Use of Ménard pressuremeter modulus in finite element models for retaining walls design

François PHILIP
EGIS Géotechnique, Lyon, France, francois.philip@egis.fr

Ziyad El Balqui
Fugro, Nantes, France, z.elbalqui@fugro.com

Pierre SCHMITT
Soletanche Bachy, Rueil-Malmaison, France, pierre.schmitt@soletanche-bachy.com

Jean-Pierre BAUD
Eurogéo, Avrainville, France, baud@eurogeo.fr

ABSTRACT: Pressuremeter tests with an unloading / reloading cycle were carried out for the design of the permanent diaphragm walls of the underground stations of line 3 phase 3 of the Cairo metro project. The goal was to define the soil behavior parameters used in finite element calculations with the Plaxis software. This work has been further developed within the framework of the ARSCOP national research project and has been divided into three parts:

- Relationship between the deformation modulus and the pressuremeter modulus, past and recent interpretations relating to finite element method calculations of retaining walls.
- Modeling pressuremeter tests with unloading-reloading cycle with Plaxis, sensitivity analysis on soil parameters. Complexity of the non-linear behavior of soils with regard to the use of an elasticity modulus E in the FEM codes.
- Calibration of Hardening Soil Model (HSM) parameters of Plaxis, from the pressuremeter tests results, in order to correlate with the inclinometer measurement results of the Nasser station.

Keywords: Ménard pressuremeter tests, cyclic test, soil deformation modulus, finite elements calculation, retaining walls and diaphragm walls.

1. Introduction

This article presents the work carried out within the framework of the french national ARSCOP project based on the retaining walls design and construction of the stations of the line 3 phase 3 of the Cairo metro, built by a consortium of 4 companies Vinci, Bouygues, Arabco and Orascom.

It appears that retaining walls design with finite element calculations are increasingly used in addition to traditional methods. The case of the Line 3 phase 3 project of the Cairo metro is a typical example, the finite element calculations were required by the developer N.A.T. (National Authority for Tunnels). In this context the geotechnical engineer has to face the difficulty of choosing the parameters of soil behavior laws to provide a sufficiently reliable prediction of the forces and the deformations of the retaining walls especially regarding the neighbouring structures in a very dense urban area. Another difficulty, because of the lack of international papers on this topic, is to agree with the other consultants and design checkers on the parameters chosen to obtain the approval.

For the engineer, the pressuremeter test allowing an in situ measurement of the deformation modulus appears ideal. But the use of this modulus as an input in computation models is not direct. The deformation rate of the measurement range of the pressuremeter test (distortion between 1 to 10%) is higher than that of the retaining wall (distorsion about 0.01%), the modulus is purely deviatoric and the deformation measured with the probe is horizontal.

This article, after a reminder of the relations between the pressuremeter modulus and the deformation modulus, will deal with pressuremeter tests with unloading / reloading loop carried out on the Cairo metro construction site. The results of the calibration of the HSM law parameters will then be presented together with the inclinometer measurements of the diaphragm walls of the Nasser station.

2. Link between soil moduli and Ménard pressuremeter modulus

The link between soil moduli and pressuremeter modulus was initially based on Louis Ménard's analysis and calculation of shallow foundations settlement.

As explained in his articles "Ménard et al [1]&[2]", Ménard, considering that settlements are nothing but a combination of isotropic compression and pure shear strains, based on two linear elastic analytical formulas :

- Isotropic component : shrinkage of a sphere submitted to an isotropic loading

$$w_s = \frac{1}{9.E_M}\sigma.D$$

- Deviatoric component : dilation of a spherical cavity, inflated within an elastic layer $w_d = \frac{1+\nu}{3.E_M}\sigma.D$

Then Ménard empirically adjusted these theoretical formulas in order to match the measured settlements.

Having demonstrated that the pressuremeter modulus is purely deviatoric, and observing that some categories of soil are less resistant than others when submitted to specific loading such as pure extension, which tends to generate tensile stress, he suggests that the pressuremeter modulus is finally corrected by a « structural » coefficient, α, in order to obtain a soil modulus $E_M/\alpha \approx E_{oed}$ more representative of usual spheric loading conditions, which include a significant part of isotropic compression, while the same rheological coefficient was used to adjust the deviatoric part of settlement to the size of the loading, and completed by classical shape coefficents.

L. Ménard [3] fixed in 1975 his equation for settlement of an isolated footing, and it was adopted in French codes and later in the Eurocodes :

$$w = \frac{\alpha}{9.E_s}\sigma.\lambda_s.D + \frac{1+\nu}{3.E_d}\sigma.D_0\left(\lambda_d\frac{D}{D_0}\right)^{\alpha}$$

In the case of normally consolidated soils, E_M/α cannot be definitively considered as an elastic modulus, since it clearly corresponds to a primary loading, exerted outside of the overconsolidated area ; in such a case, a link with soil moduli that are nowadays often introduced in numerical models could be proposed, considering for instance the hyperbolic behavior of the HSM model Fig. 1. This can be done by by simply assimilating E_M/α and the initial modulus, E_i, representative of the soil behavior in situ (that is in "K_0" conditions).

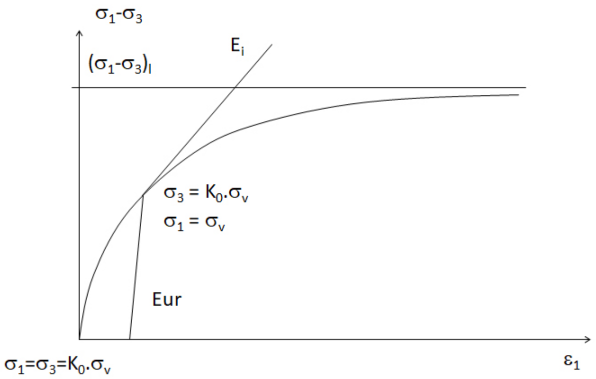


Figure 1. Graph of hyperbolic behavior and soil moduli

This primary loading modulus obviously needs to be distinguished, and is substantially lower than the elastic unloading-reloading modulus E_{ur}, when considering a triaxial stress path, or than the alternate modulus E_a, when considering a purely deviatoric stress path. Baud et al.[5] proposed to retain the initial relationship of Menard in order to define the α coefficient : $E_a = E_M/\alpha^2$.

In the case of a normally consolidated soil, for which initial stress conditions may be described by the at rest earth pressure coefficient K_0 = 1-sinΦ, and for which plastic behavior may be described by Coulomb criterion, associated with a passive earth pressure coefficient K_p = (1+sinΦ)/(1-sinΦ), it may easily be shown that in situ initial conditions theoretically correspond to a deviatoric stress σ1-σ3 that is equal to a half of the ultimate plastic deviator $(\sigma1-\sigma3)_l$.

It is then possible to make a link with the conventional secant modulus E_{50} that serves as a reference in the HSM model : as a matter of fact it may be shown that, in the case of a hyperbolic behavior, the tangent modulus is in each location equal to half the secant modulus, so that it can be concluded that $E_{50} = 2.E_i = 2.E_M/\alpha$.

The latter relationship is nowadays more especially used as it reasonably matches a significant number of back-analysis calculations, based on measured displacements of retaining structures in normally consolidated soils (higher values being generally encountered in overconsolidated soils).

When a linear perfectly plastic model is used, such as the so-called Mohr-Coulomb model, that does not make it possible to distinguish between primary loading and unloading-reloading stress paths, the representative soil modulus that needs to be introduced in the calculation cannot be the same in all circumstances :

- when the soil is essentially submitted to a primary loading (foundation built on a normally consolidated soil), the representative modulus should rather be chosen close to $E_i=E_M/\alpha$;
- this is no longer true when the soil is essentially subjected to an unloading-reloading stress path (such as in front of a retaining structure, for which the soil is unloaded due to the excavation and reloaded due to the horizontal displacement, or behind the same retaining structure, where the soil is unloaded due to the horizontal displacement and possibly reloaded due to prestressing of anchors).

In the last case, back-analysis calculations generally show that the representative modulus must be chosen intermediate between the primary loading modulus $E_i = E_M/\alpha$ and the unloading-reloading modulus E_{ur}, conventionally equal to 3 . E_{50} = 6 . E_M/α.

In practice, a relevant order of magnitude has been shown to be $E = 4.E_M/\alpha$, considering either continuum numerical models such as finite elements, or traditional calculations using the subgrade reaction coefficient k, associated with the classical relationship k = 3.6 / $(E_M/(\alpha.a))$ [3], where "a" is the interaction length along which the retaining structure mobilizes passive earth reaction.

It must be emphasized that both approaches generally lead to similar results, provided that they rely on the same values of initial primary soil moduli E_M/α.

In practice, it seems that the only reason why calculations of retaining structure based on pressuremeter measurements are rarely used for international projects is the empirical definition of the α coefficient.

This is the reason why pressuremeter tests including unloading-reloading cycles have been undertaken for the Cairo metro project.

Test results have been systematically analyzed as part of the French national research project ARSCOP : it may be anticipated that systematizing such tests should enable a rational definition of the coefficient α based on the $E_a = E_M/\alpha^2$ relationship, that has already been validated by a significant number of tests.

3. PMT survey for Cairo Metro CML3

Several lines of the Cairo metro have been built in recent years by the Egyptian joint venture - Vinci, Bouygues, Arabco and Orascom, on behalf of the NAT. Geotechnical drilling included core drilling with identification and geomechanical testing of samples, SPT profiles, CPT profiles and pressuremeter soundings have been performed. During the successive studies, an increasingly efficient collaboration was set up between the drilling teams from Ardaman and the French operators from Eurogéo specializing in PMT, in order to develop the best pre boring method for the pressuremeter in the alluvium of the Nile which mainly consists of sandy-silty, sometimes with more or less thick clay layers. The risk of drill hole collapse in a predominantly granular material under the water table has led to the preference of a destructive pre-drilling of 63 to 66 mm in diameter, with the use of a bentonite slurry and the addition of barite powder. The tests were also performed using the 44 mm three-cell Ménard probe inside a 63 mm OD / 49 mm OD outer diameter tube, for each successive drilling stage for 3 tests. During the placement of the probe in its slotted tube and throughout the tests, a low flow of bentonite is maintained overflowing from the drilling head, so that the mud column and the slotted tube hold the borehole wall stress conditions before the beginning of the test as close as possible to the earth pressure at rest, and ensure less remodeling of the borehole walls. As drilling progresses by stages, a temporary lining casing (diameter 95-104 mm) is lowered to minimize losses of drilling fluid in a relatively permeable medium.

The pressuremeter tests used in this article are those of the pressuremeter campaign for the CML3 project in particular for the Sudan, Kit-Kat and Nasser stations. The tests were carried out according to the Ménard EN-ISO 22476-4 procedure, and all make it possible to determine an E_M module in the pseudoelastic phase and a limit pressure PLM ; a part of the tests gave rise to an unloading-reloading loop in the pseudoelastic phase from which a cyclic module Ea is obtained.

Fig. 2 gives an overview of the results of the 3 stations cited between 5m and 50m deep, which represent mainly sandy layers. The profile can be classified into 3 levels :

1. from the ground surfce to 11 m depth, soft soils consisting of more recent alluvium and urban fills
2. between 11 m and around 22m depth, upper sands
3. from 22m to 50m depth, lower sands.

On Pressiorama diagram [p^*_{LM} /p0, E_M/p^*_{LM}, in Fig.3, lower sands appear to be very homogeneous , with a mean E/PL ratio around 7, and extreme values ranging from 4 to 15 ; these rather low values had already been observed in Cairo sands in previous surveys for former Metro lines.

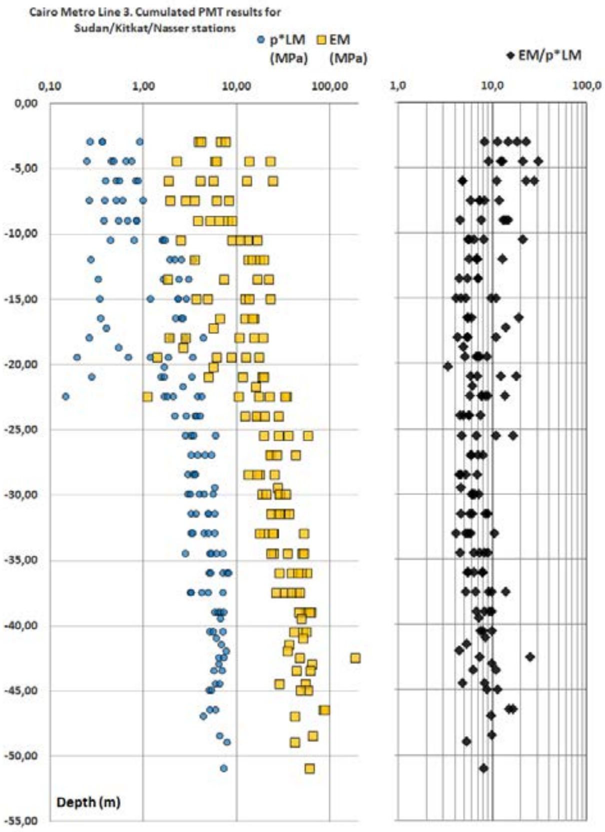


Figure 2. PMT results p^*_{LM}, E_M and E_M/p^*_{LM} ratio from 3 boreholes in Cairo Metro Line 3

In the sand of Cairo CML3, the average ratio Ea/E_M measured is around 5. This ratio is lower than those found in sands on experimental sites in France and whose results are presented in "Combarieu et Canepa [9]". We believe that this difference is coming from the drilling method used in Cairo, which required the use of bentonite mud heavier than usual in order to stabilized the borehole in sand material under water. This explanation is found consistent with the observations of Combarieu and Canepa [9] which have shown that the drilling method has an influence on the value of E_M more than on the value of Ea..Menard's idea in his article [1] of linking α coefficient to the Ea/E_M ratio would therefore integrate the influence of the drilling method in a certain way. We can propose (Baud [6] in this ISC6 symposium) to correct the expression by an index of borehole decompression *d* based on curve's initial curvature. For Cairo tests, a mean value d=0.15 has been retained to plot alpha values in Fig. 3. Nevertheless, the resulting values range from 1/4 to 1/2 and are centered around 1/3,which is the current value given for sandy soils.

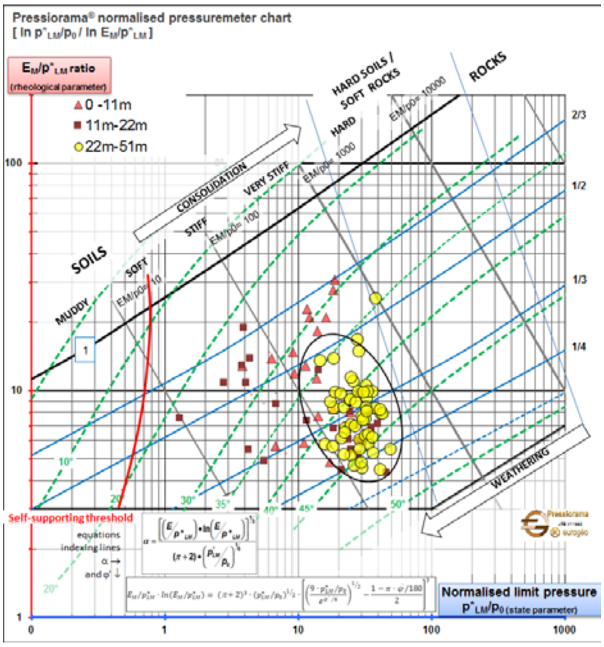


Figure 3. Classification of PMT results from 3 boreholes in Cairo Metro Line 3

4. Modeling pressuremeter tests including an unloading / reloading cycle

In this part, calibration results of 2D Plaxis models attempting to replicate the pressuremeter tests with unloading/reloading cycle are presented. As part of the Cairo Line 3 Phase 3 metro project, Eurogéo has carried out several pressuremeter tests with an unloading/reloading cycle at different depths. Only tests that took place in sands are treated in this part and have been used for FEM models by Z. El Balqui et al. [7].

The model is axisymmetric, the borehole is modelled with a radius of 30cm with a depth of 1m below the test's depth. A hydrostatic pressure, calculated with a density of 11 kN/m^3, acting as bentonite is applied to the walls of the borehole.

The soil is governed by the Hardening Soil Model (HSM): non-linear elastic combined with the Mohr-Coulomb plasticity criterion.

The Hardening Soil Model in Plaxis is characterized by several reference parameters $E_{50,ref}$, $E_{oed,ref}$ et $E_{ur,ref}$, p_{ref} and m . This soil model also enables users to take into account the increase in the modules with the minor main stress. For the sake of simplification, this effect of module increment has been canceled by taking m = 0, then we have :

$E_{50} = E_{50,ref}$

$E_{oed} = E_{oed,ref}$

$E_{ur} = E_{ur,ref}$

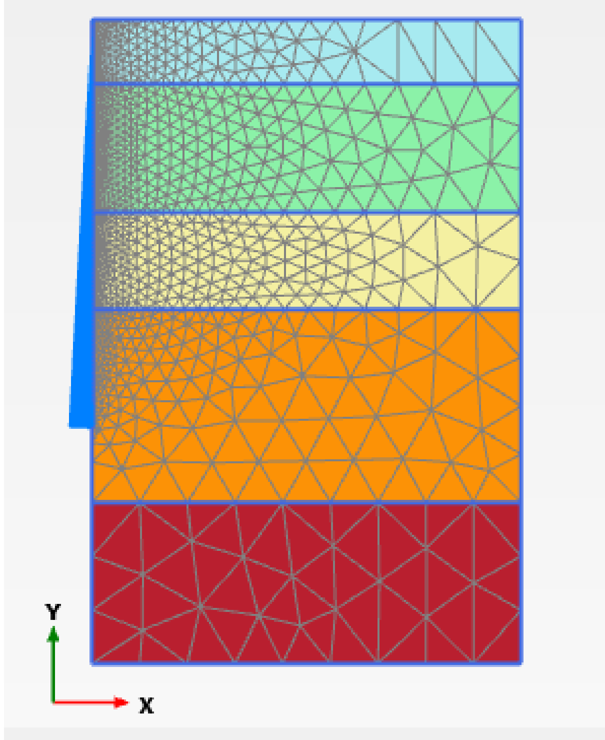


Figure 4. Pressuremeter test view of Plaxis model

The correlations that achieved the best calibration of the pressuremeter tests are:

$E_{50} = 3*E_M$

$E_{ur} = E_a$

Where: E_a is the alternate modulus of a cyclic pressuremeter test.

Based on the above, it should be noted that only tests with a ratio E_a/E_M larger or equal than 6 can be modeled with the Plaxis HSM law. In fact, this law requires that the E_{ur} modulus is greater or equal to $2*E_{50}$:

$E_a = E_{ur}$ with $E_{ur} \geq 2*E_{50}$ ➔ $E_{ur} \geq 2*3E_M$ ➔ $E_a/E_M \geq 6$.

The table below summarized the tests modeled under Plaxis2D:

Table 1. Summary of pressuremeter tests modelled with Plaxis

Test location	Ratio E_a/E_M	Model	Quality of calibration
SUDAN z=9m	6.8	HSM	Very good
SUDAN z=12m	4.8		Good
SUDAN z=24m	4.8		Good
MASPERO z=15m	4.4	HSM	Acceptable
MASPERO z=18m	4.5	HSM	Acceptable
MASPERO z=22.5m	5.9	HSM	Very good
MASPERO z=34.5m	4.7	HSM	Acceptable
MASPERO z=37.5m	4.8	HSM	Good
NASSER z=18m	3.0		Poor
KIT-KAT z=12m	6.2	HSM	Very good
KIT-KAT z=24m	5.0		Good
KIT-KAT z=30m	3.8		Poor
KIT-KAT z=33m	3.1		Poor
KIT-KAT z=39m	6.7	HSM	Very good
KIT-KAT z=46.5m	4.1		Acceptable
BH z=30m	6.0	HSM	Very good

The calibration quality of the tests having a ratio E_a/E_M around 6 is very good, while those whose ratio is much lower than 6 are impossible to reproduce in Plaxis with a Hardening Soil Model.

The calibration results of two pressuremeter tests are presented in detail below:

- Station Sudan, depth of test z=9m:

Table 2. Sudan Plaxis calibration parameters

	E_{50} (Mpa)	E_{ur} (MPa)	φ (°)	Ψ (°)	c (kPa)
HSM 1	20.0	40	45	10	1
HSM 2	18.5	37	45	12	5

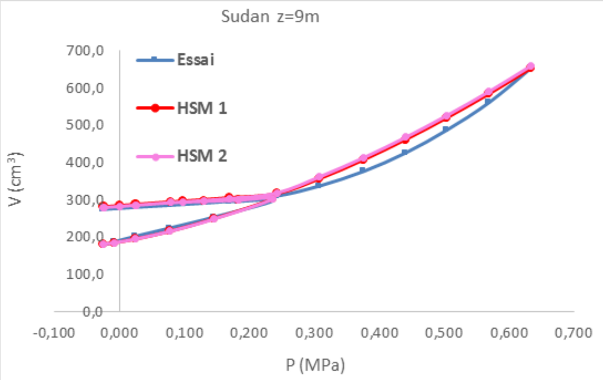


Figure 5. Sudan pressuremeter test depth z=9m

Table 3. Sudan Pressuremeter modulus

	E_M (MPa)	E_a (MPa)
Test	5.2	35.2
HSM 1	5.2	36.5
HSM 2	5.2	25.3

- Kit-Kat station, depth of test z=12m

Table 4. Kit Kat Plaxis calibration parameters

	E_{50} (Mpa)	E_{ur} (MPa)	φ (°)	Ψ (°)	c (kPa)
HSM 1	11.5	23	40	10	5
HSM 2	12.0	24	38	8	5

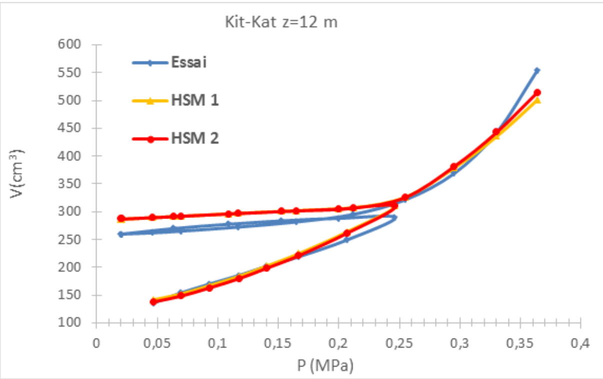


Figure 6. Kit Kat pressuremeter test depth z=9m

Table 5. Kit Kat Pressuremeter modulus

	E_M (MPa)	E_a (MPa)
Test	3.5	21.8
HSM 1	3.2	27.5
HSM 2	3.2	28.7

It should be noted that in order to match with the second part of the curve beyond the loop, high shear resistance parameters for sand had to be used. Lower shear resistance values led to a fast plasticity compared to the curve of the test.

It must also be remembered that we have been limited by the law HSM on Plaxis which imposes $E_{ur} \geq 2E_{50}$, without this limitation the calibration of the tests whose ratio E_a/E_M is less than 6 could have been possible.

To conclude, this method of calibrating a test with an unloading / reloading cycle using a Hardening Soil Model is relatively simple to implement and for the ratio $E_a/E_M \geq 6$ the moduli are well reproduced. The calibrations give $E_{ur} = E_a$, which shows that the calculation model is relevent to reproduce the pressuremeter test in the cyclic part, where the behavior of the soil is of linear elastic type. Nevertheless some questions were raised. As expected, things are more complex for the simulation of the first loading part of the curve, and for which we can only make two extreme assumptions:

1. the tangent modulus of the first loading is equal to E_M/α, which should correspond to a secant modulus E_{50} of the order of $2E_M/\alpha = 6.E_M$ in accordance with §2; however, this assumption assumes that the model is able to simulate a triaxial test and a pressuremeter test with a single set of parameters, which is equivalent to admiting that the empirical "coefficient of structure" α introduced by Ménard to precisely take into account the different responses of soils according to their nature in these two fields of extremely different constraints (vertical compression in one case, horizontal extension in the other), can be found by a simple numerical calculation, which is unlikely.
2. the tangent modulus of first loading is equal to E_M, so a secant modulus of the order of $2.E_M$, which seems a priori more logical for the simulation of a pressuremeter test which is not triaxial, but which may not be well adapted to take into account the fact that, as reported by Flavigny and / or Cambou, the pressuremeter test in extension induces from the beginning a significant plastification, which transforms the measured modulus into an apparent modulus, lower than the intrinsec modulus that must be introduced into the law of behavior?

In any case, the calibrations give an E_{50} modulus of the order of $3.5E_M$, which is well within the predictable range of 2 to 6. In addition, the fact that unrealistic shear parameters must be introduced to calibrate the model demonstrates the limitations of models primarily based on triaxial tests in reproducing a pressuremeter test, as reported from the beginning by Ménard, a difficulty/limitation which is usually encounteredwhen calculating the modulus value.

5. Presentation of Nasser station

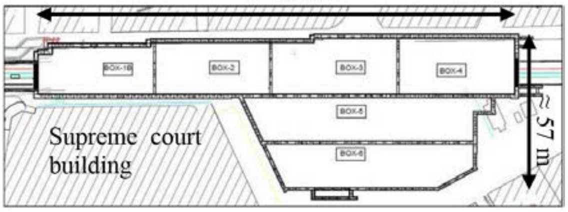


Figure 7. Plan view of Nasser station

Nasser station is one of the major underground station of the line 3 phase 3 extension. The whole station is divided into 6 boxes where boxes 1 to 4 are separated together by three transversal bentonite cement non reinforced diaphragm walls with the objective of limiting the reinjection works to one box (injected plug made of soft gel, bottom at 70m depth and thickness about 12m) in case of leakage of the plug. Whereas the boxes 5 and 6 are separated by two structural reinforced diaphragm walls required for the service stage. Below the plan view of the station with the main dimensions and showing the buildings arround at a very close distance from the DWall (sometimes not more than 2m).

The geotechnical context is relatively homogeneous along the station, The following layers were encountered from the top to bottom:

- A layer of backfill of about 3m
- A layer of clay of about 9m
- A layer of upper sand moderately dense to dense of about 10m
- A layer of lower sand very dense

The ground level is at +20.5m, the deepest excavation level is at -12.5m and the Ground water level at 18.4m.

Table 6. Nasser design geotechnical parameters

	Top level	γ	c_u	c'	φ'	E'	α	E_M
	(m asl)	(kN/m³)	(kPa)	(kPa)	(°)	(MPa)	(-)	(MPa)
Fill	20.5	17	-	0	27	17	0.50	4.2
Clay	17.0	19	80	0	27	18	0.67	8.0
Clayey silt north	12.7	19	70	0	27	16	0.50	4.0
Clayey silt south	12.7	18	50	0	27	11	0.50	2.9
Upper sand	8.0	19	-	0	40	150	0.33	16.0
Lower sand	-2.0	20	-	0	40	350	0.33	38.0

The thickness of external DWall of box 1 to 4 has been set to 1.5m and the panels size on site have been limited to the width of the hydrofraise tool equal to 2.8m. The purpose was to limit as much as possible the displacements in order to minimize the impact on the surrounding buildings.

Sequence of works:

Nasser station is the first station crossed by the TBM which starts from Attaba station. The 6 boxes have not been excavated at the same time in order to enable the TBM to go through Nasser station as early as possible. Boxes 1 to 4 have first been excavated whereas excavation of boxes 5 and 6 started once the raft of Boxes 1 to 4 had been completed and with the TBM inside the station.

The sequence of works for box 1 to 4 is as follows:

1. D walls construction and injected plug
2. Pumping tests inside each box and draw down of ground water inside boxes 1 to 4 at -13m
3. First excavation to 18.3m
4. Roof casting
5. Excavation to 9.8m
6. Casting of strut 1 axis at 10.5m
7. Excavation to 4.3m
8. Casting of strut 2 axis at 5.0m
9. Excavation to -0.5m
10. Casting intermediate slab axis at 0.05m
11. Excavation to -4.3m
12. Casting of strut3 axis at -3.6m
13. Excavation to -8.3m
14. Casting strut 4 axis at -7.6m
15. Bottom excavation at -12.4m
16. Casting of the Raft
17. Removing of struts 3 and 4

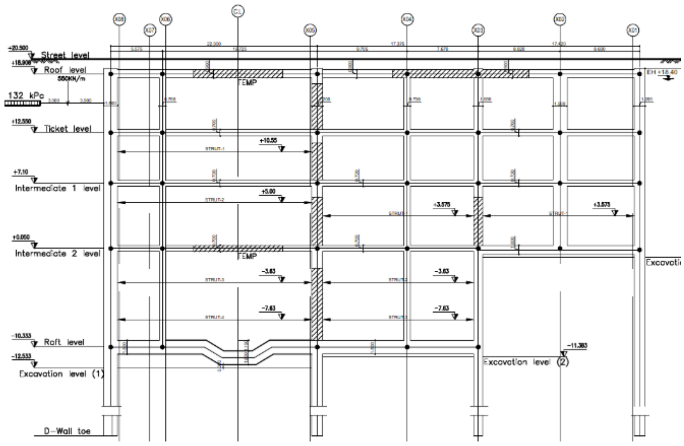


Figure 8. Section of Nasser station

6. Plaxis HSM calibration results from inclinometers

The calibration calculations were carried out from the same Plaxis sections as the ones used for the design of the diaphragm walls. It should be noted that the design has been done with a perfectly plastic linear elastic law using the Mohr Coulomb criteria where the elastic moduli have been evaluated by means of the correlation $E = E_M/\alpha^2$ [4], with E_M the pressuremeter modulus and α the Ménard's reological coefficient. It can be noted that this correlation leads to slightly lower values than those obtained for retaining walls $E = 4E_M/\alpha$ [5]. It has however been difficult to convince and get the approval in a situation involving an Egyptian consultant who is not very familiar with the pressuremeter tests and whose reference books in english reported the correlation $E = E_M/\alpha$.

For the calibration calculations, the Plaxis section corresponding to the inclinometers position has been used with the the only modification being the use of the HSM constitutive law instead of the linear elastic perfectely plastic model. Several sets of parameters have been tested and the best fit has been found with the following HSM parameters:

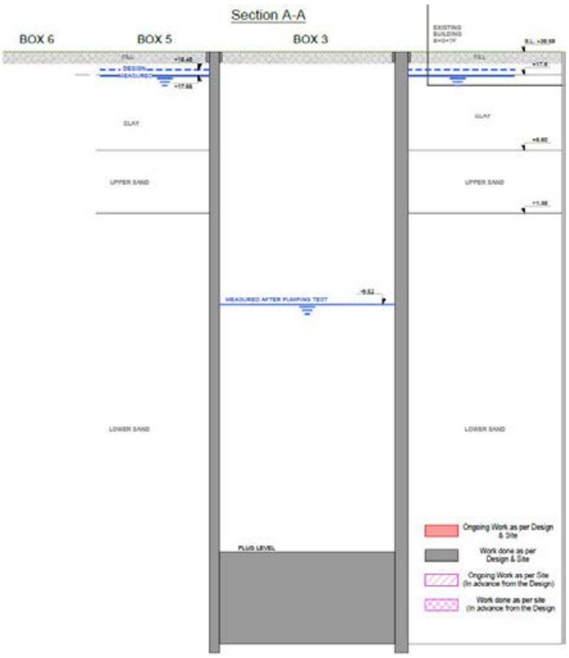
- $P_{ref} = 100$ kPa
- $m = 0$
- $E_{50} = 2*E_M/\alpha$
- $E_{oed} = E_{50}$
- $E_{ur} = 3*E_{50}$

It is to be noted that Nasser station is still under construction at the time of writing of this article, and the available inclinometers results stop just after casting the raft and removing the struts above of boxes 1 to 4 and before the start of works for boxes 5 and 6.

Fig. 9 to 13 present the calibration results of section CS 15 North (INC4) obtained during the construction progression :

1. Pumping test
2. Excavation to strut 2 at -4.3m
3. Excavation to strut 3 at -4.3m
4. Excavation to strut 4 at -8.3m
5. Excavation to raft at -12.4m

6.1. Calibration results, section CS15 North: Pumping test



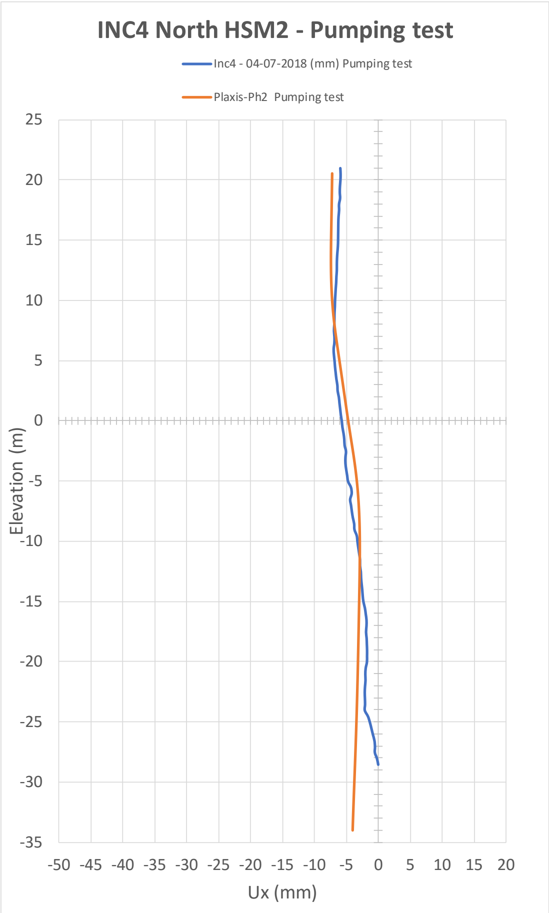


Figure 9. Calibration results – Phase pumping test

6.2. Calibration results, section CS15 North: Excavation to Strut 2 at -4.3m

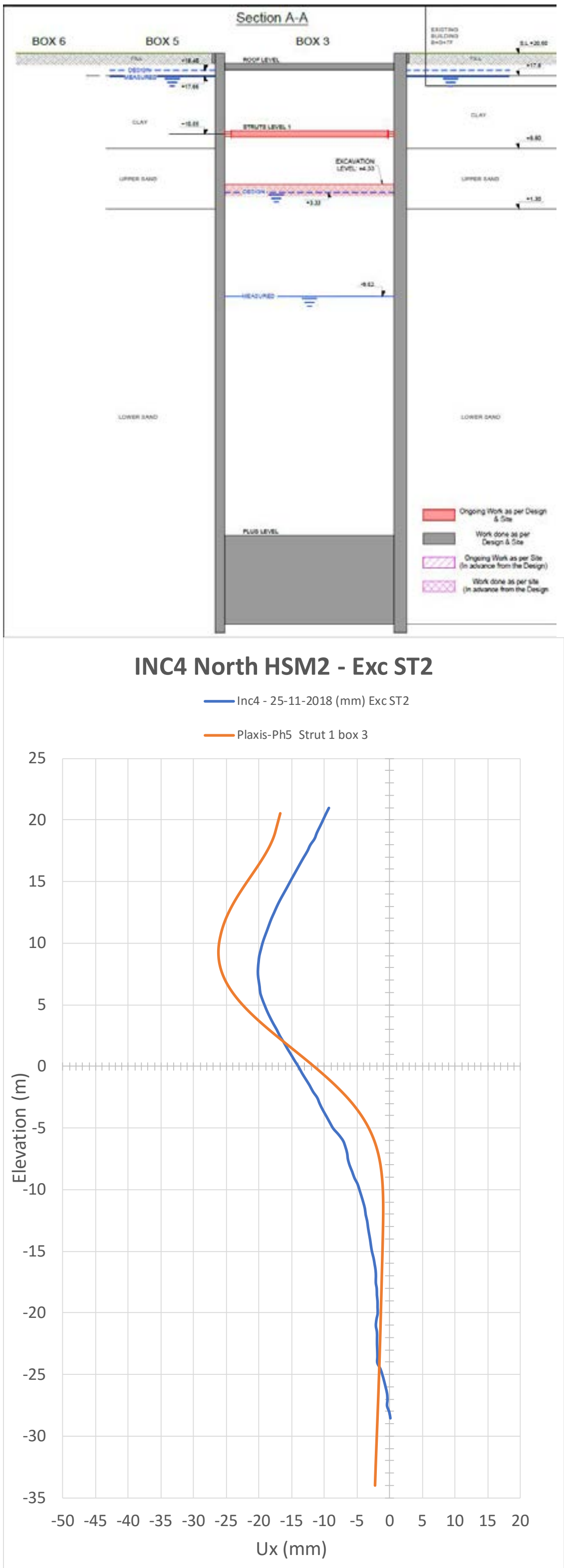


Figure 10. Calibration results – Phase Exc. Strut 2

6.3. Calibration results, section CS15 North: Excavation to Strut 3 at -4.3m

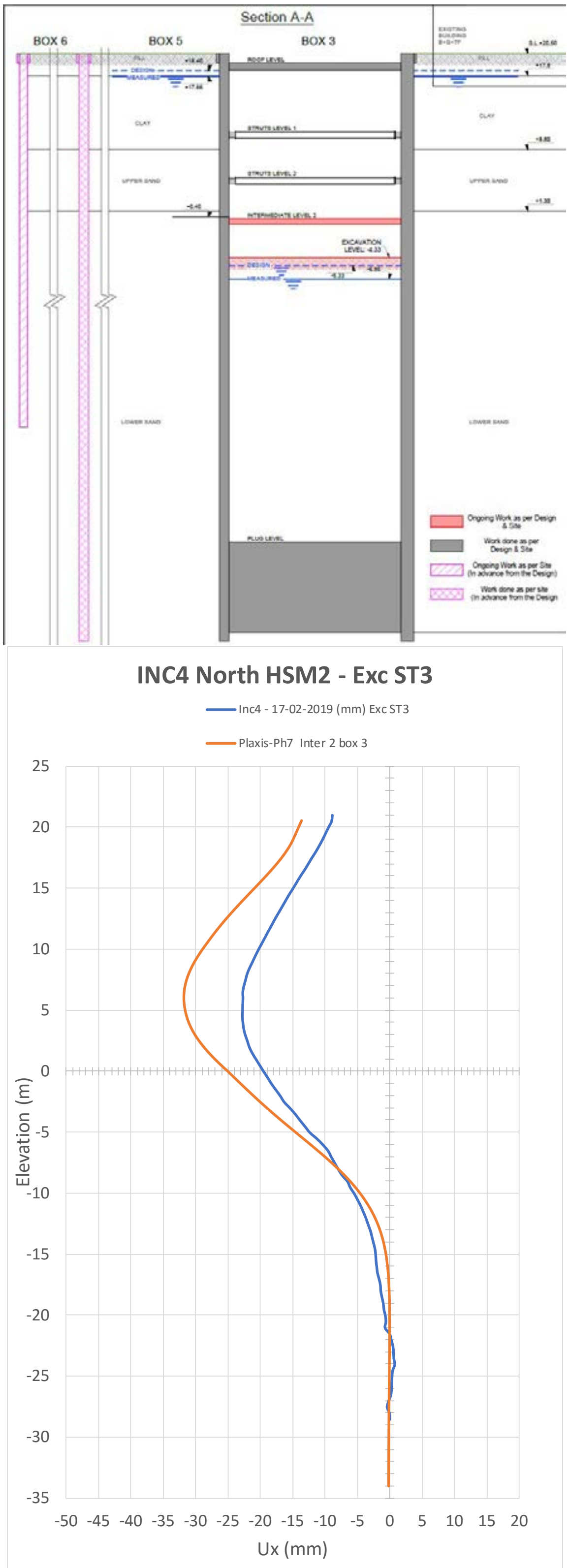


Figure 11. Calibration results – Phase Exc. Strut 3

6.4. Calibration results, section CS15 North: Excavation to Strut 4 at -8.3m

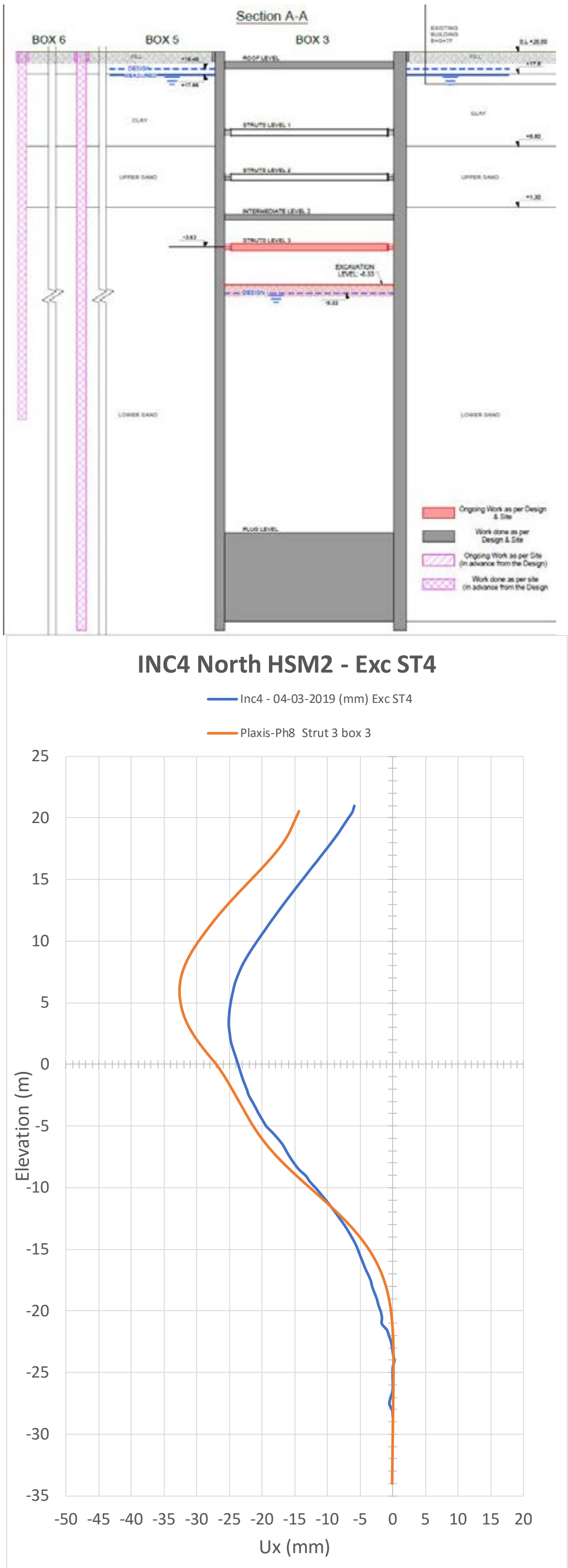


Figure 12. Calibration results – Phase Exc. Strut 4

6.5. Calibration results, section CS15 North: Excavation to Raft at -12.4m

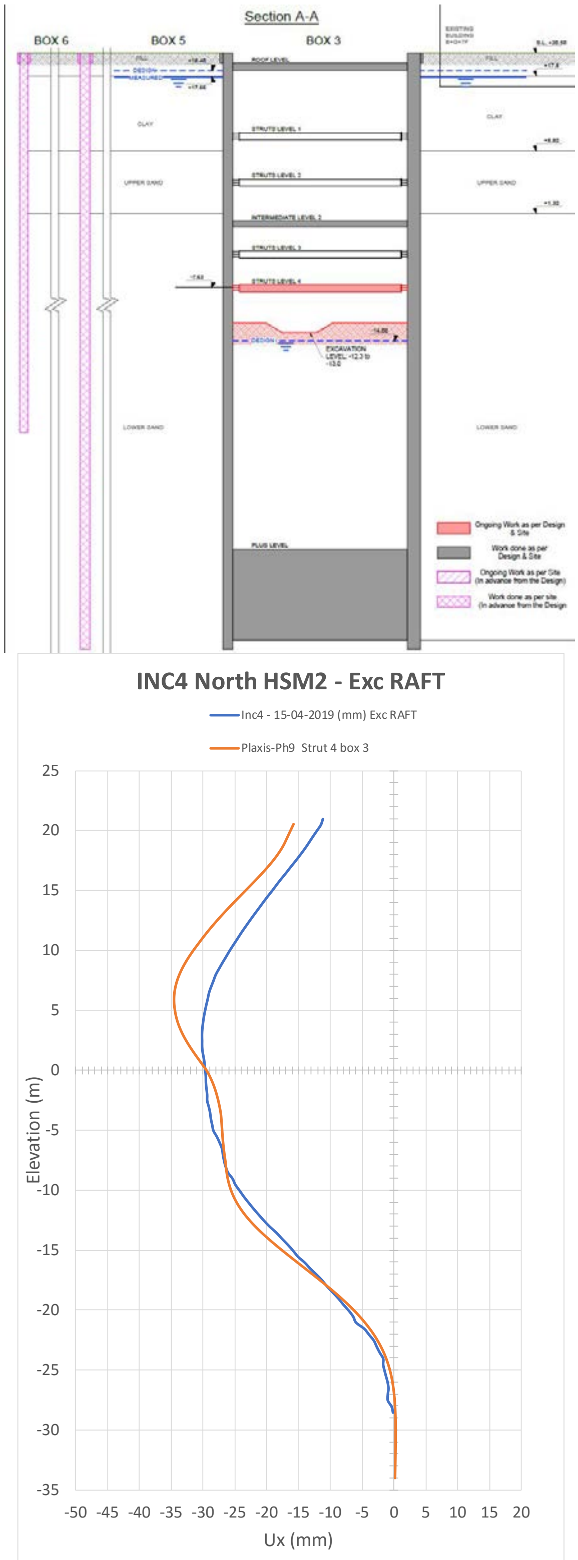


Figure 13. Calibration results – Phase Exc. To raft

7. Conclusion

Reproducing pressuremeter tests of the Cairo metro using a Plaxis finite element calculation with the HSM law enabled us to obtain very good calibrations but only when the ratio E_a/E_M is at least equal to 6. This comes from a limitation of the HSM law in Plaxis which imposes $E_{ur,ref} \geq 2*E_{50,ref}$. However, there is a need to introduce abnormally high shear parameters and the correlations between HSM and pressuremeter parameters differ from those found for retaining walls calculations.

The link between the deformation modulus and the pressuremeter modulus should not be limited to the correlation $E = E_M/\alpha$ because it leads to wrong results particulary in the case of retaining walls calculations [7]. For retaining walls, it has been shown [5] that it is possible to use a HSM law whose input parameters can be correlated with the pressuremeter modulus as follows:

- $P_{ref} = 100$ kPa
- $m = 0$
- $E_{50} = 2*E_M/\alpha$
- $E_{oed} = E_{50}$
- $E_{ur} = 3*E_{50}$

The calibration of a Plaxis HSM calculation of Nasser station's diaphragm wall with the results of the inclinometer measurements, using the above correlations, gave good results. The curvatures of the retaining wall calculated with Plaxis are found, at each phase of construction, to be relatively comparable to the inclinometer measurements. This calibration exercise, in a new context, reinforces the results found previously [5] and then increases our confidence in finding a good prediction of retaining wall deformations following this approach.

The use of finite element calculations is in certain configurations the only way to apprehend a satisfactory model of the problem, hence the importance of having a reliable established calculation methodology. But they must not unnecessarily replace the traditional method of subgrade reaction modulus, which has the enormous advantage for the engineer to be simple and fast to implement.

We will conclude by recalling the importance of instrumentation and monotoring of the deformations of structures interacting with the soil. They are not only enable the verification of the the predictions made by the calculations but also anticipate a possible wrong prediction which enables the modification of the construction with a reasonable economic impact. Instrumentation and monitoring also enable the improvement of models through subsquent calibration.

8. References

[1] Louis Ménard et Jean Rousseau « L'évaluation des tassements, Tendances nouvelles », Sols-Soils-n°01-1962

[2] L.Ménard et J.-P. Dauvisis « Etude expérimentale du tassement et de la force portante de fondations superficielles » (Experimental study of settlement and bearing capacity of superficial foundations), Sols-Soils n°10-1964 pp 11-23 (in French, synopsis in English, German, Spanish)

[3] L.Ménard « The interpretation of pressuremeter test results », Sols-Soils n°26-1975, pp 5-44

[4] P. Schmitt « De l'importance du suivi pour maîtriser le dimensionnement des ouvrages géotechniques « (The importance of monitoring to control the design of geotechnical retaining structures) Revue française de Géotechnique n°126-127 1er et 2e trimestres 2009 (in French)

[5] J.-P. Baud & M. Gambin "Soil and Rock Classification from High Pressure Borehole Expansion Tests" Geotechnical and Geological Engineering ISSN 0960-3182 Volume 32 Number 6 - Geotech Geol Eng (2014) 32:1397–1403

[6] J.-P. Baud "Soil and Rock Classification from Pressuremeter Data. Recent Developments and Applications" ISC'6, Budapest, 2020.

[7] ARSCOP "Projet National nouvelles Approches de Reconnaissance des Sols et de Conception des Ouvrages géotechniques avec le Pressiomètre") Rapport interne Z. El Balqui, F. Philip & J. Vescuere " Utilisation des essais pressiométriques Ménard pour l'application aux calculs des soutènements par éléments finis" (Use of Ménard pressuremeter tests for the calculations of the retaining structures with finite elements method) (2018) (in French)

[8] K. Serrai, C. Plumelle, P. Schmitt "Analysis of measured deflections of a diaphragm wall in Colombes using finite element calculations" 16th International Conference on Soil Mechanics and Geotechnical Engineering – 2055-2006 – doi: 10.3233/978-1-61499-656-9-1119

[9] O. Combarieu, Y. Canepa, "L'essai cyclique au pressiomètre" (The cyclic test with the pressuremeter) Bulletin des laboratoires des ponts et chaussées - 233 - JUILLET-AOÛT 2001 - RÉF. 4381 - PP. 37-65 (in French)

[10] O.Combarieu, "L'usage des modules de deformation en geotechnique" (The use of deformation modules in geotechnical design), Revue française de Géotechnique n°114 1er trimestres 2006. (in French)