizons is σ'3 at the centre of each horizon.

Table 1. Horizon 1 (0m to 7m). Back-analysed parameters.

Parameter	Wall BA	PM BA@2.35m	PM BA@5.35
G0 (MPa)	90	90	90
Eur (MPa)	100	76	39
E50 (MPa)	23	38	14
ϕ ' (°)	40	42	42
$\gamma_{0.7}$	8.65E-04	1.90E-04	1.50E-04
K_0	1.0	4.125	1.2

Table 2. Horizon 2 (7.0m to 10.5m). Back-analysed parameters.

Parameter	Wall BA	PM BA
G0 (MPa)	145	-
Eur (MPa)	56	-
E50 (MPa)	6,5	-
ϕ' (°)	41	-
$\gamma_{0.7}$	2,80E-04	-
K_0	1,0	-

Table 3. Horizon 3 (10.50m to 11.5m). Back-analysed parameters.

Parameter	Wall BA	PM BA@10.45m
G0 (MPa)	85	85,0
Eur (MPa)	61,5	45,0
E50 (MPa)	6,5	8,0
ϕ' (°)	28	30,0
$\gamma_{0.7}$	7,30E-04	5,8E-04
K_0	1,0	1,15

Table 4. Horizon 4 (11.5m to bottom). Back-analysed parameters.

Parameter	Wall BA	PM BA@10.45m
G0 (MPa)	300	-
Eur (MPa)	123.3	-
E50 (MPa)	20	-
ϕ' (°)	41	-
$\gamma_{0.7}$	3.70E-04	-
K_0	1.0	-

Figures 9, 10 and 11 present the back-analyses results for pressuremeters PM@2.5m, PM@5.35m and PM@10.45m.

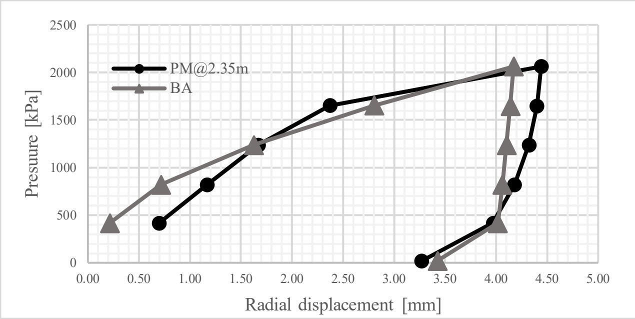

Figure 9. Back-analysed PM@2.35m

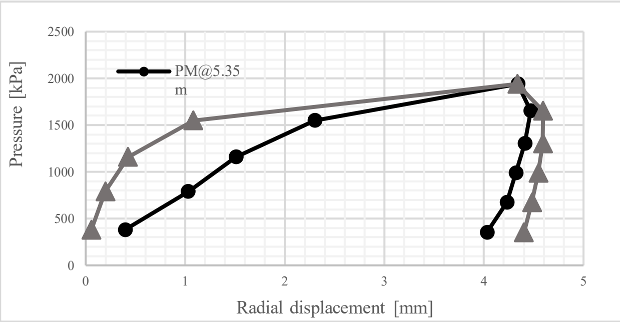

Figure 10. Back-analysed PM@5.35m

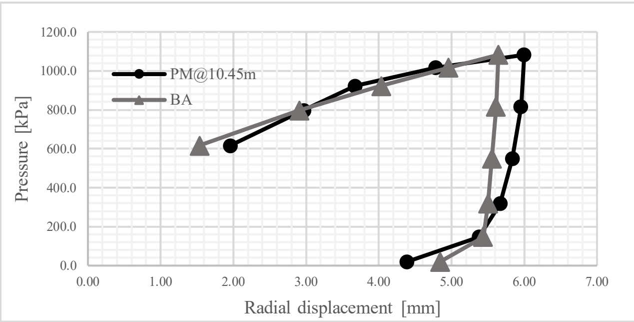

Figure 11. Back-analysed PM@10.45m

As constitutive models are calibrated as a whole it can be misleading to compare percentage differences between each parameter. Instead of comparing for each horizon the difference between each parameter for the wall and pressuremeter back analyses it is more fruitful to compare the horizontal displacements obtained for each model. Figure 12 present the horizontal displacements measured by the most representative inclinometer and the different wall models. The results obtained by Jara (2023) with the same data is also shown. In that work the author carried out back-analyses for each pressuremeter test adjusting the parameters manually in Plaxis. According to the author a period of 3 weeks was needed to obtain the results. For this work, a period of 2 working days -16 hours – was recorded. The time saved by DAARWIN is salient and the improvement in prediction is approximately threefold. In Figure 12 it can be seen that the maximum prediction error by back-analysing the pressuremeters is approximately 1mm, a notable feat for geotechnical deformation predictions. As there were no pressuremeter test conducted at horizons H2 and H4 the soil parameters for these horizons are taken from the Wall BA.

It is important to highlight that although the Wall BA curve represents a better fit of the actual wall movements, this curve corresponds to the best fit that a machine learning algorithm can obtain using Plaxis after having been fed with the actual soil movements, i.e., this curve does not represent a prediction. In contrast, the PM BA curves could have been obtained before the actual wall construction had taken place.

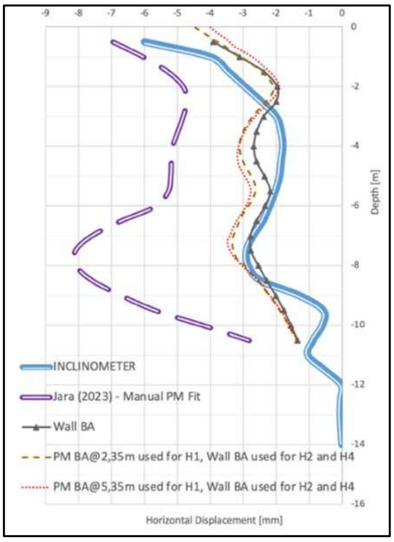

Figure 12. Horizontal displacements. Wall BA for H2 and H4, PM BA for H1 and H3.

4.2. Four-storey building

Table 5 show the back-analysed parameters for the pressuremeters for each horizon. The cells in white denote the parameters have been left free to be determined by DAARWIN. The cells in grey denote the parameters have been fixed to the specified value. The reference pressure for all horizons is σ'_3 at the centre of each horizon.

Table 5. Horizons. Back-analysed parameters.

Parameter	PM@1.7m	PM@3.9m	PM@6.0m
G0 (MPa)	72.8	106.9	216.1
Eur (MPa)	61.0	53.0	156.0
E50 (MPa)	4.0	18.0	78.0
$\gamma_{0.7}$	6.2E-4	4.0E-5	4.0E-5
ϕ' (°)	37	34	41
K_0	4.80	3.88	4.43

Figure 13 presents the back-analysed pressuremeter curve performed by DAARWIN. For conciseness, only 1 curve will be shown here.

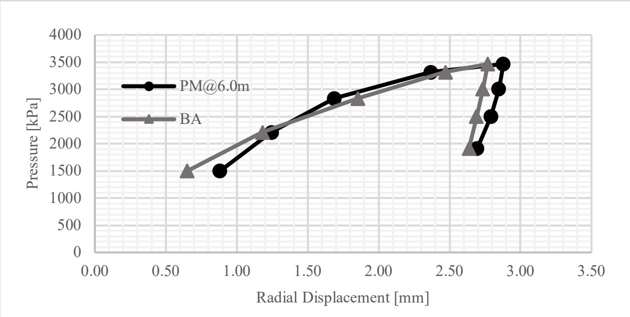

Figure 13. Back-analysed PM at 6.0m.

Figure 14 compares the Plaxis 3D computed accumulated settlements against the measured ones for each loading phase and for a representative foundation point. As it can be seen the prediction is very good, with an error of approximately 0.5mm.

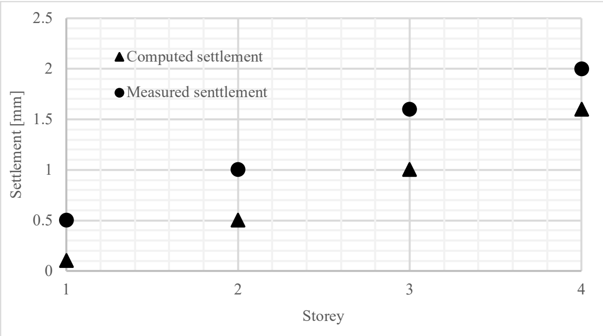

Figure 14. Computed vs measured cumulative settlement.

4.3. Five-storey building

Table 6 show the back-analysed parameters for the pressuremeters for each horizon. The cells in white denote the parameters have been left free to be determined by DAARWIN. The cells in grey denote the parameters have been fixed to the specified value. The reference pressure for all horizons is σ'$_3$ at the centre of each horizon.

Table 6. Horizons. Back-analysed parameters.

Parameter	PM@10.5m	PM@11.0m	PM@14.6m
G0 (MPa)	332.9	264.9	333.6
Eur (MPa)	60.0	88.0	14.0
E50 (MPa)	28.0	23.0	51.0
$\gamma_{0.7}$	1.0E-6	7.0E-6	2.0E-6
ϕ' (°)	37	34	39
K_0	2.65	1.03	2.03

Figure 15 presents the back-analysed pressuremeter curve performed by DAARWIN. For conciseness, only 1 curve will be shown here.

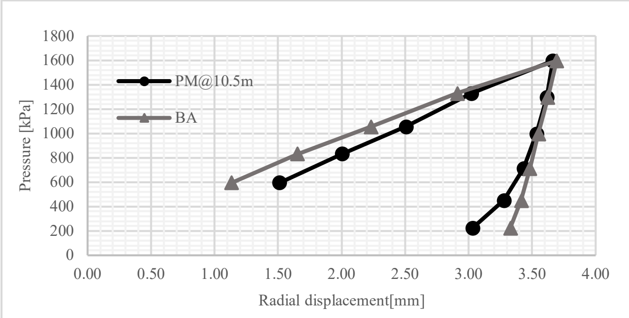

Figure 15. Back-analysed PM at 10.5m.

Figure 16 compares the Plaxis 3D computed accumulated settlements against the measured ones for each loading phase and for a representative foundation point. As it can be seen the prediction is remarkable, with a maximum error of about 1mm.

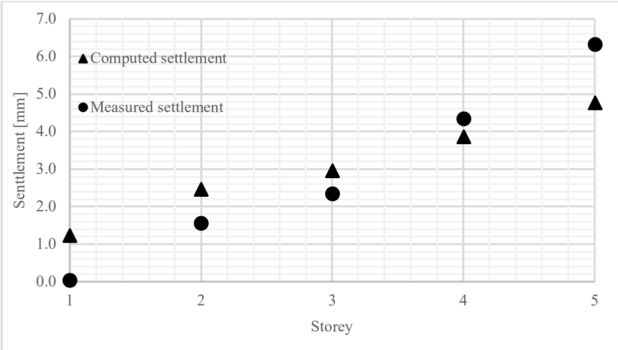

Figure 16. Computed vs measured cumulative settlement.

4.4. General results

An interesting result obtained with the proposed procedure is the determination of Ko. Values ranging from 4 to close to 1 have been obtained with increasing depth. A companion paper (Martinez et al. 2025) describes in detail a procedure for determining K_0 and the state parameter (ψ_0) (Been & Jefferies, 1985) using finite elements and artificial intelligence and validates the method against calibration chamber results. In the same manner, additionally to the K_0 values already presented above, the state parameter for all pressuremeter tests is calculated and compared against the CPTu-determined values. The ψ_0 are calculated with the same methodology described by Martinez et. Al (2025) using the CASM (Yu, 1995, 1998) constitutive model. The results are presented in figures 17, 18 and 19. The comparison coincides quite well and it cannot be discerned which test represents real soil conditions best, as boreholes and CPTus were drilled in slightly different locations. Notwithstanding, it is clear that this method offers a valid way forward for calculating the state parameter which, very importantly, is not based on correlations or empiricism but sound fundamental considerations.

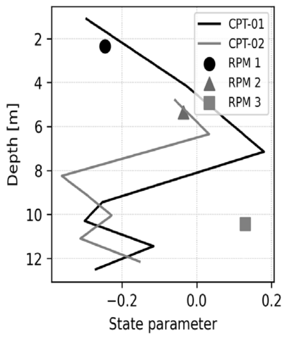

Figure 17. CPTU- and PM- ψ_0 derived values. Retaining wall site.

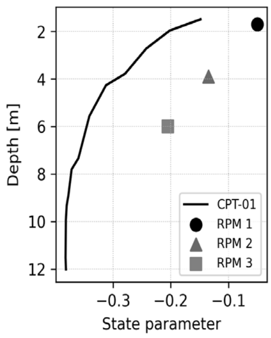

Figure 18. CPTU- and PM- ψ_0 derived values. Four-storey building site.

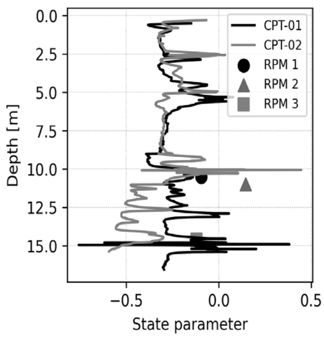

Figure 19. CPTU and PM ψ_0 derived values. Five-storey building site.

A general Hughes et al (1977) loading analysis for sand would give much lower ϕ' values than the ones reported in this article and very different to the ones obtained by the CPTu. For example, for H-1 in the case of the soldier pile wall a Hughes et al (1977) analysis gives ϕ' = 32° in contrast with ϕ' = 42° given by the pressuremeter-FEM analysis and ϕ' = 40° given by the CPTu. The reason for this huge difference is because the analytical method ignores the elastic deformations in the plastic deforming region and it was shown by Yu (1990) that neglecting elastic strains in the plastic zone tends to give a softer pressuremeter response and therefore underestimates the measured angle of friction. This is of great importance as it shows that adopting more realistic soil models when interpreting pressuremeters can have enormous consequences for the values of the soil parameters obtained.

The angle of shearing resistance values obtained by the pressuremeter closely match the CPT defined values. CPT-derived ϕ' values for a silty sand can be considered reliable in this case as the calibration chambers in which CPT- ϕ' values are determined closely resemble the silty sand deposit conditions - no boundary effects, homogeneous grain size, regular geology, etc. Notwithstanding, with the proposed approach the ϕ'-pressuremeter values are not determined by correlations, they are determined by strict fundamental soil mechanics principles. This would imply that the approach can be applied to more complex materials such as residual soils, highly weathered rock, gravels, or even materials that would hardly classify as soil as the salty rocks encountered for mining the nowadays in fashion Lithium. The authors have tested these materials with pressuremeters and have obtained insightful results which will be discussed elsewhere.

These are all materials in which unaltered samples are hard to obtain and transport into a laboratory to test in a triaxial test, for example. Even if the materials were to be sampled in a disturbed fashion, accepting the loss of natural structure, their maximum particle size to sample size ratio would normally imply the use of uncommercial laboratory apparatuses - recent research suggests this ratio is much larger than the factor of 10 normally used and can reach a factor as high as 20 (Cantor & Ovalle, 2023). As pointed out by Hughes and Whittle (2022), the pressuremeter mobilises a much larger volume than traditional laboratory tests - 50 times more than a commercial triaxial probe, hence, solving this issue for an important range of materials.

5. Conclusions

In order to assess the applicability of performing a geotechnical characterisation by means of numerically modelling a high resolution pressuremeter curve, 3 sites that were developed with different structures were monitored during its construction. The first site consisted of an 8.6m deep soldier pile retaining wall, the second a 4-storey high residential building and the third a 5-storey high residential building. Each site was characterised with either 3 or 4 high resolution pressuremeter test at each representative horizon. The construction process was modelled in either Plaxis 2D or 3D and the soil properties for each horizon were defined by back-analysing the pressuremeter curve with an AI software called DAARWIN that uses PLAXIS. It was shown that the Plaxis finite element model calibrated with the referred procedure matched the monitoring with a maximum error of approximately 1mm, hence demonstrating that the pressuremeter-derived soil properties accurately represent the ground behaviour and improves the soil characterisation that would have been obtained by traditional pressuremeter interpretation methods.

Horizontal deformations were predicted in the case of the soldier pile wall and vertical deformations were predicted in both buildings' cases, hence predicting 2 different stress paths with the radial horizontal stress path measured by the pressuremeter. This would also confirm the wide field of applications appropriate for pressuremeters and reduce concerns regarding soil anisotropy, at least in the tested geology.

The proposed method is in principle suitable for the geotechnical characterisation of many types of problems: dams, slopes, piles, foundations, etc. and many geotechnical materials: residual soils, heavily weathered rock, gravel, clays, etc. as the pressuremeter is an in-situ test that can be pre-drilled for its installation.

A notable feature of using pressuremeters in conjunction with finite element models is that constitutive models are calibrated as a whole with all soil parameters being properly interconnected to reproduce the full stress-strain response, something that cannot be achieved by conventional SPT or CPTu tests. The simplifications historically used for solving the mathematics involved in reducing the boundary value problem to a single element curve can be overridden, hence adopting much more sophisticated and realistic soil models and consequently obtaining more accurate geotechnical properties. This can be achieved resorting to strict fundamental soil mechanics principles without the need for empiricism.

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