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Using pressuremeter to obtain parameters to elastic-plastic models for sands

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ABSTRACT: This paper suggests how to estimate fundamental soil parameters to be used in constitutive laws by using pressuremeter test data. An elastic perfectly plastic theory is used involving dilation and an increase of the soil stiffness with the increase of the mean normal stress level: the PLAXIS Advance Mohr-Coulomb model of a F.E.M. code. This way, eight parameters can be identified, each of them having a physical meaning to civil engineers. This constitutive law is used to model triaxial tests on the same sands which were later tested by pressuremeter in a calibration chamber. It is shown that it is possible to characterize densification and dilation during these tests in the 10^{-2} strain domain.

1 INTRODUCTION

The pressuremeter test (PMT) is part of the in situ tests for which a theory can be built up since boundary values are known. Consequently PMT data should help derive fundamental parameters for soil constitutive laws. However, as for any other in situ tests, stresses and strains around a pressuremeter probe are not uniform and research workers must revert to numerical analysis to achieve their task. Several results have already been presented by Prevost and Hoeg (1975), Cambou et al. (1995), Sharour et al. (1995), Hicher et al. (1995), Gambin et al. (1996).

In this paper the behaviour of the soil around a pressuremeter is recreated using a elastic perfectly plastic model with dilation threshold according to the PLAXIS F.E.M. code available to every geotechnical engineer. This procedure will later help them to directly use the results of our research work. Further the chosen model, namely the "Advanced Mohr Coulomb" one is a simple model only involving soil parameters which have a physical meaning for the engineer.

During the first part of our work, having obtained the stress-strain parameters of the sand by triaxial tests, one series of PMT carried out in a calibration chamber at the "Laboratoire Sols, Solides, Structures - Lab. 3S" of Grenoble, by Mokrani (1990) is modelled. Conditions of placement of the sand and boundary conditions were perfectly controlled. In a second stage and using a back analysis relationships were set up between the PMT data and model parameters.

It must be pointed out that due to the careful placement of the sand around the pressuremeter probe in the calibration chamber our analysis is not affected by the usual drawbacks of PMT, namely stress relief and remoulding by drilling of the bore hole walls. Then it is judicious to compare pressuremeter modulus and soil modulus in the first part of the PMT loading, i.e. for 10^{-2} strain level. For a higher strain level, that is 10^{-1} it is also possible to compare the pressuremeter limit pressure and the critical state of the sand, as long as the numerical analysis involves a readapted procedure.

2 THE NUMERICAL ANALYSIS

We have already mentioned that the analysis uses the PLAXIS F.E.M. code developed at the Delft Technical University (Veermer et al, 1996) available to geotechnical engineers since the late 80's, and more specifically the "Advanced Mohr Coulomb" model.

2.1 *The constitutive law*

The "Advanced Mohr Coulomb" model is an elastic perfectly plastic model which exhibit two additional features:

- non linear elasticity, i.e. stress-dependent stiffness modulus;
- dilation cut-off, which is required for dense sands and for low stress levels after extensive shearing.

The elasticity parameters of the model are: ν, Poisson ratio and E, the stress dependent modulus, formulated as a power law:

$$E = E_{ref} \left(\frac{p}{p_{ref}}\right)^n \tag{1}$$

where: E_{ref} is the reference modulus corresponding to a reference stress p_{ref} = 100 kPa; p is the effective mean normal stress: $p = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$ (for cohesionless materials) and n is the power factor.

It must be noted that equation (1) does not affect the volume change which is only a function of ν, ψ, n_i et n_{max}.

The plasticity parameters are:

c: cohesion (zero for sand);

φ: the friction angle at peak ;

ψ: the dilation angle;

n_i et n_{max}: are the initial and maximum soil porosity;

Since c and φ fulfill the Mohr Coulomb perfectly plastic criterion, the ultimate deviator q_f is:

$$q_f = (c \cot\varphi - \sigma_3')\ \frac{2 \sin\varphi}{1 - \sin\varphi} \tag{2}$$

Soil volume changes are governed by ψ in the plastic potential functions whereby the dilation takes into account the energy which is dissipated along sliding surfaces during failure. Based on laboratory tests on various sands, the following formula between φ and ψ was proposed by Bolton (1986):

$$\varphi = 0.8\psi + \varphi_{crit} \tag{3}$$

where φ_{crit}, the critical friction angle measured during a drained triaxial test on a sample which exhibits a zero dilation rate in the plastic phase.

The initial and final porosity parameters n_i and n_{max} help obtain the constant dilation level which correspond to the critical state of the material:

$$\Delta\varepsilon_v = \frac{ln(1-n_i)}{ln(1-n)} \tag{4}$$

when n= n_{max}, ψ=0 in the PLAXIS model.

3 THE EXPERIMENTS

3.1 Material

A fine Hostun sand (Rf) was used which is well known in French geotechnical laboratories: d_{50}=0.32mm; $\frac{d_{60}}{d_{10}} = 2$; e_{min}=0.555; e_{max}=0.961; ρ_s=2.65 g/cm³.

3.2 Triaxial tests

A serie of conventional triaxial tests carried out on fine Hostun sand (Rf), with different relative densities between 2 and 98 per cent, confining pressure $\sigma'_2=\sigma'_3$ being 100 and 300 kPa, was collected from Bousquet et al (1993).

Figure 1 presents these triaxial results in a reference scale, which is very appropriate to check triaxial test results.

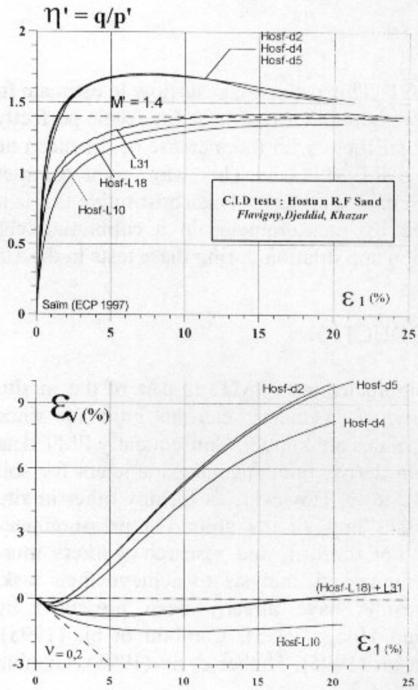

Figure 1. Triaxial test results.

3.3 PMT in calibration chamber

The calibration chamber is a cylinder 1.2 m in diameter and 1.5 m high.

Through confining rubber membranes it is possible to apply vertical pressures (by upper and lower flat membranes) and horizontal pressure (by a lateral cylindrical membrane) up to 600 kPa. Outside shell and lids of the chamber are perfectly rigid.

The pressuremeter probe is 55 mm in diameter and 160 mm long, made of a latex membrane supported by a rigid cylindrical core. The probe is placed vertically in the centre of the chamber and the sand, in a dry condition, is pluviated from a constant height around and also above the probe to achieve a constant density. Thus the sand is moulded around the probe.

A series of tests was performed by Mokrani and Foray (Mokrany, 1991) on the fine Hostun sand (Rf), with a vertical stress varying between 100 and 500 kPa and relative densities between 40 and 88 per cent.

Contrarily to the usual Menard pressuremeter curves, the curves in the calibration chamber (figure 2) exhibit neither a recompaction phase nor a quasi linear segment. So, in this situation the G modulus is calculated in the initial part of the curve, pressure loss and volume loss being taken into account.

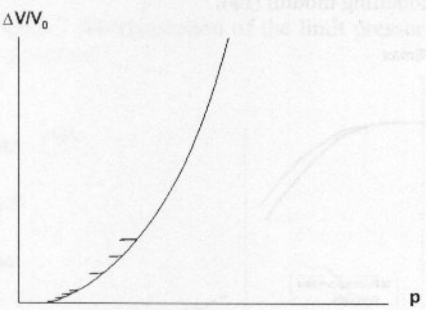

Figure 2. Typical PMT results in calibration chamber.

4 SAND BEHAVIOUR MODELLING

4.1 Triaxial tests modelling

Analysis is made in axisymetrical small strains; the sand being assumed drained.

During the fitting operation we privileged the best fitting of volume change as a function of strain, since dilation plays an important role in the PMT data. For the 10^{-2} strain level the best fit of the (ε_v, ε_1) curves requested to use the E_{ref} value as a secant modulus at 50 per cent of the maximum deviator. Table 1 gives the fitting values for triaxial tests. One can note the expected increase of the E_{ref} modulus when density increases and decrease of φ and ψ when the p value increases.

It must be noted the low values of E_{ref} obtained from modelling, varying from around 10 to 30 MPa. As was already mentioned this modulus is a reference secant modulus for 50 percent of the maximum deviator stress at a confining pressure ($\sigma'_2=\sigma'_3=p$) of 0.1 MPa. In consequence this modulus is much lower than the elastic modulus obtained by precision triaxial tests for 10^{-5} to 10^{-6} strain levels. The analysis of triaxial results in the same sand, expressed in terms of $\frac{e^2 E_s}{p^{3/4}}$ as a function of log ε_1 (figure 3), shows that the reference modulus correspond to a strain level around 2×10^{-2} . The Figure 3 may also be used to determine the appropriate secant modulus value for a particular geotechnical work, say for 10^{-3}.

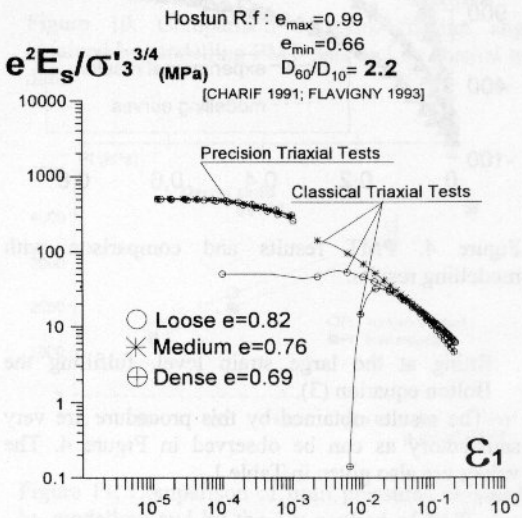

Figure 3. Method for extrapolating conventional triaxial test results for strain levels < 10^{-1} - 10^{-2}.

4.2 Pressuremeter test modelling

Again the analysis is in axisymetry, there are 192 triangular elements, each with 15 nodes. Dimensions and boundary values are those of the chamber.

4.2.1 Parametric analysis

In our model the parameters which mostly affect the pressuremeter data are E_{ref}, ν, φ and ψ as well as K_0, the pressure at rest coefficient. The density parameters seem to play a secondary role.

The fitting procedure was as follow:

- the K_0 coefficient is taken as 0.35, the mean value experimentally obtained;
- the density parameters n, n_i and n_{max} interpolated from the triaxial tests and the Poisson ratio are kept constant during the fitting;
- the E_p modulus then is selected to obtain the best fit at the origin of the pressuremeter curve.
- φ and ψ parameters are chosen to have a best

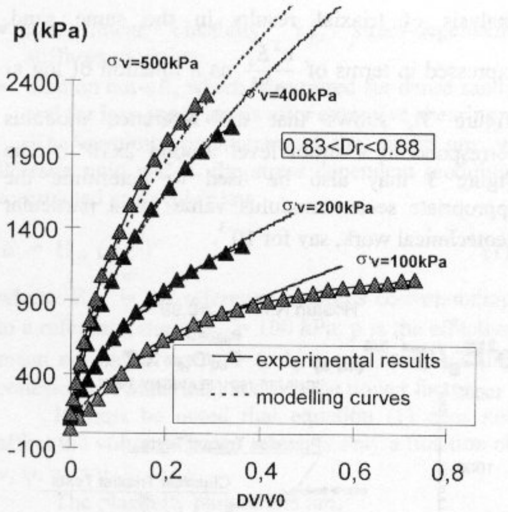

Figure 4. PMT results and comparison with modelling results.

fitting at the large strain level, fulfilling the Bolton equation (3).

The results obtained by this procedure are very satisfactory as can be observed in Figure 4. The values are also given in Table 1.

5 COMPARISON BETWEEN EXPERIMENTAL AND MODELLING RESULTS

5.1 Comparison of E moduli

Figure 5 shows the correlation between the PMT E moduli (E_{exp}) and those obtained by the fitting (E_p). We can observe that the values are very close, showing that the procedure is appropriated.

Using data from Biarez and Hicher (1994) and Charif (1995) which give the variation of E with strain level for the dense sand, Figure 6 demonstrates that the E moduli obtained perfectly fit the curve for 10^{-2} strain.

In consequence, if another strain level is more representative for the geotechnical design, another modulus must be choosen, for instance from data presented in Figure 3.

5.2 - Comparison of limit pressures

The conventional limit pressure as proposed by Louis Menard and his co-workers is obtained for a

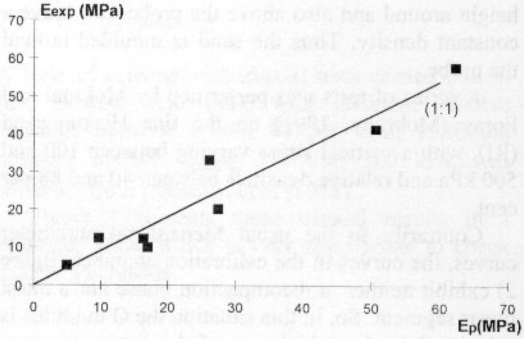

Figure 5. Comparison between PMT moduli (E_{exp}) and modelling moduli (E_p).

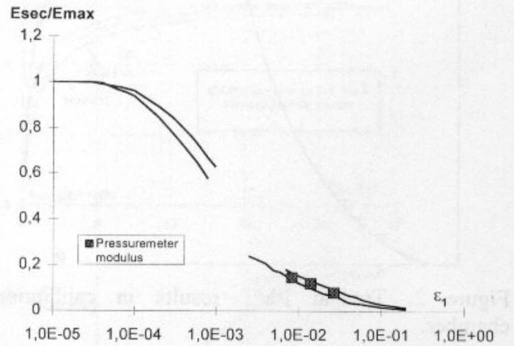

Figure 6. Strain level corresponding to E moduli obtained from PMT in calibration chamber for dense sand.

volume of the cavity twice the initial volume. Since this value is not obtained during most of the tests, the limit pressure is estimated here by log-log method and the inverse volume method (ASTM D 4719, 1986). Comparison with limit pressures derived from model is shown on Figures 7 and 8. For one part the good agreement between the two limite pressures is due to the way the fitting was obtained. Still it is only for the sand samples in loose condition that the two asymptotes are superposed.

5.3 Comparison of fundamental parameters

The fundamental parameters derived from modelling (E_{ref}, ν, φ, Ψ and pl) are in good agreement with those obtained by triaxial test resuts as well by the classical interpretation of PMT data.

The Figures 9, 10 and 11 shows some

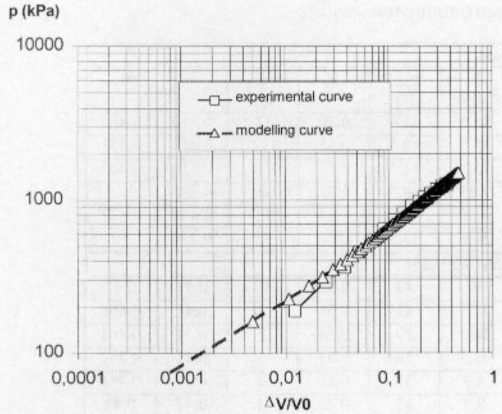

Figure 7. Determination of the limit pressure by lo-log method.

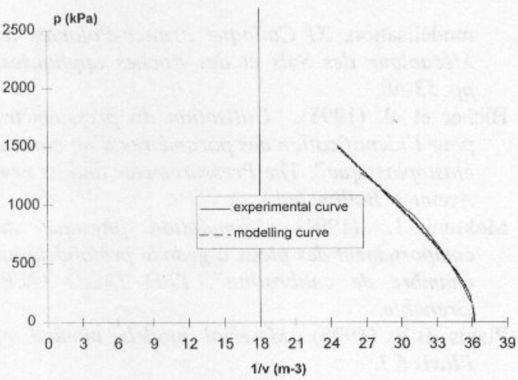

Figure 8. Determination of the limit pressure by the inverse volume method.

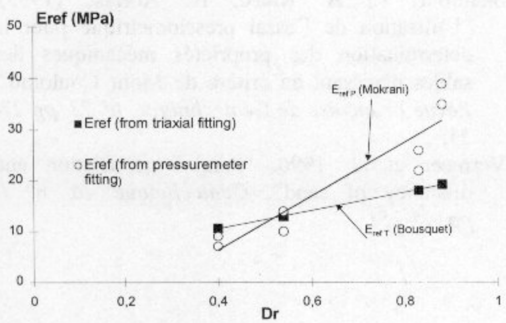

Figure 9. Comparison of Eref moduli obtained by modelling PMT data and by triaxial test data.

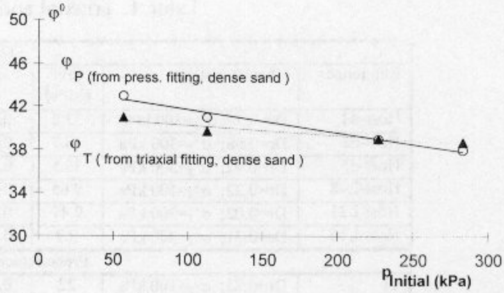

Figure 10. Comparison of peak friction angle obtained by modelling PMT data and by triaxial test data.

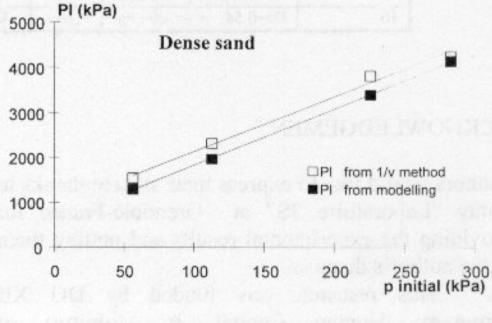

Figure 11. Comparison of limit pressures obtained by modelling and by the 1/v method of PMT data analysis.

comparisons of parametres obtained by different methods.

6. CONCLUSIONS

By using the PLAXIS F.E.M. code it was possible to show that fundamental soil parameters can be found to be in good agreement when they are derived either from triaxial tests or from pressuremeters tests. Dilation is well modelled in both cases, whatever the level of strain, since the model has a limit to dilation corresponding to the critical state with no volume change.

Still the back analysis of the PMT data requests either to have some additional information from laboratory tests or rely on parameter correlations such as the Bolton equation to be finalised.

Table 1. Triaxial and pressuremeter test results

Reference	Characteristics	Eref (MPa)	n	E (MPa)	φ_{peak} (°)	ν	Ψ (°)	n_i	n_{max}
Triaxial test									
Host-d4	Dr=0.77; σ'_3=300 kPa	33.2	0.83	82.6	41.6	0.38	13	0.410	0.456
Host-d2	Dr=0.98; σ'_3=300 kPa	18.7	0.83	46.5	40.5	0.35	11.6	0.410	0.453
Host-d5	Dr=0.92; σ'_3=300 kPa	30.5	0.83	75.9	41.3	0.41	14.6	0.410	0.467
Host-L18	Dr=0.22; σ'_3=100 kPa	9.65	0.83	9.65	34.5	0.22	0	0.470	
Host-L31	Dr=0.02; σ'_3=300 kPa	9.47	0.83	23.6	34.5	0.27	0.5	0.470	0.472
Host-L10	Dr=0.31; σ'_3=300 kPa	9.9	0.83	24.6	33.5	0.20	63	0.470	
Pressuremeter results (K_0=0.35)									
11	Dr=0.83; σ'_3=100 kPa	22	0.45	17	43	0.37	10	0.41	0.47
12	Dr=0.83; σ'_3=200 kPa	26	0.45	27.5	41	0.35	9	0.41	0.468
8	Dr=0.88; σ'_3=400 kPa	35	0.45	50.5	39	0.34	7	0.41	0.461
10	Dr=0.80; σ'_3=500 kPa	39	0.45	62.3	38	0.33	5	0.41	0.458
13	Dr=0.4; σ'_3=100 KPa	7	0.60	4.9	33	0.28	0	0.47	0.485
14	Dr=0.4; σ'_3=200 kPa	9	0.60	9.7	31	0.26	-1	0.47	0.48
15	Dr=0.54; σ'_3=400 kPa	10	0.60	16.3	30	0.24	-2	0.47	0.48
16	Dr=0.54; σ'_3=500 kPa	14	0.6	26.1	29	0.25	-5	0.47	0.48

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