PMT's parameters and one-dimensional consolidation settlements assessment

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ABSTRACT

During past fifty years, more than four millions pressure-meter tests have been conducted in order to allow the construction of the foundations of the bridges of the new network of French motorways and railways and of the numerous buildings erected in France during this period. It has induced great improvements in the understanding of the best use of this test in order to be the most relevant in the design of their foundations. We will present a summary of this important know how, but limiting our presentation to the assessment of one-dimensional consolidation settlement.

RESUME

Durant les cinquantes dernières années, plus de quatre millions d'essais pressiomètriques ont été réalisés pour permettre la construction des fondations des ponts du nouveau réseau français d'autoroutes et de chemins de fer et des nombreux immeubles construits en France pendant cette période. Cela a induit des progrés très importants dans la compréhension de la meilleure utilisation de cet essai afin d'être le plus pertinent possible dans la conception des projets de fondations. Nous présentons un résumé de cet important savoir faire, mais en nous limitant ici à l'évaluation des tassements de consolidation unidimensionnelle.

Keywords: one dimensional consolidation settlements, soils, hard soils and soft rocks, hard rocks.

1. Introduction

Geotechnical engineers have very often to assess one dimensional consolidation settlements under mat foundations, sometimes after a preliminary deep excavation, or under embankments.

We will distinguish three different categories of geotechnical materials: soils, hard soils and soft rocks and hard rocks, in order to show how pressure-meter tests help us to be the more relevant in this approach.

2. Soils. PI* $\leq 1.5 MPa\ (clays) to\ 5\ (gravels)$

Soils are characterized by their grain size distribution, their fines plasticity, their dry density, their void ratio, and their water content. Can be also noticed the presence of salt or of organic matter. We will suppose here that they are absent. For clayey soils, generally limit pressures are lower than 1.5 MPa. For sandy gravelly ones, it can grow up to 5MPa.

2.1. Grain size distribution

Considering a soil, which grain sizes vary from 2 µm to D mm, smaller dimension of the bigger grains, the gradation curve will give us the percent passing in weight at 400 µm, we will write %400µ. It is on this fraction that Atterberg limits will be achieved and that are measured wl liquidity limit and PI plasticity index.

Two situations are then possible: either, the 400µm-D grains are scattered in the 0-400µm fraction or they are in contact with each other's. If they are not, then a consolidation process explains the density of the 0-400µ fraction. We have shown (Gress,2021) that the void ratio e_{0-D} of the 0-D soil was linked with the dry density of the 0-400µm fraction $\gamma_{d0-400\mu}$, the passing %400µ and the density of the grains γ_s , through the relationship:

$$\gamma_{d0-400\mu} = \frac{\%400\mu . \gamma_s}{e_{0-D} + \%400\mu} \qquad (1)$$

Moreover, the volume occupied by the voids e_{0-D} and the 0-400 µm grains must be less than half of the total volume, if 400µm-D grains are in contact. Then:

$$e_{0-D} + \%400\mu \leq 0.5\ (1 + e_{0-D}) \qquad (2)$$

And then: $\%400\mu \leq 0.5\ (1 - e_{0-D})$ (3)

And:

$$\%400\mu \leq \frac{\gamma_{d0-400\mu}}{\gamma_s + \gamma_{d0-400\mu}} \qquad (4)$$

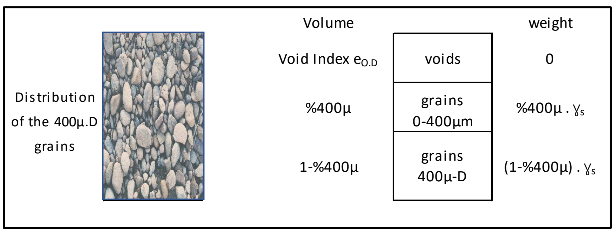


Figure N°1. 400-µ grains in contact.

The figure hereafter shows, in a diagram %400µ as a function of the density of the 0-400µ fraction, three zones.

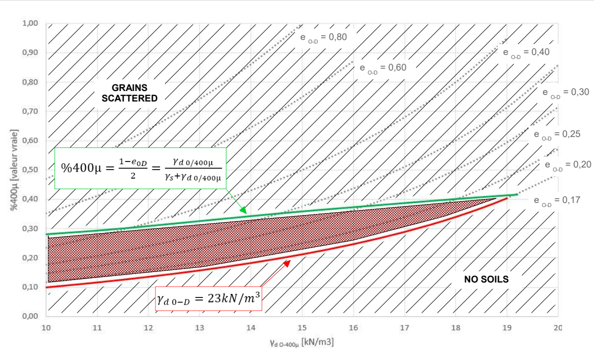


Figure N°2. In grey zone, 400µ-D grains are in contact.

On figure N°2, we notice three zones:

-lower one, no points: γ_{d0-D} has to be less than 23 k N/m3, remembering that:

$$\frac{1}{\gamma_{d0-D}} = \frac{\%400\mu}{\gamma_{d0-400\mu}} + \frac{1-\%400\mu}{\gamma_s} \qquad (5)$$

-intermediate one in grey: 400µ-D grains are in contact,

-upper zone: 400µ-D grains are scattered.

We have also shown (Gress,2012) that when 400µ-D grains are scattered, a consolidation process explain 0-400µ part density and then $\gamma_{d0-400\mu}$ is linked to γ_s, $VB_{0-400\mu}$ methylene blue value of the 0-400µ fraction, σ'_{v0} overburden vertical pressure and σ'_p pre-consolidation pressure through the relationship:

$$\frac{\gamma_w}{\gamma_{d0-400\mu}} = 0.445 + 0.02(1.37 + VB_{0-400\mu})(4.2 - \log\sigma'_{eq})$$

With $\sigma'_{eq} = \sigma'^{\,0.2}_{v0}\,\sigma'^{\,0.8}_{p}$ σ' $in\ kPa$ γ_w water density (6)

When 400µ-D grains are in contact, then $\gamma_{d0-400\mu}$ is probably lower, due to possible arching effects through the 400µ-D grains.

2.2. Plasticity of the fines.

Atterberg limits, wl and PI, and methylene value VB_{oD} are needed. The blue methylene test allows to quantify the quantity VB_{od} of dry blue methylene that coats the internal and external surface of the clayey particles of 100 grams of the 0-d fraction, d being an intermediate diameter of the grains between 2 µm and D.

We can write:

$$VB_{0-D} = \%d.VB_{o-d} = \%400\mu.VB_{0-400\mu} = \%2\mu.\ VB_{2\mu} \qquad (7)$$

For French soils, for a sensivity St less than 4, $VB_{0-400\mu}$ is well correlated for 0-400µ soils to wl and PI through the two relationships:

$$Wl = 0.20.(1-\%2\mu) + 0.063.VB_{0-400\mu} \qquad (8)$$

$$PI = -0.04.(1-\%2\mu) + 0.045.VB_{0-400\mu} \qquad (9)$$

Knowing wl and PI, it is possible to assess the value of the passing at 2µm.

2.3.Boulanger and Idriss susceptibility to liquefaction index S_{BI}

Boulanger and Idriss, (Idriss,2005) working on liquefaction susceptibility, have shown that when the value of 1 -SBI is equal to zero for PI less than 3, the soil behavior was sand like. When it is equal to 1, that is for PI greater than 8, it was clay like. SBI is given by the expression:

$$SBI = \frac{1}{(1+(\frac{PI}{6.4})^8)^2} \qquad (10)$$

When PI value is between 3 and 8, we will consider that soil behavior is still sand like.

2.4. Theory of Janbu, extended by Gress for saturated 0-400µ soils

We will consider that soils are saturated when:

$$w_{ret} \le w_n \le w_{sat}$$

Water content between water retention level and that at saturation.

Janbu (Janbu,1967)) have then proposed that one dimensional settlements could be assessed through the relationship:

$$\frac{d\epsilon}{d\sigma} = \frac{1}{m\sigma r}\left(\frac{\sigma_r}{\sigma'}\right)^{(1-j)} = \frac{1}{M_t} \qquad (11)$$

m = Janbu modulus; j = stress component; σ_r =100 kPa ; σ = strain; ε = deformation; Mt tangent modulus.

To extend this proposal, we suggest to write :

$$j = SBI \qquad (12)$$

When grains 400µ-D are in contact one with each other's, we suggest to consider that these soils belong to category 1.

If on the contrary, 400µ-D grains are scattered, then the 0-400µ fraction belongs to category 1, if PI is less than 3, to category 2 if PI value is between 3 and 8 and to category 3 if PI is greater than 8.

2.5. One dimensional consolidation settlement of saturated categories 1 and 2 soils.

Massarsch (1994) Menard (1958) Gress (2019)

2.5.1. Soil category 1: 400µ-D grains in contact or if not, PI of the 0-400µ fraction less than 3.

We have: 1 – j = 0 and then:

$$\varepsilon = \frac{1}{m\,\sigma_r}\ \Delta\sigma \qquad (13)$$

2.5.2. Soil category 2: 400µ-D grains scattered and PI value between 3 and 8.

For example, when PI =4.5, then j = SBI = 0.5 and:

-if $\sigma'_v \ge \sigma'_p$:

$$\varepsilon = \frac{1}{5m_r}(\sigma'^{\,0.5}_{p} - \sigma'^{\,0.5}_{v0}) + \frac{1}{5m}(\sigma'^{\,0.5}_{v} - \sigma'^{\,0.5}_{vO}) \qquad (14)$$

-if $\sigma'_v \le \sigma'_p$:

$$\varepsilon = \frac{1}{5m_r}(\sigma'^{\,0.5}_{v} - \sigma'^{\,0.5}_{v0}) \qquad (15)$$

m: modulus number (dimensionless)

mr recompression modulus number often five to twelve times greater than m.

For the assessment of σ'_p, we have shown (Gress, 2012) that for normally consolidated soils , we had the relationship:

$$\frac{10}{\gamma_{dNC}} = \frac{10}{\gamma_s} + 0.075 + 0.315(\mathrm{Wl}-0.075)(4.2 - \log \sigma'_{v0}) \quad (16)$$

And that for over-consolidated soils, we had the same with γ_{dSC} instead of γ_{dNC} together in k N/m3 and σ'_{eq} instead of σ'_{v0} together in kPa.

Then, working on these two relationships, we can write:

$$\log\left(\frac{\sigma'_p}{\sigma'_{v0}}\right) = 39.68 \frac{\frac{1}{\gamma_{dNC}} - \frac{1}{\gamma_{dSC}}}{wl - 0.075} \quad (17)$$

Massarsch (Massarsch,2019) has proposed a relationship, giving the value of m, knowing the cone tip resistance qt during a static cone penetration test:

$$\mathrm{m} = \mathrm{a} \frac{qt^{0.5}}{(\sigma_r \, \sigma'_{v0})^{0.25}} \quad (18)$$

qt, σ'_{v0} in kPa, σ_r = 100 kPa and a being an empirical parameter, function of soil nature and it's compacity.

Gress et al (Gress,2021) have recently proposed the expression hereafter for a:

$$\mathrm{a} = \frac{3}{2} \frac{\sigma'_{v0}{}^{0.25}}{\alpha_M \sigma_r{}^{0.75}} qt^{0.54} \text{ in kPa} \quad (19)$$

fitting with the values proposed by Massarsch, α_M being the rheological parameter proposed by Menard. It varies from 0.33 to 0.61 through the relationship:

$$\alpha_M = 0.33\ (2 - \mathrm{j})\ . \quad (20)$$

A comparison is given hereunder between' a 'calculated by expression (19) for σ'_{v0} = 100 kPa and values suggested by the Canadian Foundation Engineering Manual (CFEM):

Soil type	compacity	qt kPa	α_M	a Gress	a CFEM
Silts	Loose	1000	1/2	12.5	12
	Compact	2000	0.5	18.2	15
	Dense	3000	0.5	22.6	20
Sands	Loose	1500	1/ 2.5	19.5	22
	Compact	3000	0.4	28.3	28
	Dense	5000	0.4	37.3	35
Gravels	Loose	2500	1/3	31.1	35
	Compact	5000	0.33	45.2	40
	Dense	10000	0.33	65.7	45

Table N°1. Comparison between the two ''a'' determinations.

Clays are analysed in paragraph 2.7.

Then we can write m σ_r as:

$$\mathrm{Mt} = \mathrm{m}\ \sigma_r = \frac{3}{2\alpha_M} qt^{1.04} \quad (21)$$

Moreover, we have noticed (Gress,2019) a good correlation between qt and Menard PMT parameters, that is pl*, net limit pressure and E_M, Menard modulus, given by the relationships here after:

$$\mathrm{qt} = (pl*)^{1.25} \text{ in kPa} \quad (22)$$

$$E_M = (pl*)^{(1+\alpha_G)} \text{ in kPa} \quad (23)$$

$$\text{With } \alpha_G = 0.4 \frac{(VB_{0-D} + 0.7)}{(VB_{0-D} + 1)} \quad (24)$$

VB_{0-D} varies from 0.5 to 2.67 and then α_G from 0.32 to 0.367.

Expression (21) can then be written:

$$\mathrm{m}\ \sigma_r = \frac{3}{2\alpha_M} (0.95 \text{ to } 0.985)\ E_M \quad (25)$$

Knowing that, K being the bulk modulus:

$$\mathrm{Mt} = \frac{(1-\vartheta)}{(1+\vartheta)(1-2\vartheta)} \mathrm{Et} \text{ and } \mathrm{K} = \frac{Et}{3(1-2\vartheta)} \quad (26)$$

we can write with $\vartheta = 0.33$:

$$3\mathrm{K} = \frac{1+\vartheta}{1-\vartheta} M_t \cong 2\ M_t \text{ (Cordary, 1981)}$$

When 1-j = 0, then Mt = m σ_r and the value of K becomes:

$$\mathrm{K} = \frac{E_M}{\alpha_M} = \frac{qt^{1.04}}{\alpha_M} \text{ in kPa with } \alpha_M = 0.33\ (2 - j) \quad (27)$$

relationships working for 0-D soils.

Distributed loads at soil surface on a large area induces one-dimensional deformation given by:

$$\varepsilon_{zz} = \frac{\sigma_m}{K} \text{ with } \sigma_m = \frac{(\sigma_{xx} + \sigma_{yy} + \sigma_{zz})}{3} \quad (28)$$

2.6. One dimensional consolidation settlement of unsaturated 1or 2 soils categories.

When working on unsaturated 1 or 2 soils categories, we must be aware that measured parameters in an unsaturated state must be corrected by their water-content, if we want to anticipate their values in a saturated state, knowing the relationship (Gress,2018):

$$\frac{*2}{*1} = \left(\frac{W_{n1}}{W_{n2}}\right)^n \quad (29)$$

*i being the parameter value in a water-content w_{ni},
And with:

$$\mathrm{n} = \frac{3VB_{0D}{}^{2.8}}{e^{VB_{0D}}} + 0.667\ VB_{0D}{}^{0.12} \quad (30)$$

Unfortunately, this correction is very rarely put forward and implies that in-situ geotechnical tests should be always doubled by density and water-content measures.

2.7. One dimensional consolidation settlement of saturated 3 soils category.

Here 1 – j = 1

2.7.1.Soils with dimensions of grains less than 400µm

The assessment of settlements is here conducted using relationships:

If $\sigma'_v > \sigma'_p$

$$\varepsilon = \frac{1}{m_r} \mathrm{Ln}\left(\frac{\sigma'_p}{\sigma'_{v0}}\right) + \frac{1}{m} \mathrm{Ln}\left(\frac{\sigma'_v}{\sigma'_p}\right) \quad (31)$$

$$\text{With: } \mathrm{m} = \mathrm{Ln}10 \frac{1+e_0}{C_c} \text{ and } m_r = \mathrm{Ln}10 \frac{1+e_0}{C_s} \quad (32)$$

If $\sigma'_v \leq \sigma'_p$

$$\varepsilon = \frac{1}{m_r} \mathrm{Ln}\left(\frac{\sigma'_v}{\sigma'_{v0}}\right) \quad (33)$$

Where: σ'_v final effective stress in kPa, σ'_p preconsolidation pressure in kPa, σ'_{v0} initial effective stress in kPa, m Janbu modulus (dimensionless) and m_r recompression modulus number (dimensionless).

Parameters C_c, C_s and σ'_p are usually measured through oedometer tests.

Two major difficulties are:

- The quality of the said intact samples
- The limit of the maximum acceptable value of the dimension of the grains in the oedometer box fixed at 3mm ($\frac{19}{6}$).

It is then interesting to use some relevant correlations as: $C_c = 0.9\,(\mathrm{wl} - 0.1)$ (Terzaghi, 1967) (34)
wl for it's real value, not in percent,

$$C_c = 2.7\,\frac{w_{sc}-0.075}{4.2-\log({\sigma'_{v0}}^{0.2}\ {\sigma'_p}^{0.8})} \qquad (35)$$

w_{sc} water content for it's real value, in the overconsolidated state, due to Gress,(Gress,2012) derived from Herrero. We can also assume:

$$C_s = 0.2\,C_c \qquad (36)$$

Preconsolidation pressure is also a very difficult parameter to correctly measure. It is interesting to compare the laboratory results to the correlations hereafter (Gress,2019)):

${\sigma'_{v0}}^{0.2}{\sigma'_p}^{0.8} = (qt - \sigma_v)^{0.8\,m} = pl*^m$ in kPa (37)

m Mayne factor equal to:

$$\mathrm{m} = 1 - 0.28\frac{1}{1+(\frac{I_c}{2.65})^{25}} \qquad (38)$$

Where I_c Robertson behaviour index is either given by the CPT'u parameters or estimated by the relationship due to Gress (Gress,2019):

$$I_c = 3.6\,\frac{wl-0.115}{wl-0.025} \qquad (39)$$

The value of settlement is calculated at the end of primary consolidation at a time t_p.

Finally creep has to be taken into account through the additional value given by the hereafter relationship:

$$\varepsilon_{zzc} = \frac{C_\alpha}{1+e_p}\log\frac{t_f}{t_p} \qquad (40)$$

C_α Creep index very often taken equal to $0.04C_c$
e_p void ratio at the end of primary consolidation,
t_f taken equal to 10 years, t_p in years.

2.7.2. 0-D Soils (grain size distribution from 2µm to D mm).

For a 0-D soil, we can write:

$e_{0-400\mu} = \frac{e_{0-D}}{\%400\mu}$,or: (41)

$e_{0-D} = \%400\mu\ .\ e_{0-400\mu}$ (42)

And then:

$\Delta e_{0-D} = \%400\mu.\ \Delta e_{0-400\mu}$ (43)

This implies that:

$Cs_{0-D} = \%400\mu.\ Cs_{0-400\mu}$ (44)

$Cc_{0-D} = \%400\mu.\ Cs_{0-400\mu}$ (45)

And:

${\sigma'}_{p0-D} = {\sigma'}_{p0-400\mu}$ (46)

Where C_s swelling index, C_c compression index and σ'_p preconsolidation pressure.

2.8. One dimensional consolidation settlement of 0-400µ unsaturated 3 category

For soil category 3, Fredlund (Fredlund, 1993) relationship is interesting taking into account total vertical pressure σ_v and suction variations as:

$$\varepsilon_{zz} = C_r\ \log\frac{\sigma_{vf}+s_f}{\sigma_{vi}+s_i} \qquad (47)$$

the difficulties here being to correctly evaluate s_i and s_f, initial and final suctions, and C_r. For the latter, the best way is to compare the behavior of the sample through a shrinkage sample test and through a soil water retention test. But, generally, we assume $C_r = C_s$.

With Fredlund relationship, we will calculate either settlement or heave.

2.9. Conclusions for soils

For 1 and 2 categories of soils, CPT'u and pressuremeter tests will be good tools to assess one dimensional consolidation settlement in a saturated state. For unsaturated states, we will have to know water contents compared to saturation in order to correct the calculated values. For category 3, only oedometer tests will give relevant parameters.

3. Hard soils and soft rocks 1.5 $< Pl* \leq$ 4 Mpa. (Guilloux, 2005)

In this family, we have marls, overconsolidated clays, chalk, cemented sands, salts, silty sandstones, for example. Some authors have tried to correlate Vs, shearing waves velocity, with Menard modulus, letting believe that E_M is a reliable parameter.

It is not the case, because this type of materials are causing four types of difficulties, we must know. They are prone to swelling, they can creep, they can collapse and they can be sensitive to attrition.

They will very often swell submitted to a decrease of the overburden pressure. The swell may be mechanical and chemical. If swelling is not able to develop, then heavy stresses will be generated.

For dams being build along the Rhone river, needing preliminary deep excavations, 25 cm heaves have been noticed in stiff marls. (Cambefort, 1983)

In the eastern part of France, 3 meters heave have been noticed in the basement of the Magasins Reunis shop in Nancy, due to cardboard schists. (Cambefort, 1983)

Chalk can creep under heavy stresses.

In Le Mans town, eolian stiff cemented by carbonates loess, but having low densities, can collapse due to water infiltrations around the buildings.

In Meaux, deep artesian groundwater table is washing the fine of stiff sands, due to upwards induced flows. Menard net limit pressure is greater than 4 MPa, but Vs is less than 200 m/s and dry density around 18 k N/m3.

Finally, these soils, though being apparently stiff, can show high permeability values, and in the case of Meaux for example: 10-3 to 10-4 m/s.

As a conclusion for this family, we will stress on the fact that they need adapted geotechnical tests in order to characterize specific behaviours, the pressuremeter tests being not able to highlight.

For stable hard soils or soft rocks, we will have to use rock mechanics tests like for hard rocks.

4. Hard rocks. PI* ≥ 4 MPa

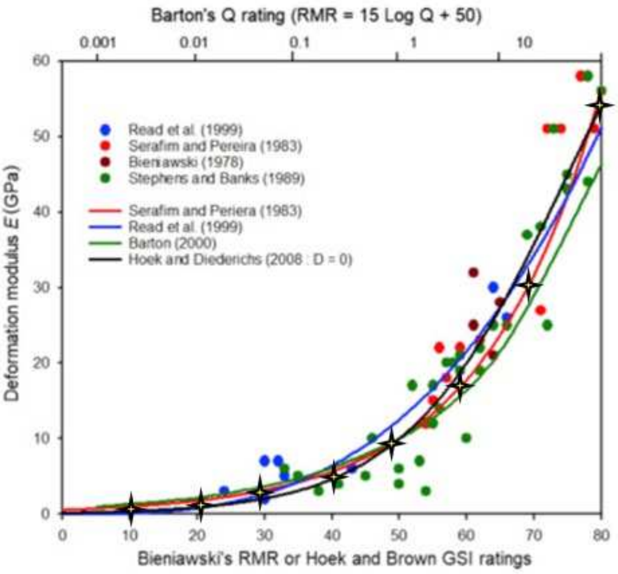
Hard rocks represent a family for which pressuremeter tests meet their limits due to the apparatus design. It is rare to measure Menard modulus greater than 500 MPa, even when the rubber probe is used. When we use the metallic protection, it is worse, the probe being too stiff then to shape the wall of the borehole, the volume of the probe at low pressure being then underestimated.

Working on rock deformation modulus, the chart correlating Emr to GSI can be represented by the relationship (Gress,2023), see figure N°3:

$$E_{mr} = 24.47\left(\frac{GSI+3.54}{136.20-GSI}\right)^2 \text{ in GPa} \quad (48)$$

For hard rocks GSI is greater than 75 and then E_{mr} is greater than 40 GPa, that is 40 000 MPa.

Here we will need to use rock mechanics tests in order to be the more relevant possible.



: Relationship 48.

Figure 3. Comparison between four authors approach and Gress Relationship.

5. General Conclusion

The pressuremeter apparatus has allowed french geotechnical engineers to make during the past fifty years numerous relevant designs of bridges and buildings foundations. Nevertheless, It is clear through what has been shown here, that a general rule for geotechnical studies is to mix different geotechnical tests, one test being not able to reveal the complexity of soils and rocks behaviors, but the relevance coming from the confrontation of different approaches.

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