Pressuremeter Testing and its Applications[1]

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The pressuremeter test is a static loading test carried out in the ground in situ, during which the expansion of the borehole wall is measured as a function of the pressure applied by an expandable cylindrical probe.

Meaning of this test and the use in geotechnical design of the simple parameters derived from the analysis of this test were thoroughly investigated by Louis Ménard and his co-workers, These primary results are used on the basis of rules developed from theory and observation on full scale foundations (Ménard, 1962a, 1963). More recently, the target of academic research work has been to derive soil fundamental parameters from this test. It is demonstrated that soil rheological parameters can be obtained either analytically (Baguelin et al., 1972) or by using the inverse method technique (Cambou et al., 1989).

Another special feature relating to the pressuremeter is that of the influence of its implementation, even within the context of a standardised test (ISO 2010). Most of the times, the pressuremeter probe is lowered in pre-bored holes: any remoulding of the borehole wall must be minimized and the time lag between borehole drilling and testing must be shorten to reduce any stress relaxation around the cavity to avoid alteration of the results.

To reduce these variables, the concept of the self-drilling pressuremeter was born [BAG 73]. There are also pressuremeters which have probes that are sunk into the soil by ramming [AMA 83] or screwing (for example, with the help of a static penetrometer frame). Lastly, there are pressure measurement devices designed for rock, which are

generally called dilatometers within the context of boreholes, the implementation of which and the breakdown of the results can be done in the same way as for the Ménard pressuremeter [SIM 87].

[1] This paper is based on Chapter 4 of the text book "Reconnaissance des terrains in situ" edited by I. Shahrour and R. Gourves, 2005, Hermes Publisher, Paris

4.1. Introduction

It was in the 1930s that the first known tests were conducted on soil deformation in the walls of a borehole placed under a pressure strain [KOG 33] using cylindrical probes, either equipped with two rigid curved metal half-shells, or consisting of a rubber envelope.

But it was Louis Ménard [MEN 55] who was the true pioneer of this technique, having filed a patent while he was still an engineering student at the *École Nationale des Ponts and Chaussées* (French National Civil Engineering School]. The history of the fine-tuning and development of the Ménard pressuremeter has already been given elsewhere [GAM 90].

Having quickly observed that obtaining what are referred to as the soil's intrinsic parameters was impossible using the calculation tools that existed at that time, Louis Ménard oriented himself towards using primary results (pressure measurement modulus, pressure measurement limit pressure) in schemas for modelling the behaviour of soil in relation to a foundation compared to that of the expansion of the pressure measurement probe [MEN 62b, 63].

Moreover, from the late 1960s onwards, the LCPC (French National Civil Engineering School Central Laboratory) in Paris launched research with a view to overcoming the variables arising from the probe implementation technique. The result of this was the PAF model of self-drilling pressuremeter [BAG 73], which was quickly followed in Great Britain by the Camkometer [WRO 72] initially designed to measure Ko (as the name indicates) and which subsequently became the *self-boring pressuremeter* or SBP.

The various types of pressuremeters used in soils and their geographic distribution in Europe in 1990 were described by a report No.4 on the state of knowledge by the European Regional Technical Commission of the International Society of Soil Mechanics and Foundation Works [AMA 91], with the exception of those consisting of two rigid half-shells and probes sunk directly. The development of the use of the pressuremeter in Japan since the 1950 has been described elsewhere [MOR 95].

Pith a view to estimating the development of soil deformation *in situ* over the long term, a pressuremeter for measuring the soil's creep was also developed; the Diflupress L.D. [BUF 90]. The current trend is oriented towards apparatuses for the electronic acquisition of measurements and computerised output enabling processing on an office computer [GAM 95a].

In the rest of this presentation we will limit ourselves to dealing solely with the Ménard pressuremeter, the only one used intensively on a day-to-day basis, whilst indicating the main differences with the other pressuremeters, whenever these arise.

The Ménard pressuremeter can be used to characterise the majority of soils (from soft clay to stiff marl) and soft rocks. Tests at depths of over one hundred metres are commonly conducted as there is no risk of rejection. The tests can be executed on land, or at a marine or aquatic site from a floating platform. The design of the Ménard pressuremeter enables deformation moduluses of 2 to 500 MPa (for relative deformations between 10^{-3} and 10^{-2}) to be measured and limit pressures of 300 kPa to 10 MPa to be measured.

4.2. Description of the apparatus

As indicated, what is involved is determining a strains/deformations function for the soil in a borehole's wall. You can measure the probe's deformations either volumetrically, or using radial deformation sensors. The latter generally only enable maximum expansion measured over a much smaller radius, namely 5 to 20 %, as opposed to 54 % for volume measurements (Table 1).

The Ménard pressuremeter [AFN 00] basically includes (see Figure 4.1):

– A cylindrical probe surrounded with a dilatable envelope; this probe must be quite thick so that it can simulate the expansion of a cavity of infinite length [AJJ 98, HAR 74, LAI 73]; it is lowered into a borehole drilled beforehand, to a depth set in advance;

– A pressure/volume controller enabling pressure to be applied in stages, and enabling the increase in the volume of the probe's measuring cell to be measured as a function of the time;

– A tube linking the two aforementioned components.

Class	**Type (country of origin)**	**Maximum Pressure (MPa)**	**Thickness of the Probe D/L**	**Expansion Measurement System**
Pressuremeters with preliminary borehole drilling	Ménard (France, Canada)	6	8–16 *	Volumetric
	Texam (Canada)	10	7	Volumetric (imposed)
	LLT (Japan)	2.5	8-10 *	Volumetric
	Elastmeter (Japan)	10 or 20	8	3 sensors on a single plane
	HPD (Great Britain)	20	6	3 x 3 sensors on 3 planes
	Mazier Dilatometer (France)	10 or 20	11	3 sensors on a single plane
	Tri-mod (Canada)	10	6	6 sensors on 2 planes
	Probex (Canada)	30	6	Volumetric
Pressuremeters (self-boring)	PAF (LCPC, France)	2.5	2	Volumetric (imposed)
	SBP (Great Britain)	4	6	3 sensors on a single plane
	Boremac (Canada)	4	5	Volumetric
Pressuremeters (that are sunk into the ground)	Minipressuremeter (France)	2.5	10	Volumetric
	Pencel (Canada)	2.5	8	Volumetric (imposed)
	Fugro (Netherlands)	10	10	3 sensors on a single plane

* Depending on the diameter of the probe

Table 4.1. *Example of existing pressuremeters [AMA 91]*

4.2.1. *The cylindrical probe*

This probe comes in three different diameters, and is designed to be lowered into boreholes with a diameter of AX, BX, or NX (DCDMA standardisation system) or in other words 44, 60 and 76 mm in diameter. The ratio between the length of the deformable portion of the probe and its diameter is about 7 to 10 for Ménard pressuremeter probes. The probe consists of three cells: a central cell filled with water, and two cells inflated with gas. This arrangement enables expansion measurements to be conducted within a strictly radial field of strains, as well as eliminating the effects of edges, and avoiding excessive variations in the volume to be measured.

Depending on the type of probe, the cells' envelopes are juxtaposed (E probes) or nested (G probes). Each cell's envelope is referred to as a "membrane" and probe's envelope is known as a jacket. Membranes and jackets are made of synthetic rubber; the jackets of G probes can be reinforced with metal or textile fibres, or using supplementary protection made of flexible metal strips which are partially superimposed. If necessary, the probe is protected by a lantern-pattern tube made of thick metal split into at least six generatrixes or quarter helices. E probes only enable tests up to 2.5 MPa [MEL 71]; to the contrary, G probes enable 10 MPa to be achieved. With certain probes other than the ones used by the Ménard pressuremeter (including borehole dilatometers) the expansion can be measured using electronic feelers; a pressure of 20 MPa can be achieved.

4.2.2. *The Pressure Volume Controller (P.V.C.)*

This monitoring box enables the pressure in the three cells to be regulated and the variations in the volume of liquid in the central cell in the drill head to be measured using a metal cylinder acting as a graduated chamber. The P.V.C. is linked upstream to a compressed gas source (generally nitrogen; oxygen must not be used) and downstream to the connection tube with the probe.

The pressure measurements in the central cell and in the guard cells are carried out using traditional pressure gauges or more precise electronic pressure sensors which can cover the whole range of pressures used (10 kPa to 10 MPa). Additional pressure gauges can be installed to ensure sufficient accuracy of the results. The pressure variations are adjusted by a main regulator (possibly a controlled leakage model to ensure better pressure stability during each stage) and a control valve for ensuring the essential pressure difference between the central cell and the guard cells so that, among other things, the existence of additional hydrostatic pressure in the central cell can be taken into account. This arrangement can be replaced by a set of automatically controlled solenoid valves.

The volume measurements are carried out using an indicator along the length of the volumeter which, in the case of digital data entry, is done using a physical system in the volumeter.

4.2.3. *The connection tube*

This includes either three plastic tubes (E probe), or two more rigid plastic tubes, which are possibly coaxial (G probes). These tubes must have a low expansion coefficient which enables good accuracy to be obtained in terms of the results.

Figure 1 shows a P.V.C. monitoring box with traditional pressure gauges and a reading indicator. This apparatus is often accompanied by an automatic recorder for the readings obtained from pressure and volume sensors.

4.2.4. *The other designs.*

The LCPC's PAF self-boring pressuremeter has a single-cell probe preceded by a drilling module. The control and measuring box is activated by movement of a piston in the volumeter filled with water [BAG 78]. The Cambridge pressuremeter (SBP) also consists of a single-cell probe fitted with a preliminary borehole drilling system. The membrane's radial deformations are measured using several metal feelers. The pressurisation programme is servoed to expansion of the probe measured radially [BEN 86].

Borehole dilatometers have a probe with a sleeve that can be dilated using a gas or liquid. The sleeve's radial deformations are measured using three sensors in the central part [AFN 02].

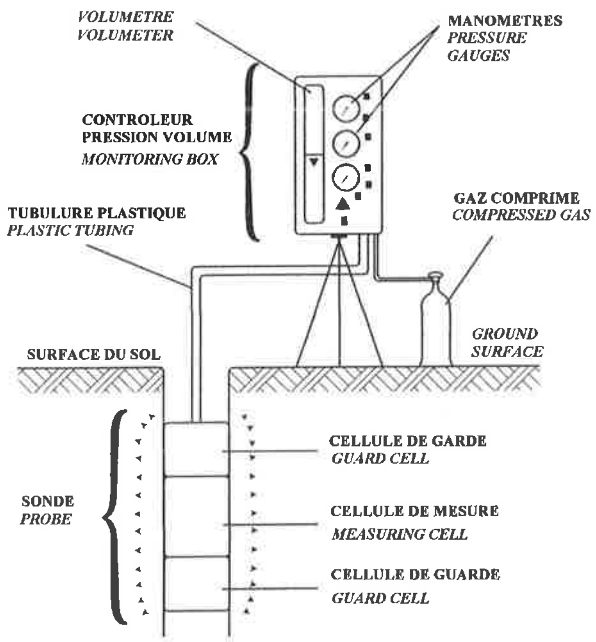


Figure.4.1. *A Ménard pressuremeter*

4.3. Operating procedure

The tests done using a Ménard pressuremeter are static tests with imposed strains, while tests using the self-boring pressuremeter were initially done using imposed deformations; currently they are carried out using a mixed procedure.

Before any new series of tests, you need to carry out double calibration in relation to the specific resistance for the probe's envelopes (membrane + jacket) and the specific expansion for the apparatus as a whole (controller + tube + probe). Should feelers be used, a measurement of the thinning of the probe's envelope as a function of expansion may be necessary. The soil's raw response curve will have to be corrected for erroneous values, as indicated below [AST 00, AFN 00].

4.3.1. *Specific resistance of the Ménard probe*

Determining the specific resistance of the Ménard probe is carried out with the probe placed at the level of the P.V.C. and consists of carrying out an expansion test in the open air. For each pressure p applied in stages up to the specific limit resistance for the probe (see Figure 4.2), you measure the volume V that the cell assumes upon stabilisation. You can thus trace the curve for the probe's specific resistance as a function of the volume introduced $p = p_e(V)$.

4.3.2. *Specific expansion of the Ménard pressuremeter*

The apparatus's specific expansion is measured using the probe mounted with the tube and the P.V.C. monitoring box which are to be used during the tests envisaged. The probe is placed in a thick metal tube with a diameter slightly larger than the probe and which is deemed to have negligible dilation up to 10 MPa. For each pressure p applied, you measure the volume V of the probe (see Figure 4.3). This allows you to determine the conventional no-load volume V_S for the probe which is involved in calculating the pressure measurement parameters and the volumetric expansion coefficient "*a*" for the whole apparatus in cm^3/MPa.

4.3.3. *Putting the Ménard pressuremeter's probe in place*

The Ménard pressuremeter's probe is generally placed in a borehole with a diameter scarcely larger than the diameter of the probe when at rest. With a view to reducing remoulding of the walls of the borehole to a minimum, and thus carrying out the test on ground that is as close as possible to its initial state, drilling techniques are recommended depending on the type of soil being surveyed [AFN 00].

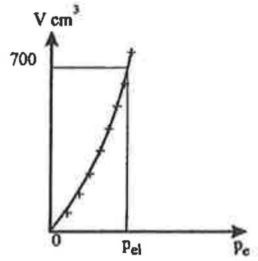


Figure 4.2. *Specific resistance curve for a Ménard probe*

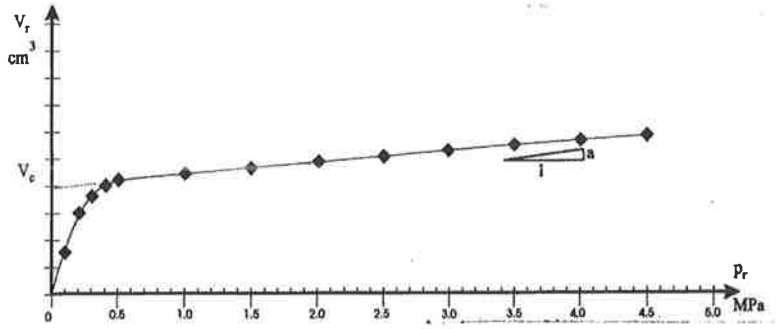


Figure 4.3. *Specific expansion curve for a Ménard probe [AFN 00]*

The probe's volume under no-load conditions is $V_s = 0.25\pi d^2 - V_c$ where d is the diameter of the calibration tube and V_c is the ordinate at the origin of the adjustment line for P > 500 KPa.

In submersed sand and gravel with which there is drilling sludge that does not enable the borehole walls to be kept stable, it is possible to ram the assembly comprising the probe and a thick, split metal tube in accordance with several generatrixes enveloping it. Possibly, firstly you may drill a pilot hole with a smaller diameter. Experience has shown that the results obtained can be broken down according to the same rules.

When you use a probe that is rammed or sunk into ground other than submerged sand or gravel, the results will only come close to the standardised tests if the ram head is of a diameter that is larger than the probe's diameter so that it enables minimum relaxation of the borehole wall.

In the case of a test involving a self-boring pressuremeter, you need to keep the soil intact located on the vertical virtual cylinder extending from the probe's cutting curb. This means you have to position the disaggregation tool very precisely in the drilling module's metal cavity [WRO 82]:

– If the tool is too high in relation to the edge of the cutting curb, the soil will clog and the virtual cylinder will dilate,

– If the tool is too low, the soil will become decompressed and the virtual cylinder will contract.

In both cases, the soil will have already been subjected to an initial test drilling beforehand.

4.3.4. *Performing a Ménard pressure measurement test*

As the pressure measurement test is a true standardised loading test, you need to follow certain rules.

4.3.4.1. *Differential pressure adjustment*

As the probe has to be lowered to a depth Z, you need to perform the differential pressure adjustment on the P.V.C. whilst taking the following into account:

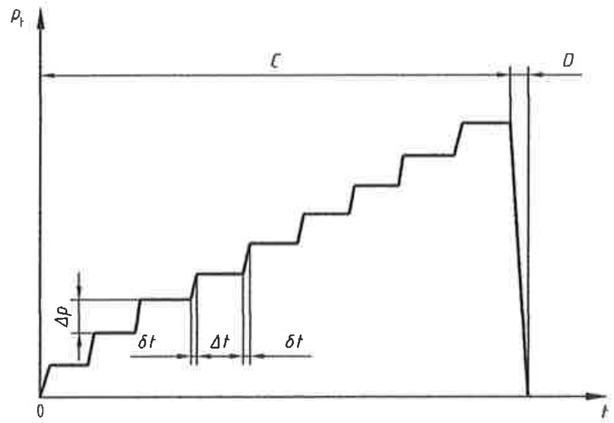
– The compensation in the guard cells for hydrostatic pressure $\gamma_w Z$ which only predominates in the central cell and which cannot be measured by P.V.C.;

– For G probes (nested cells), the excess pressure that needs to be applied to the compressed central cell between the spaces materialising the guard cells so that it can dilate and maintain a constant length when the pressure increases.

This adjustment is carried out using the differential pressure valve (or a set of solenoid valves), as specified in the standard [AFN 00], Appendix B.

4.3.4.2. *The test itself*

The programme for a test complies with the function p(t) given in Figure 4.4 [AFN 00]. The pressure-deformation curve is obtained by placing pressure on the soil in stages involving an arithmetic progression, with each stage being maintained for 60 seconds. For each stage, you measure the volumetric deformations V at 1 second, 15 seconds, 30 seconds and 1 minute after having obtained the pressure set by operating the main relief valve or via remote control of the set of solenoid valves. The raw pressure measurement curve is obtained by connecting the points for each pressure p and each volume $V_{60\ sec}$ (Figure 4.5).



- p: Pressure

- δt: Pressure increase time
- T: Time
- C: Loading phase
- Δt: Stage duration
- D: Unloading phase
- $\Delta p = 0.1$ pl
- $\delta t < 20$ seconds
- $\Delta t = 60$ seconds

Figure.4.4: *Programme for a Ménard pressure measurement test [AFN 00]*

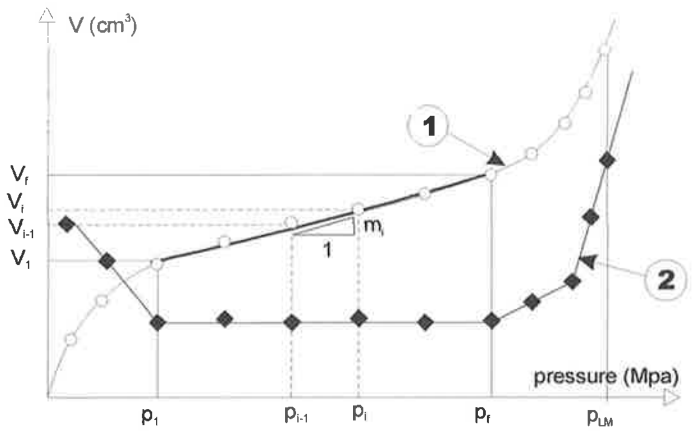


Figure 4.5. *Ménard pressure measurement curve. 1) Pressure measurement curve 2) Creep curve (ordinates scale enlarged)*

The corrected pressure measurement curve is obtained by deducing the probe's specific resistance for the same volume from each pressure p_r read, and by deducing the expansion of the apparatus equalling a x p_r (with a being the apparatus's expansion coefficient) for each volume V:

– $P_{corrected} = p_r - p_e (V)$

– $V_{corrected} = V - a\,p$

As the pressure measurement test is carried out subject to imposed strains, the pressures are entered as abscissas and the volumes as ordinates, which is the opposite of most mechanical tests (which are subject to imposed deformations for reasons of convenience). Of course, when tests with other types of pressuremeters are carried out subject to imposed deformations, the co-ordinates axes are reversed. On the graph for the Ménard pressure measurement curve, the creep curve is frequently indicated, or in other words, for each pressure stage, you draw the stabilisation trend for the volume variations represented by the value $V_{60sec.}–V_{30sec.}$

The range p_1-p_2 in which you calculate the pressure measurement modulus can be defined as matching the near-horizontal part of the creep curve, limited at the upper end by the creep pressure or around the segment of the pressure measurement curve with the least slope. You can also use the double-hyperbolic method, which also allows the representativeness of the test to be assessed [BAU 92, VAN 79].

4.3.4.3. *Cyclic test*

This type of test enables a cyclic deformation modulus to be determined. In the French standard [AFN 99], you only calculate the Ménard modulus upon reloading. The same applies for the borehole dilatometer [AFN 02]. For other apparatuses, you calculate the average unloading/reloading modulus.

Tests using a flexible-wall borehole dilatometer are always carried out using loading/unloading cycles [AFN 02]. These cycles are also recommended for tests involving the self-boring pressuremeter [CLA 92].

4.3.4.4. *Non-standardised tests*

What is involved are mainly tests involving a self-boring pressuremeter. More often than not, the measurements are carried out subject to imposed deformations, however to obtain greater accuracy with smaller deformations, often imposed strains are used.

On the graph for a curve for a PAF in clay, you also draw the shear strength curve estimated using several methods [BAG 78]. On the graph for an SBP curve, you will generally notice one or more loading and reloading cycles [WRO 82]. These tests are in the process of being standardised at European level, as is the case for the test for a borehole dilatometer (see section 4.5.2)

4.4. Interpretation of the measurements

The goal of a pressure measurement test is to provide mechanical characteristics enabling either the sizing of civil engineering works to be carried out in a subsequent phase, or research to be conducted with a view to refining our knowledge about the behaviour of soils when placed under pressure. In the first case, you can settle for simple parameters, while in the second case, you have to identify the parameters for the soil's rheological behaviour.

The pressure-deformation relation obtained during a pressure measurement test must allow these parameters for the soil to be defined based on various theories according to the hypotheses adopted for the expansion of a cylindrical cavity:

– In a linear, elastic medium [LAM 52], without seeking to interpret the breaking phase,

– In an elastoplastic medium:

- Without a change in volume

-For a consistent soil [BIS 45],

-For soils that are abrasive and consistent [MEN 55, MEN 56],

- Or with a change in volume, in the elastic phase only [HIL 50] or also in the plastic phase [MON 94, 95b; SAL 66; VES 72],

– In a non-elastic medium without a change in volume [BAG 72, LAD 72, PAL 72]),

– In a dilating linear medium [FRA 74],

– In the general case, with a change in volume [LAD 61, WRO 75] and dilatancy [HUG 77],

– In a coherent, with a remoulded ring [BAG 72, 75].

Complementary cases are developed in [BAG 78], again with no reference to the role of the time factor. A study of the interpretation of the results obtained using a probe sunk into the ground was carried out for sand [HUG 85] and for clay [HOU 88]. Other cases were analysed in the proceedings of the last two international conferences on pressuremeters: ISP3 at Oxford [BGS 90] and CIP4 at Sherbrooke, Quebec [BAL 95]. Lastly, a review of the theories used for making use of tests on soft rocks and cemented soils was given [HAB 97] which includes the phenomena of cleaving or opening up fissures.

4.4.1. *Interpretation of a Ménard pressuremeter test*

This interpretation is carried out at the simplest level [AFN 00]. To determine a deformation modulus E_M, it is assumed that the soil is governed by the law of linear elasticity. To determine a breakage characteristic p_{LM} without involving hypotheses about the soil's behaviour in terms of plasticity, reasoning is used similar to what is adopted for determining the breakage of the soil under a plate subjected to a loading test.

Manufacturers of apparatuses and users have developed software for breaking down the test results in accordance with the standard. It should be noted that it is not possible to measure the pressure of the soil at rest p_0 using a Ménard pressuremeter during a standardised test. In section 4.4.2.3, a way is shown of how to carry out this measurement using a passive pressuremeter probe.

4.4.1.1. *Ménard deformation modulus calculation (E_M)*

This modulus is measured in the vicinity of the segment with the gentlest slope in the pressure measurement curve: it is in fact within this range, called the "pressure measurement modulus range", that the soil's response matches that of a practically intact soil, but which is placed under pressure in a field involving major deformations [GAM 98].

When pressurised, the probe's jacket firstly comes into contact with the wall of the borehole and then may compress the soil if there was a slight relaxation. While the pressure is building, a ring of soil subject to micro-plastic and then plastic shearing develops around the probe. You must bear in mind that within the range adopted for calculating the modulus, the soil is already subject to pre-breaking behaviour. It no longer has linear elasticity: the modulus is measured for relative deformations in the order of 10^{-2} and not for elastic deformations of 10^{-6} [GAM 96, MEN 61].

However the formula used for calculating the modulus is the one used for expansion of a cylindrical cavity subject to radial deformations, in a state of linear elasticity. It has been demonstrated [BAG 78] that it remains valid in dilating materials.

$$\frac{\Delta R}{R} = \frac{1}{2G}\Delta p \qquad [1]$$

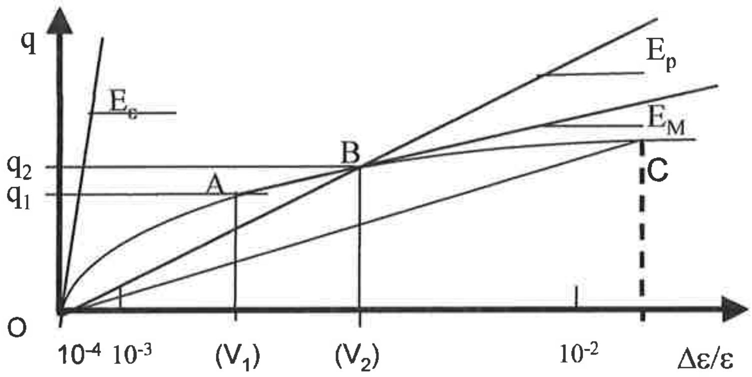
- G is the shear modulus for the range of strains caused,
- R is the radius of the hole's cross-section,
- ΔR stands for the increase in this radius as a function of an increase in pressure Δp on the cavity's wall.

To obtain a modulus E, you use the usual formula that gives Young's modulus:

$$E = 2(1+\nu)G \qquad (2)$$

ν being Poisson's coefficient for the soil.

The convention is to use $\nu = 0.33$ for calculating the Ménard pressure measurement modulus E_M. This E_M modulus cannot therefore be compared either to a true elastic modulus for the soil measured for deformations of 10^{-6} to 10^{-5} and used as part of the waves propagation phenomenon, or with the modulus measured during a loading test on the soil using a plate [GAM 02]. Ménard frequently used the terms "pseudo-elastic modulus" or "micro-plasticity modulus" to distinguish E_M from the micro-deformation modulus E_ε [MEN 61, 62a], which nowadays is referred to as E_o. The measurement range for these various moduluses is shown in Figure 4.6. The E_M modulus measured between the strains q_1 and q_2 (q_1 other than zero) is proportional to the slope of the strain-deformation curve's secant line AB between 10^{-3} and 10^{-2}. It is as low as the modulus of the soil measured under a loaded plate when breaking occurs (slope of OC to the nearest geometric factor).



- $\Delta\varepsilon/\varepsilon$: Relative deformation (arithmetic scale)
- q: Strain applied
- E_ε: Elastic modulus
- E_p: Modulus secant to the plate
- E_M: Pressure measurement modulus

Figure 4.6 *Measurement ranges for the moduluses on the strain-deformation curve*

4.4.1.2. *Determining the Ménard limit pressure p_{LM}*

As is the case with a loading test for a plate, you determine the average breaking pressure when the plate is driven in to an extent that equals $\frac{1}{10}$ of the plate's dimensions, likewise the Ménard limit pressure p_{LM} is conventionally defined as the pressure for which the volume of the initial cylindrical cavity where the central cell is located, is doubled. This expansion therefore corresponds to a relative radial deformation of $\sqrt{2}$. A pressure measurement creep pressure can also be defined, which is necessary for better assessing the representativeness of a test, but this does not play a major role in the sizing of foundations.

4.4.2. *Using the pressure measurement test for determining intrinsic parameters*

This interpretation can only be carried out from measurements obtained during an ideal pressure measurement test performed either using a self-boring pressuremeter, or in a sand box where the material is placed at a selected density around the probe, which is positioned beforehand in the centre of the box. This interpretation requires a choice to be made concerning hypotheses relating to the law of behaviour for the soil [CAM 93].

4.4.2.1. *Shear modulus calculation*

As, in the linear elasticity theory, this modulus is independent of the conditions for volume variations in the soil, it is calculated using formula (1). When breaking down the results of the tests using a self-boring pressuremeter you may, due to the shape of the pressure measurement curve, in addition to the modulus tangent to the origin (which is similar to the elastic modulus), give secant moduluses for each range of pressures and cyclic moduluses on the loading/reloading loops. The advent of a remoulded soil ring modifies these values [BAG 72]. However, in light of the measuring apparatuses involved [FAH 90], it is not always possible to compare the tangent modulus measured to the real elastic shear modulus.

4.4.2.2. *Estimating the intrinsic shear strength parameters*

You can use two techniques for estimating the soil's shear strength parameters:

– An analytical formulation for the expansion of the cylindrical cavity which enables c and φ to be calculated ,

– An inverse method in which you numerically adjust a law of behaviour through comparison of the theoretical results with those obtained experimentally.

4.4.2.2.1. Analytical formulation

If you consider a linear/plastic law of behaviour without a change in the soil's volume (or in other words without contractancy or dilatancy), the relation between the theoretical limit pressure p_L (or in other words the abscissa of the asymptote at the pressure measurement curve parallel to the axis of expansion) and the parameters c and φ may be written analytically, as we are reminded of in [MEN 63]:

$$p_L = (1+\sin\varphi)(p_o + c\cot\varphi)\left[\frac{E}{2(1-\nu)\sin\varphi} \text{x} \frac{1}{p_o + c\cot\varphi}\right]^a - c\cot\varphi \qquad [3]$$

where $a = \dfrac{\sin\varphi}{1+\sin\varphi}$ and ν is Poisson's coefficient. This formula is valid when Young's modulus is constant and when the vertical strain is the main intermediate strain.

You will note that Salençon [SAL 66], using slightly more elaborate starting hypotheses (the volume varies as a function of the average strain in the elastic and plastic phases), but still assuming that the vertical strain remains the main intermediate strain and that the modulus E is constant, gave the following expression:

$$p_L = (1+\sin\varphi)(p_o + c\cot\varphi)\left[\frac{E}{4(1-\nu^2)\sin\varphi} \text{x} \frac{1}{p_o + c\cot\varphi}\right]^a - c\cot\varphi \qquad [4]$$

For the case φ = 0, you find the following:

– With Ménard:

$$p_L = p_o + c_u\left[1 + \mathrm{Log}\frac{E}{2(1+\nu)\,c_u}\right];$$

– – With Salençon:

$$p_L = p_o + c_u\left[1 + \mathrm{Log}\frac{E}{4(1-\nu^2)c_u}\right]$$

which, if we had known E (the true elastic value), and p_L (the true asymptotic value) by breaking down the results of the test itself (see sections 4.4.1.1 and 4.4.1.2) and p_o moreover (see section 4.4.2.3), should have allowed these equations to be solved so that c_u could be obtained in clay. Likewise, when c_u equals 0, we would have been able to solve equations (3) or (4) in order to obtain φ in clean sands. In the more general case $(c \neq o, \varphi \neq o)$, analytical resolution would only have been possible if we had had at least one second relation between c and φ through laboratory tests.

But as these formulas were established for an elastic linear and the purely plastic behaviour of the soil, the E modulus to be taken into account is not E_M (see Figure 4.6), the limit pressure p_L is not the conventional limit pressure p_{LM} and the p_o measure is obtained using a complementary test (see section 4.4.2.3).

In fact, in all these approaches, the soil is considered to be a continuous, single-phase medium. This model may be adopted, whether or not (as has been done above) you add the volume variation hypothesis in order to represent the behaviour of a clay that is drained or not drained respectively. To the contrary, in the case of a powdery soil, you cannot set aside other phenomena such as:

– The contractancy/dilatancy which requires the volume variations due to shearing to be taken into account in order for it to be simulated,

– The soil's sensitivity, or in other words, the reduction of its resistance to shearing subject to major deformations (which also applies to clays).

You will find a study on these points elsewhere [BAG 78] as well as on the role of a remoulded soil ring around the borehole.

Other authors have continued with research acting on the assumption that the vertical strain could play a role in the phenomenon of plasticity [MON 90, 95b]. Formula (3) can be modified to take into account the dilatancy of angle ψ: to obtain c, φ and ψ, a third relation is required [MON 94].

Some of these authors have also strived to deduce these parameters based on the value of the Ménard conventional limit pressure [MON 94, 95b; COM 96], but whilst always keeping E constant in formula (3), modified to take the dilatancy into account.

4.4.2.2.2. Inverse method

Since the conditions at the limits of the soil mass tested are known, the pressure measurement test should be ideal for determining the mechanical parameters of the soil in the case of more complex models, provided that there is an orientation towards an inverse resolution method. With this method, you postulate a law of behaviour and you compare the theoretical cylindrical expansion curve with an experimental pressure measurement test curve, preferably one achieved using self-boring.

In 1961 Gibson & Anderson [GIB 61] proposed the adjustment of a line on the pressure measurement curve in terms of logarithmic co-ordinates in order to obtain the non-drained shear strength by:

$$p = \sigma_h + c_u \ln \frac{\Delta V}{V} \qquad [5]$$

where σ_h refers to the horizontal strain at rest in the soil. More recently, numerous authors, such as Hugues or Monnet, have proposed adjustments to curves in different co-ordinates systems either based on the precisely known initial state [HUG 77] or based on an unloading/reloading cycle in order to obtain the intrinsic parameters of the soil [MON 97].

Likewise Cambou & Boubanga [CAM 89] proposed an optimisation algorithm for the parameters in a finite elements study based on the laws of behaviour of Duncan & Chang [DUN 70], Cambou & Jaffary [CAM 88], and Cam-Clay. Hicher & Michali [HIC 95] used Hujeux's elastoplastic law [AUB 82] to identify soft clay types.

Shahrour, Kasdi & Abriak [SHA 95] studied the possibility of determining the parameters of the Mohr–Coulomb's non-associated model (E, ν, φ and ψ) for a given type of sand based on the pressure measurement test by using the inverse method technique for solutions relating to major deformations. They showed the difficulties involved in this method due to the close linkage between the three parameters E, φ and ψ. Determination requires supplementary data such as an unloading/reloading cycle or the determination of the critical friction angle (which is not susceptible to remoulding) based on laboratory tests.

4.4.2.3. *Determining the pressure coefficient of land at rest*

Insofar as K_0 can have an influence within the context of classic calculations in soil mechanics, it is worthwhile seeking to measure it *in situ*. This moreover was the initial goal of the Camkometer [WRO 72], as its name, featuring the abbreviation K_0, indicates.

Pressuremeter probes inserted into a borehole prepared in advance do not allow K_0 to be measure due to relaxation of the strains in the borehole's wall during the time interval between the end of drilling and the start of the test, as well as due to the complexity of the phenomenon of the jacket's application to this wall. However, by leaving a probe inflated to a volume V (for which p is initially higher than the estimated value for p_0) in the soil until stabilisation occurs, you can obtain a reliable value for p_0. Probes of this type, called "Geocells", which can be disconnected from the P.V.C. monitoring box during the relaxation phase, are available.

On the other hand, with a great deal of experience, you can measure K_0 using a self-boring pressuremeter [BAG 72, BEN 95].

4.4.2.4. *Using the pressuremeter as a sensor*

We have seen that you can envisage leaving the passive measurement pressure probe in the ground, inflated at a low pressure level. Experience shows that this pressure tends to stabilise itself, with the probe acting as a sensor.

In soil types that are not very susceptible to swelling/contraction phenomena, with these probes it is possible to monitor variations in the average strain as a function of civil engineering operations such as boring a tunnel.

In soils that swell or retract depending on their water content, you can also measure the swelling pressure subject to nil deformation [FLA 91, SMI 95].

4.4.3. *Conclusion*

Although, theoretically, the pressure measurement test is the only *in situ* test which can form the subject of a precise analytical solution, solving the problem is not easy, because the strain and deformation fields are not uniform. Thus, we can better understand why Ménard, faced with this problem in the late 1950s, opted for comparison of the soil's response during a pressure measurement test with the response under a foundation, which led him to direct use of the semi-empirical parameters obtained with the help of his tests for sizing foundations, as we shall see below.

Apparatuses that cause the expansion of a borehole are the only ones that can directly give a value for the soil's stiffness for a relative deformation which corresponds to the one created by the foundations of well-designed works. These apparatuses are therefore well suited to measuring the deformation moduluses which are of interest to those involved in the construction sector. This explains why the Ménard pressuremeter has become the tool of choice for checking the densification levels of sandy soils over major thicknesses [DEB 98, GAM 95b].

4.5. Regulatory aspects

In France, initially carrying out the Ménard pressure measurement test was standardised by the French administrative authorities [MEL 71], then its use for sizing foundations from the parameters obtained received ministerial approval [LCP 72]. These standards were updated several times in line with changes in the apparatuses placed on the market and growth in the database of observations regarding the behaviour of real foundations.

4.5.1. *French, European and American regulations*

In England, only recommendations were given regarding the self-boring pressuremeter test [CLA 92, MAI 87]. In the United States, since 1987, standardisation has related to all the pressuremeters placed in boreholes [AST 00]. Moreover, the Federal Highway Administration also published [FHW 89] recommendations accompanied by a videocassette for the use of pressure measurement tests for sizing surface foundations and deep foundations, as well as moulded walls. These recommendations are a faithful reflection of French regulations. In Russia, section 3 of the GOST 20276-85 standard relating to estimating the deformation parameters of soils *in situ* deals with pressuremeters placed in a borehole. Several Central and Eastern European countries have also established standards for the Ménard pressuremeter.

The European Standardisation Committee (ESC) is now responsible for standardisation within the European Union. Chronologically, the first text studied was Eurocode 7 concerning geotechnical calculations. It supports national standards relating to the sizing of foundations and therefore makes reference to calculations based on the results of pressure measurement tests. Moreover, the ESC, in agreement with the International Standardisation Office (ISO), is in the process of drawing up a certain number of standards with input from specialists in the profession, relating to carrying out geotechnical tests based on the expansion of a cylindrical probe in a borehole and determination of the quantities measured:

– ISO 22476-4 Ménard pressure measurement test,

– ISO 22476-5 Flexible dilatometer test,

– ISO 22476-7 Borehole jack test.

Initially, only two types of tests in this category will form the subject of technical specifications; TS 22476-6 and 8 respectively for the self-boring pressuremeter and the pressuremeter positioned by forcing it down.

Consequently, the standards and specifications relating to carrying out the tests that we are going to review will only remain valid until such time as international standards are accepted by the countries in question.

4.5.1.1. *List of French regulations*

Among the AFNOR standards mentioning the Ménard pressuremeter are:

– In the field of carrying out tests and determining "pressure measurement" parameters:

- General standard P94-110-1 of January 2000 [AFN 00] and experimental standard XP P 94-110-2 concerning the test with a cycle, of December 1999 [AFN 99],

– In the field of using Ménard parameters in civil engineering:

- Standard P11-212 (formerly DTU (Standardised Technical Document) 13.2 of June 1978) of September 1992 for deep construction foundations,
- Standard P11-711 (formerly DTU 13.12 of March 1988) of March 1998: Rules for calculating surface foundations.

Recently, the borehole dilatometer test was also standardised [AFN 02].

Moreover, in the CCTG (general technical clauses specification) applicable to public works contracts, part 62, section V titled "Technical Rules for the Design and Calculation of Foundations for Civil Engineering Works" [MEL 93] is broadly based on the use of results from tests *in situ*, modelled on the one established by Ménard for the design and sizing of footings, piles and shafts subject to an axial load (with justification of their stability and their settling) as well as piles subject to horizontal stresses. This document took the place of the old FOND 72 file [LCP 72] concerning the same subject.

As for Eurocode 7, it henceforth matches AFNOR standards:

– P 94-250-1 Geotechnical Calculation, General Rules (published on 1 July 2003),

– P 94-250-2 Geotechnical Calculation, Calculation based on laboratory and *in situ* tests, to be published in November 2004. There is a chapter in it which deals with use of the Ménard pressuremeter and its results for sizing foundations, with mention of the French rules in the appendices.

4.5.2. *Other rules and recommendations in the Far East*

Firstly, these standards are quoted in relation to Japan, where they were first applied.

4.5.2.1. *Japanese regulations*

As the pressuremeter was used in Japan from the 1960s onwards, there are old recommendations there, the most recent of which was included in the general document called "Reconnaissance and tests in rock" (1989).

A Japanese standard applicable to pressure measurement tests carried out in a borehole was published by the Japanese Geotechnical Committee under reference JGS 1421-1995.

Recommendations are also provided in the following documents (in Japanese) [MOR 99]:

– "Soil reconnaissance methods" (1995) published by the Japanese Geotechnical Committee,

– "Technical manual for *in situ* tests in boreholes" (1997) published by the Kanto Geotechnical Consulting Engineers Association.

4.5.2.2. *Chinese regulations*

The following are noted in chronological order:

– Standard JGJ 69-90 of 1 December 1990 concerning the use of the PY borehole pressuremeter featuring 2 volumeters, drawn up by the Ministry of Construction, published by the Changzou Institute of Architectural Design, and published by the Planning Printing Works,

– Standard TB 10046-96 for pressure measurement tests for foundation soils in the field of railroad civil engineering, Ministry of Railways,

– Standard YS 5224-2000 of 1 July 2001 for pressure measurement tests, drawn up for the Metals Industry Society by the Changsea Reconnaissance Institute, published by the Planning Printing Works,

– Code GB 50021-2001 of 10 January 2001, National Standard of the People's Republic of China for geotechnical studies, drawn up by the Ministry of Construction, published by the China Architecture and Building Press, in which pages 108-109 relate to the pressuremeter,

4.5.3. *The basis for design and sizing rules based on the Ménard pressuremeter*

As the drafting of the regulations referred to hereinabove does not include the basis for how they were developed and as they are not educational documents, to find out more, we recommend that you consult recent works on the subject [APA 96]. Below, only an outline of the drafting of these rules is provided as a reminder.

Having understood the difficulty involved in obtaining the intrinsic parameters for a soil (insofar as these parameters exist) based on a pressure measurement test, even with an unloading/reloading cycle, Ménard preferred to compare the soil's reaction during loading of the walls in a borehole with the reaction observed under all types of foundations, both in terms of breakage and deformations.

4.5.3.1. *General vertical stability condition*

Ménard was not the first [SKE 53] or the last [VES 77] person to compare the soil's reaction at the base of a pile with that around a spherical or even a cylindrical cavity [SAL 97] subject to expansion, but he was the only one who had an apparatus enabling him to make *in situ* comparisons between the soil's reaction either around a spherical pressuremeter, or around a cylindrical pressuremeter, both around and under a foundation. The theory and experimentation involved were presented in a series of article (from [MEN 62a] to [MEN 75]). Experimental comparisons have been continued until the present day, particularly by the French National School of Civil Engineering Laboratories Group, which recently enabled these design rules to be refined [MEL 93].

The idea's starting point is the fact (Figure 4.7) that the deeper the pile, the more the role assumed by the soil's elastic compressibility predominates over the role assumed by the limit shear strength in the phenomenon of displacement and breakage under and around the tip of the pile (excluding friction along the shaft of the pile). Thus, Terzaghi's formula on the load-bearing force[2] of the tip of a pile based on rigid plastic soil behaviour must give way to an elasto-plastic theory when the foundation's base is at a depth that is greater than the critical depth.

Rather than build up this theory (in the face of a lack of tools), Ménard preferred to proceed using theoretical correlations and experiments. At a depth greater than the critical depth, there is a correlation between the limit pressure measured using a Ménard pressuremeter and the limit pressure measurement during expansion of a sphere in the same soil. Therefore there is a correlation between the final strain under the tip of a pile (function of the spherical expansion of the volume plastified around this point) and the pressure measurement limit pressure. This can be written using the following mnemonic:

$$q_l - q_o = k\,(p_l - p_o) \qquad [6]$$

where:

- q_l is the final stress under the tip of the pile,
- q_o is the vertical stress due to the weight of the soil at the same level,
- k is the Ménard bearing coefficient,
- p_l is the pressure measurement limit pressure,
- p_o is the soil's horizontal strain at rest.

1. Terzaghi only established the formula for surface footings [GAM 03].

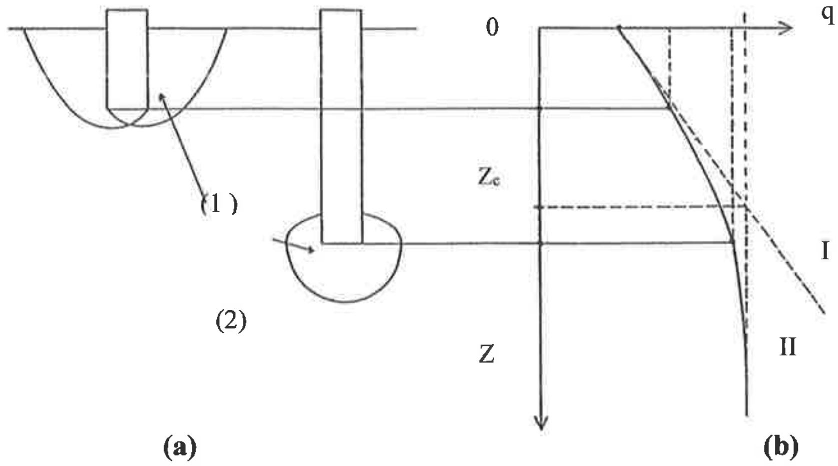


Figure 4.7. *Elastoplastic theory of the load bearing force of a foundation. a) Vertical section of the soil around the foundations: (1) Volumes sheared, (2) Volumes of soil experiencing an elastic reaction around (1); b) Variation in breaking strain q as a function of the depth Z: I) Rigid plastic theory, II) Elasto-plastic theory, Zc: critical depth.*

These are the values for $k_p{}^3$ deduced from the French National School of Civil Engineering Laboratories database, as provided in part 62, section V of the CCTG [MEL 93] as a function of the type of soil, as well as the way in which the pile is executed, and this execution may lead to remoulding at the level of the tip of the pile to a greater or lesser extent (Table 4.2).

Currently, the formula is written as follows:

$$q_L = q_0 + k_p \cdot p_{LMe},$$

introducing the net equivalent pressure p_{LMe} in order to take account of variations in the parameters as a function of the depth [FRA 99].

---

2. The symbol for the load-bearing factor k was supplemented by the index "p" when the same operating method for the parameters was extended to include static penetrometer tests, with use of a k_c value for "cone".

Nature of the ground		Elements implemented without forcing down the soil	Elements implemented with the soil forced down
Clay Loam	A	1.1	1.4
	B	1.2	1.5
	C	1.3	1.6
Sand Sand-gravel mix	A	1	4.2
	B	1.1	3.7
	C	1.2	3.2
Chalk	A	1.1	1.6
	B	1.4	2.2
	C	1.8	2.6
Marl Calcareous marl		1.8	2.6
Altered rock		1.1 to 1.8	1.8 to 3.2

Table 4.2. *Values for the load-bearing factor k_p [MEL 93]*

The values for the lateral friction resistance q_s, which are known to be a function of the soil's shear strength, are also given as a function of the pressure measurement limit pressure as well as of the type of soil and the type of pile (see Figure 4.8 and Table 4.3).

When the foundation is at a depth lower than the critical depth, lower values for k are provided in the tables and graphs (Figure 4.9). The values mainly apply to footings and shafts executed according to the same techniques as the footings, which is why the values for k_p (footings) for a depth close to the critical depth are not lumped in with the values for k_p (piles), as most piles are executed using excavation techniques that destructure the soil to a greater extent and do not enable full cleaning of the base of the cavity.

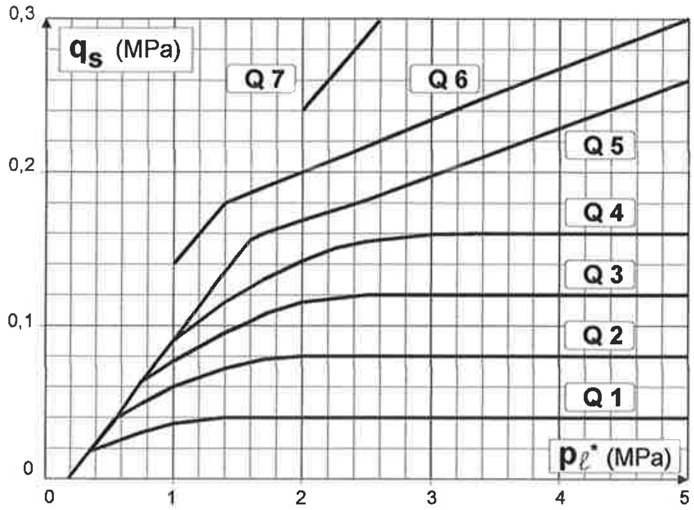


Figure.4.8. *Limit unit lateral shear strength along the length of a pile's shaft [MEL 93] depending on the type of pile, the type of soil and its limit pressure.*

On the following page you will find Table 4.3 which enables you to choose which "Q_i" curve to use depending on the type of pile and the type of soil.

4.5.3.2. *Permissible differential settling condition*

We know that Terzaghi was one of the first people to stress the need to design building foundations based on surface footings on the basis of a permissible differential settling criterion between neighbouring footing units. For these buildings, the basic rule was that the settling of the footing subject to the greatest load should not exceed 25 mm. For structures, the secondary stresses due to support level variations define the acceptable settling. In order to estimate this settling, Ménard had a pressure measurement modulus which, as we have seen, was obtained based on a shear modulus G in accordance with formula (2).

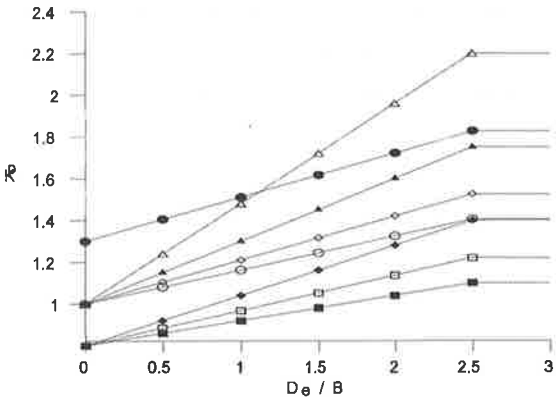
Ménard could have used either Boussinesq's theory for calculating the strains or Terzaghi's one-dimensional consolidation theory (for which the strain path is very different from the one in a pressure measurement test) but these two theories have their limits:

Rammed prefabricated concrete	Rammed metal, closed tip	Shaft (5)	Bored using permanent tubing	Bored using temporary tubing.	Bored in mud	Simple bore	Type of pile	
Q_2	Q_2	Q_2	Q_1	Q_1, $Q_2^{(3)}$	Q_1, $Q_2^{(1)}$	Q_1, $Q_2^{(1)}$	B	Clay, Loam
		Q_3				Q_2, $Q_3^{(1)}$	C	
Q_3	Q_2	-	Q_1	Q_1	Q_1	-	A	Sand, Sand-gravel mix
			Q_2	Q_2, $Q_1^{(2)}$	Q_2, $Q_1^{(2)}$		B	
	Q_3		(4)	Q_3, $Q_2^{(2)}$	Q_3, $Q_2^{(2)}$		C	
(4)	(4)	Q_1		Q_1	Q_1	Q_1	A	Chalk
		Q_2		Q_2	Q_3	Q_3	B	
		Q_3	Q_2	Q_3, $Q_4^{(3)}$	Q_4, $Q_5^{(1)}$	Q_4, $Q_5^{(1)}$	C	
Q_3	Q_3	Q_4	Q_3	Q_3	Q_3	Q_3	A	Marl
Q_4	Q_4	Q_5		Q_4	Q_4, $Q_5^{(1)}$	Q_4, $Q_5^{(1)}$	B	
Q_4	Q_4	Q_6	-	-	Q_6	Q_6		Soft rocks

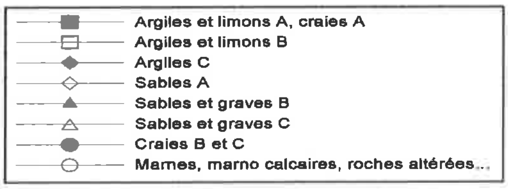
Injected (high pressure) (6)	Injected (low pressure)	Rammed, coated	Rammed, moulded *in situ*
Q_4	Q_2	Q_2	Q_2
Q_5			
Q_5	Q_3	Q_3	Q_2
Q_6		Q_4	Q_3
.	Q_2	(4)	Q_1
Q_5	Q_3		Q_2
Q_6	Q_4		Q_3
Q_6	Q_5	Q_3	Q_3
		Q_4	Q_4
Q_7(7)	.	.	.

(1) Reboring and grooving at the end of boring
(2) Very long pile (exceeding 30 m)
(3) Dry boring, tube not tacked
(4) As the lateral friction can be very low, carry out a study on a case-by-case basis
(5) Provided that neither any tubing, nor any drilled ferrule are lost
(6) Selective, repetitive injection at a low flow rate
(7) Selective, repetitive injection at a low flow rate, after treatment of fissured or broken masses.

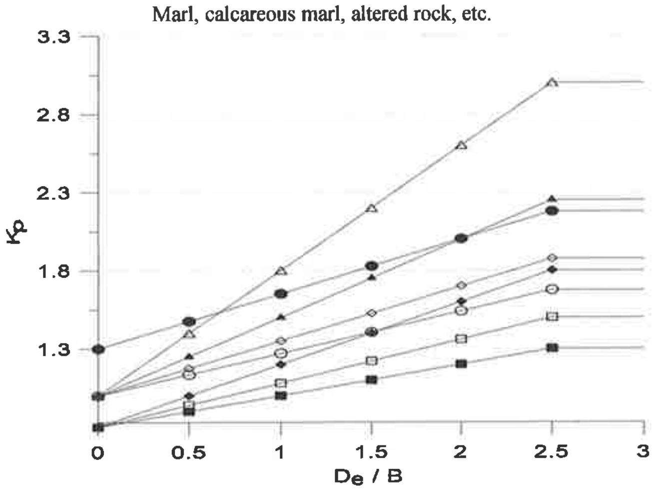
Table 4.3. *Choosing the Q_i curve to adopt depending on the type of soil and the type of pile.*



(a)



Clay and loam A, chalk A
Clay and loam B,
Clay C
Sand A
Sand and sand-gravel mix A
Sand and sand-gravel mix B
Chalk B and C
Marl, calcareous marl, altered rock, etc.



(b)

Figure.4.9. *Pressure measurement load-bearing factor for footings as a function of their relative depth D_e/B. The conventional categories for soils are also provided in [MEL 93]. (a) continuous footing, (b) square footing*

– The first one, which is the work of a mathematician, only applies to uniform loads and the user assumes that the E modulus is constant,

– The second one was only established for thin layers of saturated soils on which it is rare for a geotechnician to directly erect buildings.

Therefore, Ménard preferred to propose a new method by breaking down the settling of a given soil under a shallow, rigid foundation into two terms [MEN 61]:

– Settling caused by a spherical (or isotropic) tensor in the field of the strains induced in the soil, or in other words a variation in volume,

– Settling caused by deviation tensors in the same field of strains, or in other words a set of angular deformations,

whilst neglecting strictly elastic settling based on E_ε (section 4.1.1. and Figure 6).

This approach enabled him to:

– Introduce the equivalent of a Young's modulus in the first term:

$$E = EM/\alpha \qquad [7]$$

where α is a structural coefficient determined by experience, which varies from ¼ to 1 depending on the type of soil (Table 4.4),

– Directly use the modulus G_M deduced directly from E_M with $\nu = 0.33$ in order to establish the second term,

– Take into account variations in G and E as a function of the amplitude of the deformations, without reintroducing E_ε.

Here too, Ménard was not the only one who tried this approach [SUK 63a-b], but he was the only one came to systematise it. For Ménard, the formulas proposed correspond to 10-year settling, as long as the strains remain in the micro-plastic field of the deformations. Above and beyond that, a corrective hyperbolic factor was proposed, as we shall see below. Lastly, these formulas are not accurate when the soil studied has not reached its self-supporting threshold [MEN 75, APA 96].

The general formula for average settling w (where the order of the terms is reversed) for a rigid footing on a soil type that is relatively homogeneous soil in terms of its depth is written as follows [MEL 93]:

$$w = \frac{2}{9E_M} q\, B_o \left(\lambda_d \frac{B}{B_o} \right)^{\alpha} + \frac{\alpha}{9\, E_M} q\, \lambda_c\, B \qquad [8]$$

– E_M is the pressure measurement modulus, which may be weighted,

– q is the supplementary average vertical strain contributed by the footing to the pre-existing strain on the soil, if this is not cancelled out by the works,

– B_0 is the reference width (which equals 0.6 m with traditional probes),

– B is the width of the footing,

– λ_d and λ_c are footing shape coefficients provided in Table 4.5,

– α is the structural coefficient defined in formula [7].

When the soil is heterogeneous, E_M is replaced by an equivalent value, weighted based on values for E_M measured as a function of the depth.

It should be noted that Poisson's coefficient is not apparent, because in the term for settling caused by deviation G (the first term in the formula) it disappears, and in the term for settling of spherical origin, formula [7] integrates $\nu = 0.33$.

Theoretical critical studies of formula [8] have already been carried out [COR 81] as it does not match the one for the theory of linear elasticity and its hypotheses. The strict application of this formula, which integrates the non-linear response into the strains (variable E) and the fact that the footing is rigid lead to an excellent concurrence between the projected values and the observed values for settling [BAK 93].

It is remarkable that at the end of the competition organised at the international *Settlement '94* conference at College Station (Texas) [BRI 94] on forecasting the settling of four footing units on sand *in situ*, the winning team was the one that used the pressure measurement method described here [TAN 94]. Following this being noted, experimental studies were continued with a view to directly comparing the Ménard pressure measurement curve and the settling curve [BRI 99, 03], including in the case of off-centred loads, inclined loads, and foundations on the edge of earth banks by using (among other things), the French National School of Civil Engineering Laboratories database.

Type of material	Peat		Clay		Loam		Sand		Sand and gravel	
	E/p_l	α	E/p_l	α	E/p_l	α	E/p_l	α	E/p_l	α
Overconsolid-ated			> 16	1	>14	2/3	>12	1/2	>10	1/3
Consolidated normally		1	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4
Altered and Reworked			7-9	1/2		1/2		1/3		1/4

Table 4.4. *Values of the structural coefficient α depending on the types of soil*

L/B	1		2	3	5	20
	Round	Square				
λ_c	1	1.1	1.2	1.3	1.4	1.5
λ_d	1	1.12	1.53	1.78	2.14	2.65

Table 4.5. *Values of the λ coefficients*

When the soil consists of a double layer (or is multi-layered) and one of the layers is not overconsolidated, Ménard recommended using a one-dimensional consolidation formula to estimate the settling of the non-overconsolidated layer caused by a flexible foundation (uniform strains):

$$w = D \int_{z}^{z+D} \frac{\alpha(z).\beta(F).p(z)}{E_M(z)} dz \qquad [9]$$

– D is the thickness of the non-overconsolidated layer,

– z is the dimension of the top of the layer,

– E_M (z) is the pressure measurement modulus at depth z,

– α(z) is the soil's rheological coefficient

– β(F) is a hyperbolic corrective factor intended to take into account the increase in deformation in the vicinity of the break where F is the safety coefficient for the foundation in relation to the break in the layer of non-overconsolidated soil: $F = q_l/q$

$$-\beta = 0.66\frac{F}{F-1} \quad \text{if } F \leq 3$$

$$(\beta \rightarrow \infty \text{ when } \quad F \rightarrow 1)$$

$$-\beta = 1 \quad \text{if } F \geq 3$$

When the soil is underconsolidated, it is still susceptible to settling under its own weight, with this being even more the case when subject to overloading, depending on the external causes. While this phenomenon is well known for clay, it is less well known for sand with a lower relative density index; subject to the effects of shaking caused by heavy traffic, the ramming of piles, or an earthquake, the settling of loose sand resumes [GAM 97, TER 48]. Thus Ménard defined a self-supporting threshold as a function of p_{LM} for each type of soil, within which settling calculated using formulas [8] and [9] is only a default value. For a formation with a depth less than 10 m, the limit pressure characterising the self-supporting threshold varies from 300 kPa for clay to 600 kPa for sand.

It should be noted that to assess the development of settling over time, you can utilise the results of the Diflupress L.D. tests (see section 4.1) using a calculation programme drawn up based on the assumption that there are small flat deformations based on the finite elements method applied to a non-linear visco-elastic material [BAH 95]. The projections compared with the observations were satisfactory.

4.5.3.3. *Solving other problems*

Ménard, along with his colleagues, provided rules regarding the following problems:

– Settling of piles: Gambin [GAM 63], Cassan [CAS 66] and then Frank & Zhao [FRA 82] successively demonstrated that a method based on Ménard's principles enables a very good estimate of the settling of piles,

– Sizing of the foundation shafts and of piles subject to horizontal strains, for which, to some extent, you can compare the strains field developed by these piles in the soil to the one that is created by the pressuremeter [MEL 93, MEN 62b],

– Sizing of the supporting works for which you can define a reaction coefficient by comparing the deformation of the abutting soil in front of the screen and the settling of a foundation, by a 90° rotation of the direction of the forces and displacements [LCP 85, MEN 64].

The design of soil improvement projects involving use of ballasted columns and static horizontal compacting (also called "solid injection") can also be carried out with ease based on the results of pressure measurement tests due to the similarity of the behaviour of the soil around the pressure measurement probe and around either ballasted columns while they are being loaded, or dry mortar columns while they are being formed. For further details about all of these problems, refer to the report on the state of knowledge drawn up by the SIMSTF European Technical Commission [AMA 91].

Most of the sizing calculations can be carried out using software available either from specialist firms, or from SETRA (the Roads and Motorways Technical Studies Service) forming part of the French Ministry of Infrastructure and Housing.

4.6. Prospects

As we have just seen, the pressuremeter in itself constitutes a special class of *in situ* tests insofar as the reaction of the soil around the probe can be analysed either using a simple theory (Ménard pressuremeter) or using elaborate theories that vary depending on the types of soils upon which pressures are placed (self-boring pressuremeters):

– In the first case, it was possible to set up a system for making use of test results in order to solve most of the problems involved in sizing foundations by taking into account the elasto-plastic character of the soil's response and the variation in the deformation modulus as a function of the amplitude of this deformation,

– In the second case, either by using direct analytical methods, or using an inverse method for finding a solution, values for the intrinsic parameters of the soil studied can be determined, although by using simplistic hypotheses and, quite frequently, complementary laboratory tests.

Lastly, it has been shown that implementation of the apparatus requires a certain amount of experience, and at very least a certain amount of care depending on the type of soil studied, whether what is involved is a test using a Ménard pressuremeter or a self-boring pressuremeter. The case of tests in soft or altered rock merits a special chapter within the context of this presentation.

4.6.1. *Prospects for the Ménard pressuremeter*

There are prospects on several levels.

4.6.1.1. *Implementation*

Currently, for deep tests, a major part of the cost of the test is due to the handling operations involved in lowering and raising the drilling tool and the pressure measurement probe, with the length of a drilling phase being limited by the standard in force.

A system combining a drilling module to be defined and a pressure measurement probe should enable the current soil response mode to be maintained, but would avoid the handling operations mentioned above. No research about this was completed until recently.

4.6.1.2. *Processing of the measures*

Thanks to the automatic recording of data, processing tests can be done very quickly. Instead of processing one test per minute, you can process a hundred or so tests in a few minutes. This practice must always be supplemented by a visual check performed by the engineer in charge of the file.

4.6.1.3. *Making use of the results*

Use of the results for sizing foundations will benefit from an increasingly comprehensive database which, among other things, will enable the values for the load-bearing coefficient at the tip to be refined depending on the implementation conditions for these foundations (for example, a pile consisting of an open metal tube) and for the resistance to lateral friction, taking into account the influence of the diameter of the pile for example, a parameter which is currently neglected after having been considered about twenty years ago.

4.6.2. *Perspectives for the self-boring pressuremeter*

4.6.2.1. *Accuracy of the measurements*

Although with these pressuremeters you can measure values for higher moduluses that relate to smaller deformations, a great deal of uncertainty remains regarding the modulus/deformation relation. The LCPC's PAF technique which, up until now, required a more reliable volume measurement, seems more reliable but the PAF's probe should be extended in order to comply with a minimum slenderness ratio [HAR 74, HOU 93, LIV 71, TRA 46]. For the SBP, the increase in the number of diametric planes in which you measure the deformations is perhaps not the right direction for research compared to one involving obtaining a more reliable system of feelers.

4.6.2.2. *Making use of the results*

Due to the high degree of numerical precision of these results, it is important to gain a better knowledge of their representativeness, whether in terms of the K_o, E_o, c_{max} or c_{ult} in clay [MAI 87]. Comparisons with real triaxial or planar shear tests under the same deformation conditions must be undertaken.

Better ways of measuring interstitial pressures should be developed so that a better knowledge can be obtained of the effective strain paths during a test. In sand, a behavioural model that is compatible with cylindrical expansion has yet to be found so that use can be made of the results.

4.6.3. *Soft or altered rocks*

Currently only probes with feelers enable deformation measurements to be made which are less than one micrometre. The influence of the appearance of radial fissures during a test on the parameters measured has only been studied a little; comparative tests in the laboratory are required. The significance of the ratio between the initial load modulus and the cyclic load modulus has not yet been properly mastered, and choosing between one modulus or the other in terms of sizing will not be defined so long as more numerous observations have not been carried out on works in service.

4.7. General conclusion

In France, under the impetus of its inventor, Louis Ménard, the pressuremeter was the source of a new conception of geotechnics both in scientific terms [MEN 61] and in terms of civil engineering [MEN 63].

The apparatus offers the major advantage of carrying out a loading test on the soil and of measuring relative deformations from 10^{-3} through to the breaking point. The test can therefore provide at least two parameters; a deformation parameter, and a breakage parameter.

By promoting sizing rules for foundations based on the direct use of semi-empirical parameters[4] obtained through making simple use of the measurements obtained, involving comparison between the soil's behaviour during the test and the behaviour of the foundations, Louis Ménard opened up a new approach which the users of other *in situ* test apparatuses such as the static penetrometer, subsequently adopted [ASC 93].

Lastly, as the deformation modulus measured corresponds to the relative deformations shown by civil engineering works, credence may be added to the assertion that Louis Ménard would not have been able to develop his methods for

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3. The parameters proposed by Ménard were defined theoretically but were readjusted empirically.

the deep densification of sandy soils without the help of the pressuremeter [GAM 95b].

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