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SOME ASPECTS OF RESEARCH AND PRACTICE FOR FOUNDATIONS DESIGN IN FRANCE

ABSTRACT: After giving the general context of foundation design in France, the present paper summarizes the main features of the methods and rules used for designing foundations: shallow foundations, axially loaded piles and transversely loaded piles. These methods are established mostly with the use of results of Ménard pressuremeter tests (MPM). They are included, in particular, in the Code for civil engineering works of the Ministry of public works, called 'Fascicule 62 – V' (MELT, 1993) The original experimental research programmes are mentioned, and the present transition for adapting them to Eurocode 7 on 'Geotechnical design' is described.

POVZETEK: V prvem delu prispevka so predstavljeni splošni principi projektiranja temeljenja v Franciji. V nadaljevanju so povzete glavne metode in načela za projektiranje plitvih temeljev ter osno in prečno obremenjenjih pilotov, ki večinoma temeljijo na rezultatih meritev z Ménardovim presiometrom (MPM). Te metode so sestavni del Predpisov za inženirske konstrukcije t.i. 'Fascicule 62 – V' (MELT, 1993), ki jih izdaja Ministrstvo za javna dela. Omenjeni so tudi originalni ekperimentalni raziskovalni programi ter njihova implementacija v Evrokodu 7 o geotehničnem projektiranju.

INTRODUCTION AND HISTORICAL BACKGROUND

Since the early 1970's foundations are designed in France using extensively the results of in situ tests, namely cone penetration tests (CPT) and Ménard pressuremeter tests (MPM). This is due, in particular, to the difficulty of taking so-called 'intact' samples in most of the grounds…

This paper focuses mainly on the use of MPM test results, but some aspects are also valid for CPT results, or are of wider application, such as the predictions of foundation movements for the design of structures. Details on the use of CPTs can be found in the comprehensive papers by Frank and Magnan (1996) and Bustamante and Frank (1999).

The Ménard pressuremeter is a specific form of prebored pressuremeters (note that sometimes it can be driven inside a slotted tube) It was invented and developed by Ménard (1955) who also established the first corresponding rules for the design of foundations: settlement of foundations (Ménard and Rousseau, 1962), behaviour of deep foundations under transverse loading (Ménard, 1962-1969), and axial bearing capacity of foundations – whether shallow or deep (Ménard, 1963).

As a consequence of the growing interest in the use of MPM for soil characterisation, in the subsequent decade Ménard proposed some further developments of these design directives. The evolution of the rules for the design of shallow and deep foundations was included in a general document published by Ménard (1975). The document dealt with the analysis of available methods of shallow and deep foundation design (bearing capacity (q_u), skin friction (q_s) for piles and foundation settlements).

It is clear that the immense advantage of the Ménard pressuremeter test (MPM) is that it provides the geotechnical engineer with both a failure parameter (the limit pressure p_l) and a deformation parameter (the pressuremeter modulus E_M). It enables him to tackle with the same test the problems of bearing capacity of foundations (using p_l), as well as the problems of displacements of foundations (using E_M), i.e. the problems of deformation of the structures to be carried.

The development of the use of MPM for foundation design was, nevertheless, very often limited by the fact that it needed a new approach, outside the conventional and classical framework of soil mechanics (which had been developed mainly with the use of laboratory tests, like the triaxial test and the oedometer test) - see, for instance, the recent paper by Gambin and Frank (2009).

Indeed, the rules for the design of foundations from MPM are essentially of 'direct' type, i.e. they use direct correlations between the measured parameter (p_l or E_M) and the 'design' parameter (bearing capacity, settlement or transversal displacement). They do not require to determine first a 'basic' soil parameter (parameters of shearing resistance or oedometer modulus) to enter, subsequently, into the classical bearing capacity formulae or oedometer or elastic formulae for the settlement.

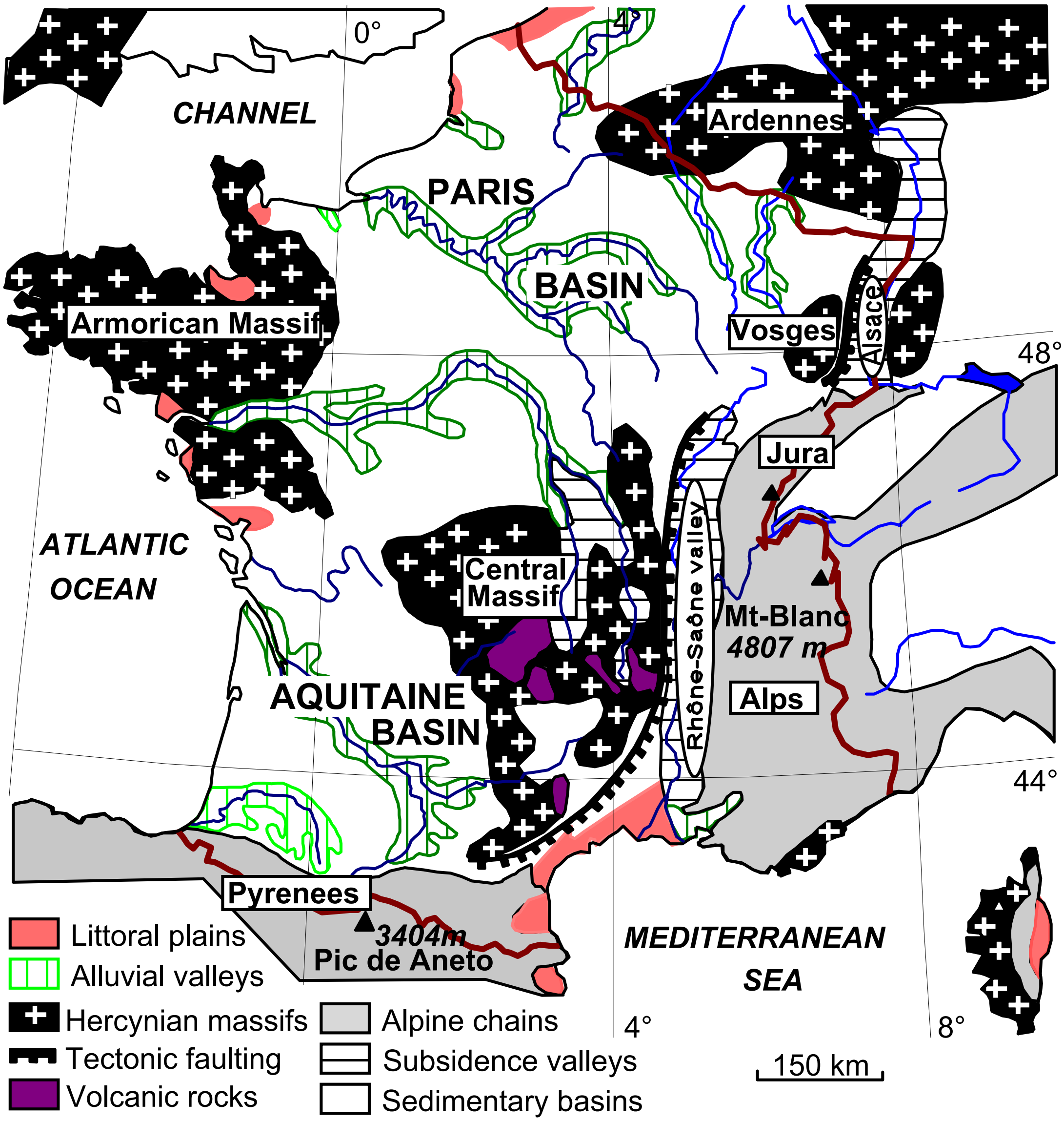

Figure 1. General map of the geology of France. (Frank and Magnan, 1996).

One of the other advantages of MPM is that it can be performed in all kinds of grounds, from soft soils, to very stiff or very dense soils and soft rocks, thanks to the preborehole. In the French geological context this has also turned out to be a great advantage. Figure 1 is a general map of the geology of France.

The Laboratoires des Ponts et Chaussées (LPCs, Highways Authority Laboratories) were very soon interested by the pressuremeter tests and its application to the design of foundations (see, for instance, among their early publications, Jézéquel and Goulet, 1965). As a matter of fact, the design rules proposed by Ménard constituted the basis for the first document of recommendations called “FOND 72”, published in 1972 by the Ministry in charge of public works of France.

Starting from the early nineteen seventies, the urgent need for updated specifications for the foundation design in France was at the origin of an intensive research work. This activity was carried out mainly by the Laboratoires des Ponts et Chaussées (LPCs); it consisted essentially of full scale testing and, after 1985, of geotechnical centrifuge testing performed on the LCPC Nantes centrifuge facilities. Interest was focused on:
- the assessment of design rules for foundations valid for civil engineering public works contracts, expanding those already existing for buildings;
- extending to the field of foundations design the limit state approach elaborated since 1979 for the design of structures;
- making these foundation design rules compatible with the latest limit state design specifications used for steel, reinforced concrete or pre-stressed concrete structures.

Thus, after more than twenty years of research effort, corresponding to the construction of important infrastructures in France, such as the motorway network and the TGV (very fast train) lines, the first draft of the Code of Practice replacing the “FOND 72” recommendations was completed. The Code of Practice, called “Fascicule 62 - Titre V”, was approved and officially adopted by the Ministry in charge of public works in March 1993 (Ministère de l’Equipement, du Logement et des Transports, MELT 1993).

The work by Bustamante and Gianeselli (1981) based on full scale testing of piles, for instance, was a milestone because it formed the basis for the revised MPM rules (and CPT rules) for the bearing capacity of piles, both for the code for buildings (DTU 13.2, 1983) and for the code for civil engineering works Fascicule 62-V (MELT, 1993).

Fascicule 62 – Titre V deals with all the usual foundation problems onshore, such as foundations of bridges, that is to say foundations subjected to axial loads, transverse loads, moments, negative skin friction and lateral thrusts due to soil movement. Frank (1999) gives, for practical purposes, an extensive summary of the contents of the Code.

In Fascicule 62 – Titre V, the original design method set up by Ménard in the 1960's is still used. However, the design charts and other quantitative material have been changed and updated following the research work of the LPCs. Furthermore, the method has been adapted to the limit state approach. Fascicule 62 – Titre V thus contains a complete set of rules, providing the foundation engineer with all the means of designing foundations. It contains precise equations or formulae for calculating bearing capacities and displacements of foundations.

With respect to limit state design, it is to be mentioned that Eurocode 7-part 1 on Geotechnical design - General rules (CEN, 2004) was precisely under preparation at the same period (the prestandard ENV 1997-1, was published by CEN as early as 1994…).

Most of the rules will be kept in the French standards needed to complement Eurocode 7. Some work for updating these rules, in order to take account of more recent experimental results, is also underway. They will form the basis of the so-called ‘geotechnical models’, in the sense of Eurocode 7 (which quotes or develops some of them, but only in an ‘informative’ manner). It is expected that the French standard for pile foundations will be completed by the end of the year (2010) and the one for spread shallow foundations should be drafted during 2010.

SHALLOW FOUNDATIONS

The results of Ménard pressuremeter tests, i.e. the limit pressure p_{le} and the pressuremeter modulus E_M, are used to derive, respectively, the bearing capacity and the settlement of shallow foundations.

Bearing capacity

The original MPM rules for shallow foundations are primarily based on the experimental data Ménard obtained on the Longjumeau and Granville sites (Ménard, 1963 and CEG L. Ménard, 1967).

In the Fascicule 62-V Code (MELT, 1993) these rules have been modified to take into account the results obtained by the LPCs either by site testing or by testing in the geotechnical centrifuge of the LCPC (located at Nantes). The modifications concern:
- the influence of the embedment (from the site tests results);
- influence of the inclination and eccentricity from site tests results, and from the tests performed by Muhs and Weiss at the Degebo in Berlin in the 1970s;
- influence of the proximity of a slope (from the centrifuge tests results).

The assessment of the bearing capacity of shallow foundations from MPM results according to Fascicule 62-V is the subject of the informative Annex E.1 in Eurocode 7 - Part 2 called “Example of a method to calculate the bearing resistance of spread foundations” (EN 1997-2, CEN, 2007).

As a matter of fact, from 1978 to 1989, the LPCs performed more than 100 tests of footings on soils in situ, at experimental sites. They investigated their behaviour up to failure, or to large settlement values, under the most varied loadings (centred, eccentric, inclined, short-duration, long-duration, and cyclic). The footings, generally 0.7 m or 1 m wide, were placed on the surface, embedded, or located near a slope.

There are only a few publications covering the efforts made by a whole group of experts to convert or use the new tests results for the Fascicule 62-V Code. Among them, the general documents by Amar et al. (1987) and Bakir et al. (1993) should be mentioned.

After 1993, the *a posteriori* validation went on, in particular with the thesis of Maréchal (1999) on the combination of various influences, and with tests on the influence of the embedment carried out at the sand site of Labenne.

In the state-of-the art published by Canépa and Garnier (2004), the reader can find a rather comprehensive bibliography on the experimental data available for shallow foundations.

The rules in Fascicule 62-V can be summarised as follows. The failure stress q_u is obtained from:

$$q_u - q_o = k_p . (p_{le} - p_o) \quad (1)$$

where q_o and p_o are the confining vertical (σ_{vo}) and horizontal (σ_{ho}) stresses; p_{le} is the equivalent limit pressure at the base of the foundation; k_p is the bearing factor, function of the type and compactness of the soil, of the relative embedment depth D_e/B (D_e is the equivalent embedment depth) and of the width to length ratio B/L, see Table 1. The soil type and compactness is determined according to Table 2 (Note that Table 2 also gives the soil type according to CPT results, q_c being the cone resistance).

In order to take into account the load inclination and the proximity of a ground slope, the following expression replaces equation (1):

$$q_u - q_o = i_{\delta\beta}\, k_p . (p_{le} - p_o) \quad (2)$$

In the Code, coefficients $i_{\delta\beta}$ in equation (2) are proposed to account for the effects of load inclination and ground slope.

Table 1. Fascicule 62-V: MPM bearing factor for shallow foundations.

SOIL	Value of k_p
Clay & Silt A, Chalk A	0.8[1 + 0.25 (0.6 + 0.4 B/L) D_e/B]
Clay & Silt B	0.8[1 + 0.35 (0.6 + 0.4 B/L) D_e/B]
Clay C	0.8[1 + 0.50 (0.6 + 0.4 B/L) D_e/B]
Sand A	[1 + 0.35 (0.6 + 0.4 B/L) D_e/B]
Sand & Gravel B	[1 + 0.50 (0.6 + 0.4 B/L) D_e/B]
Sand & Gravel C	[1 + 0.80 (0.6 + 0.4 B/L) D_e/B]
Chalk B & C	1.3[1 + 0.27 (0.6 + 0.4 B/L) D_e/B]
Marl, Calcareous Marl & Weathered Rocks	[1 + 0.27 (0.6 + 0.4 B/L) D_e/B]

Table 2. Fascicule 62-V: Conventional categories for soils and soft rocks.

SOIL		TYPE	P_{1e} (MPa)	q_c (MPa)
Clay & Silt	A	Soft	<0.7	<0.3
	B	stiff	1.2-2	3-6
	C	hard(clay)	>2.5	>6
Sand & Gravel	A	loose	<0.5	<5
	B	medium	1-2	8-15
	C	dense	>2.5	>20
Chalk	A	soft	<0.7	<5
	B	weathered	1-2.5	>5
	C	dense	>3	
Marl & Calcareous Marl	A	soft	1.5-4	
	B	dense	>4.5	
Weak Rock	A	Weathered	2.5-4	
	B	Fragmented	>4.5	

The influence of an eccentric loading is taken into account through the 'reference pressure q'_{ref}' applied by the footing to the soil, which is to be checked against the design bearing pressure.

The design bearing pressure is given by:

$$q_{ref} \leq i_{\delta\beta} (p_{le} - p_o)/\gamma_q + q_o \qquad (3)$$

with γ_q = 2 for Ultimate limit states (ULS) (for persistent and transient design situations – so-called 'fundamental combinations', as well as for accidental design situations) and γ_q =3 for the Serviceability limit states ones (SLS).

It is clear that the use of γ_q = 2 together with ULS-fundamental combinations covers in fact serviceability limit states, considering that the values of the load factors in Fascicule 62-V are exactly the ones recommended by the Eurocodes. The recommended value of Eurocode 7-Part 1 for the bearing capacity of spread foundations is $\gamma_{R;v}$ = 1.4. But, Eurocode 7-Part 1 also advocates a settlement calculation to cover the Serviceability limit states (SLS). On the other hand, apart from verifying separately both ULSs and SLSs (called the 'direct method'), Eurocode 7 also offers the possibility to cover all limit states by a calculation resembling the traditional calculation, i.e. a serviceability type with γ_q =3 (like in Fascicule 62-V) - provided that the supported structure does not require a settlement calculation, of course (called the 'indirect method' in Eurocode 7). These requirements and/or possibilities of Eurocode 7 will be taken on board in the new French standard for shallow foundations, while adapting the rules given above (equations 1 and 2).

Settlement

The settlement is calculated under the quasi-permanent SLS combinations. The original Ménard proposals are unchanged.

The assessment of the settlement of shallow foundations from MPM results is the subject of the informative Annex E.2 in Eurocode 7 - Part 2 called “Example of a method to calculate the settlements for spread foundations” (EN 1997-2, CEN, 2007).

The pressuremeter modulus E_M is a deviatoric modulus, especially well suited to the calculation of the settlement of foundations for which the deviator stress field is dominating, namely "narrow" foundations, such as the footings of buildings and bridges (as opposed to foundations for which the spherical field is dominant, such as embankments and rafts that are large with respect to the compressible layer).

The direct method for calculating settlements from the results of pressuremeter tests MPM was first proposed by Ménard and Rousseau (1962). It has since been widely used in France. Its use is simple, and is summed up in the following relation for the settlement at 10 years of a foundation embedded by at least one width B (or one diameter):

$$s(10\ \text{years}) = (q - \sigma_{vo})\ [\ 2B_o(\lambda_d B/Bo)^{\square}/(9E_d) + \alpha\lambda_c B/(9E_c)\ ] \qquad (4)$$

where B_o = 60 cm, σ_{vo} is the total vertical original overburden stress at the level of the foundation, λ_d and λ_c are shape coefficients, α is a rheological coefficient (a function of the structure and of time) and E_d and E_c are the pressuremeter moduli in the deviatoric zone and the spherical zone, respectively. This method can be applied to all soils and to all shapes of foundations (square, circular, elongated, or strip). The coefficient α allows, in particular, for the overconsolidated, intact, or remoulded character of loose soils or the degree of fracturation of rock. In the case of a foundation placed on the surface, the settlement must be increased by 20 %.

Practically, the first review articles on applications to actual projects started to appear at the beginning of the 1970s (Ménard, 1971; Bru et al., 1973).

Amar et al. (1977) analysed the results of the measurements of settlements of some 30 footings. The evolution of the settlements with respect to time were measured until stabilisation. The applied load corresponded to the conventional service load, which is a load with of factor of around 3 with respect to the failure load. Figure 2 shows the comparison with the settlements predicted by the MPM method (equation 4). The agreement is good. Most of the experimental points are located within +/- 20 % from the line with the value 1. The accuracy is felt to be sufficient for most practical problems.

Baguelin et al. (1978) produced a detailed theoretical justification of the method, together with a summary of observations, often after many years, of actual structures (bridges, water towers, and highway fills) by the LPCs (45 cases at 26 sites). The predictions are in most cases within ± 50 % of the observed long-duration settlements and often within ± 30 %.

There are many other cases of satisfactory application to actual projects, reported in the international literature (see e. g. Frank,1991).

Briaud (1986) used the pressuremeter method to interpret the settlements under short-duration loads in a score of tests of footings on 10 sites with sands, stiff clays, and silts. He concluded that an accuracy of ± 50% in the prediction of settlements can be expected from the method. He also proposed an elastic method with a Young's modulus E equal to the pressuremeter modulus for stiff clays ($E = E_M$), but twice the pressuremeter modulus E_M for sands ($E = 2E_M$).

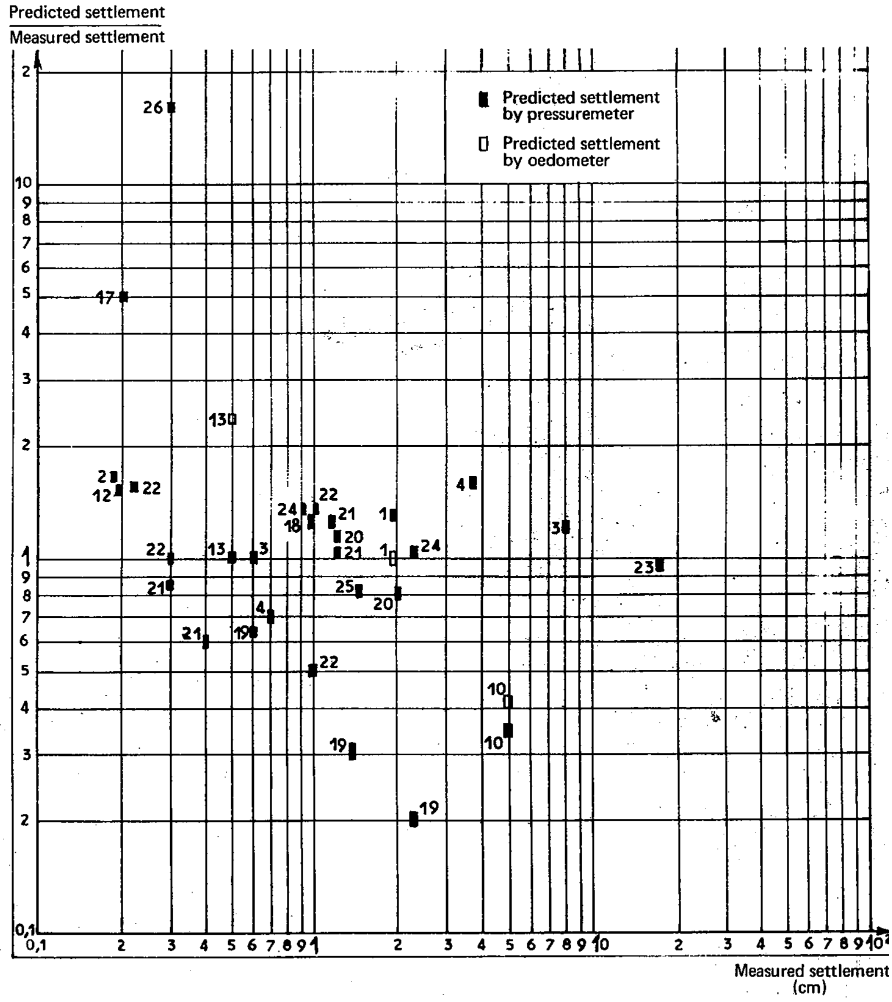

Figure 2. Comparison between measured and predicted settlements for footings (Amar et al., 1977)

Figure 3 is obtained from the results of the experimental research programme mentioned above, carried out by the LPCs from 1978 to 1989, for the cases of settlement under centred vertical loads. It compares the "measured" settlements to the calculated settlements s_M and s_E for a load equal to 50 % of the failure load. The "measured" settlements are in fact settlements extrapolated to 10 years from short-duration tests (tests in steps maintained for 30 minutes, and then extrapolated logarithmically). They include, for each load step, the immediate settlements and the observed settlements between 1 min and 30 min, in addition to the estimated settlements between 30 min and 10 years. The settlements are calculated by the MPM method s_M or by the classic linear isotropic elastic theory s_E, taking the pressuremeter modulus as the Young's modulus E. For the four types of soils studied here (sand, silt, chalk and clay), the MPM settlements are systematically equal to half the settlements at 10 years. The elastic theory, on the other hand, would estimate these settlements correctly in the case of clays and overestimate them by a factor 2 in the case of sands.

In fact, if the immediate settlements are deducted from the settlements at 10 years and a correction coefficient, with a mean value of 0.7, is applied to the measured settlements to refer the comparison to the level of the failure load divided by 3 (the factor of safety generally used for quasi-permanent loads), it is found that the pressuremeter method is altogether satisfactory in estimating the delayed long-duration settlements, which are in most cases the only settlements likely to damage the actual structure. As for the application of elastic theory, the results of Figure 3 would seem to agree with Briaud's results mentioned above : $E = 2E_M$ would have to be used for sands and $E = E_M$ for clays, and the intermediate value $E = 1.5E_M$ for silts and chalk. But the settlements considered by Briaud are settlements under short-duration load. Assuming, for the sake of argument, a ratio of 1 to 2 between the settlement over a relatively short duration and the total long-duration settlement, Figure 3 would lead rather to the conclusion that $E = 4E_M$ for sands and $E = 2E_M$ for clays. However, a closer comparison of the two series of results should perhaps be made.

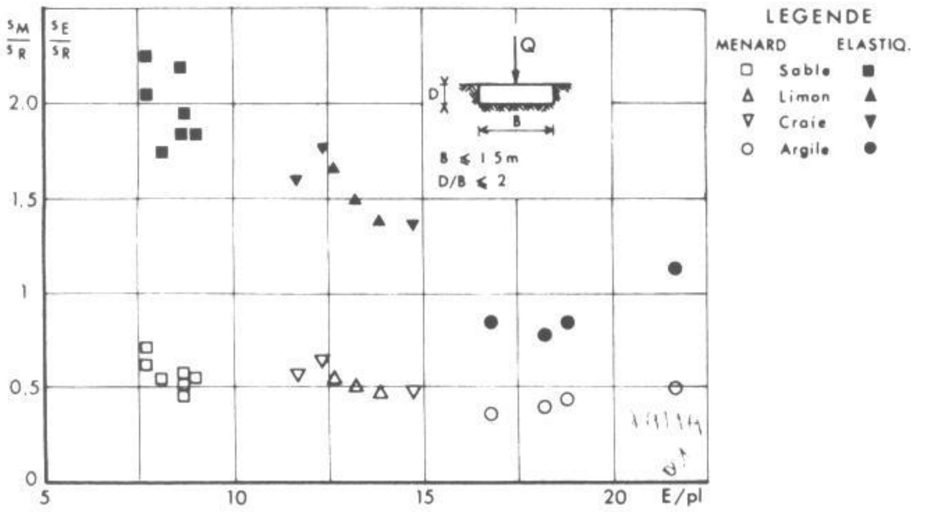

Figure 3. Comparisons between predicted and measured settlements at 10 years under $q_u/2$ (Amar et al., 1987).

The problem is complicated by the fact that the Ménard pressuremeter modulus E_M is sensitive to remoulding, which depends not only on the conditions of placement of the probe, but also on the type of soil; measurement of a modulus on an unloading-reloading cycle included in the test has thus been advocated by several researchers.

DEEP FOUNDATIONS

The results of MPM tests are used for determining the bearing capacity of piles (end bearing and skin friction), as well as for the prediction of axial and transverse (lateral) displacements.

The method for designing axially loaded piles with MPM results, both for bearing capacity and settlement (axial displacements), given in Fascicule 62-V Code are detailed, in particular, in Bustamante and Frank (1999). The design factors k_p (for tip bearing) and q_s (limit unit skin friction) for isolated piles, given in the Code, have been obtained from the original work of Bustamante and Gianeselli (1981) who performed and interpreted numerous full scale static load tests.

At the occasion of the drafting of the new French standard on pile foundations (for buildings and civil engineering works), compatible with Eurocode 7, Bustamante and Gianeselli (2006) have re-analysed the data of the full-scale tests available to date. At present, the revised values of bearing factors and charts they produced form the basis for the MPM method which will finally be published in the French standard, after some minor adaptations.

The main features of the revised Bustamante and Gianeselli (2006) MPM method for the prediction of the bearing capacity of isolated piles are given below.

Tip bearing capacity and limit unit skin friction

The assessment of the bearing capacity of piles from MPM results is the subject of the informative Annex E.3 in Eurocode 7- Part 2 called “Example of a method to calculate the compressive resistance of a single pile" (EN 1997-2, CEN, 2007). It gives the factors and charts of Fascicule 62-V (MELT, 1983), but the principles are the same as for the revised method.

Equation (1) still holds for determining the ultimate end (tip) bearing stress q_u. The limit unit skin friction q_s is directly given as a function of p_l, the type of soil and type of pile.

About the feasibility of the MPM, it is worthwhile noting the figures given by Bustamante and Gianeselli (2006), obtained on 204 sites where pile loads tests have been performed in France and abroad. These figures are given in Table 3. They show that the MPM was or could have been performed on the 204 sites (for 3 sites, there were simply not enough measurements taken).

Table 3. Field and laboratory tests feasibility (Bustamante & Gianeselli, 2006).

Test	carried out to full design length (1)	incomplete test (2)	not carried out (3)	not applicable (4)
MPM pressuremeter (p_l)	155	3	46	0
CPT (q_c)	60	79	23	42
Laboratory tests (c_u, c', φ')	21	67	69	47
SPT (N)	26	54	72	52

(1) including the full length of pile + additional metres below the pile tip
(2) due to premature refusal for CPT; sampling not possible for laboratory tests; soil strength too high for SPT
(3) feasible but not planned when the investigation campaign was decided
(4) considered from the beginning as inadequate with respect to soil nature or strength

The revised rules can be summarised as follows. They are taken from the publication by Bustamante et al. (2009) which is written in English.

Table 4 gives the description of the 418 piles analysed, as well as the group code and the type number needed to enter into the tables and charts of the method.

Table 5 gives the end bearing factor k_p, as a function of the type of pile (through the group code) and the nature of the ground.

The limit unit skin friction q_s is determined, as a function of the limit pressure p_l using one of the lines on Figure 4. Table 6 indicates which skin friction line Qi (i varying from 1 to 10) should be used according to the type of ground and type of pile (through the type number).

Table 4. Description and characteristics of 418 analysed piles (Bustamante et al., 2009).

Group Code	Type No.	Piles[2] Qty	B[3] (mm)	D [4] (m)	Pile Description
1	1	8	500-2,000	11.5-23	Pile or Barrette Bored in the dry
	2	64	270-1,800	6-78	Pile and Barrette Bored with Slurry
	3	2	270-1,200	20-56	Bored and Cased Pile (permanent casing)
	4	28	420-1,100	5.5-29	Bored and Cased Pile (recoverable casing)
	5[1]	4	520-880	19-27	Dry Bored Piles/or Slurry Bored Piles with Grooved Sockets / or Piers (3 Types)
2	6	50	410-980	4.5-30	Bored Pile with a single or a double-rotation CFA (2 types)
3	7	38	310-710	5-19.5	Screwed Cast-in-Place
	8	1	650	13.5	Screwed Pile with Casing
4	9[1]	30	280-520	6.5-72.5	Pre-cast or Pre-stressed Concrete Driven Pile (2 types)
	10	15	250-600	8.9-20	Coated Driven Pile (concrete, mortar, grout)
	11	19	330-610	4-29.5	Driven Cast-in Place Pile
	12	27	170-810	4.5-45	Driven Steel Pile, Closed End
5	13	27	190-1,22	8-70	Driven Steel Pile, Open End
6	14	23	260-600	6-64	Driven H Pile
	15	4	260-430	9-15.5	Driven Grouted[5 or 6] H Pile
7	16	15	-	3.5-2.5	Driven Sheet Pile
1	17	2	80-140	4-12	Micropile Type I
	18	8	120-810	8.5-37	Micropile Type II
8	19	23	100-1,220	8.5-67	SGP(IGU)[5] Micropile (Type III) / or SGP(IGU) Pile
	20	20	130-660	7-39	MRP(IRS)[6] Micropile (Type IV) / or MRP(IRS) Pile

[1] Some types may include several sub-types. [2] Some piles subjected to several tests. [3] Minimum and maximum nominal diameter B. [4] Minimum and maximum full embedment depth D. [5] involving a Single Global Post grouting (“injection globale unique” - IGU).[6] with Multiple repeatable Post grouting (“injection répétitive et sélective” - IRS).

Table 5. Direct design with MPM. Values for the tip bearing factor k_p (Bustamante et al., 2009).

Group Code	Clay & Silt	Sand, Gravel	Chalk	Marl and Limestone	Weathered Rock
1	1.25	1.2	1.6	1.6 *	1.6
2	1.3	1.65	2.0	2.0	2.0
3	1.7	3.9	2.6	2.3	2.3
4	1.4	3.1	2.4	2.4 *	2.4 *
5	1.1	2.0	1.1	1.1 *	1.1 *
6	1.4	3.1	2.4	1.4 *	1.4 *
7	1.1	1.1	1.1	1.1 *	1.1 *
8	1.4	1.6	1.8	1.8	1.5*

* A higher k_p value can be used but must be proven by a load test.

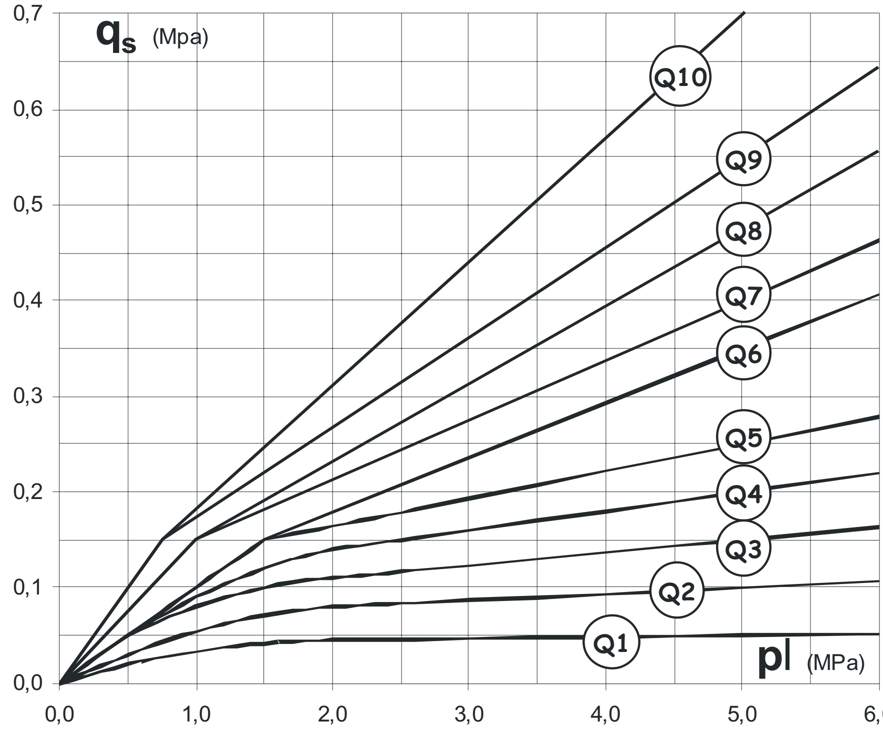

Figure 4. Direct design with MPM data. Chart for skin friction q_s (Bustamante et al., 2009).

Table 6. Direct design with MPM data. Selecting the Qi line to obtain the limit unit skin friction values q_S (Bustamante et al., 2009).

Pile Type No.	Clay, Loam	Sand, Gravel	Chalk	Marl, Limestone	Weathered Rock
1	Q2	Q2*	Q5	Q4	Q6**
2	Q2	Q2	Q5	Q4	Q6**
3	Q1	Q1	Q1	Q2	Q1**
4	Q1	Q2	Q4	Q4	Q4**
5	Q3	Q3*	Q5	Q4	Q6
6	Q2	Q4	Q3	Q5	Q5**
7	Q3	Q5	Q4	Q4	Q4**
8	Q1	Q2	Q2	Q2	Q2**
9	Q3	Q3**	Q2	Q2**	(a)
10	Q6	Q8	Q7	Q7	(a)
11	Q2	Q3	Q6**	Q5**	(a)
12	Q2	Q2**	Q1	Q2**	(a)
13***	Q2	Q1	Q1	Q2	(a)
14***	Q2	Q2	Q1	Q2**	(a)
15***	Q6	Q8	Q7	Q7	(a)
16***	Q2	Q2	Q1	Q2**	(a)
17	Q1	Q1	Q1	Q2	Q6**
18	Q1	Q1	Q1	Q2	Q6**
19	Q6	Q8	Q7	Q7	Q9**
20	Q9	Q9	Q9	Q9	Q10**

* If ground properties permit. ** Use of a higher value must be proven by a load test.
*** Cross section and perimeter estimated according to Fascicule 62 (MELT, 1993)
(a) For pile groups No.9-16 and if rock condition permits penetration, choose the q_s value proposed for marl and limestone or a higher one if this can be proven either by a load test or by reference to an existing example in the same local area.

The following system of factors of safety is used in the Fascicule 62-V Code.

Design loads Q_d must be such that at:

ULS, under fundamental combinations: $-Q_{tu} / 1.4 \leq Q_d \leq Q_u / 1.4$
and under accidental combinations: $-Q_{tu} / 1.3$(1) $\leq Q_d \leq Q_u / 1.2$
where:
$Q_u = Q_{pu} + Q_{su}$ is the ultimate limit load in compression (= end bearing + skin friction), and
$Q_{tu} = Q_{su}$ is the ultimate limit load in tension (= skin friction alone).

SLS, under rare combinations: $-Q_{tc} / 1.4$(2) $\leq Q_d \leq Q_c / 1.1$
and under quasi permanent combinations: 0(3) $\leq Q_d \leq Q_c / 1.4$
where Q_c or Q_{tc} are the corresponding (compression and tension) creep loads:
$Q_c = 0.5\ Q_{pu} + 0.7\ Q_{su}$ and $Q_{tc} = 0.7\ Q_{su}$ for non displacement piles
$Q_c = 0.7\ Q_{pu} + 0.7\ Q_{su} = 0.7\ Q_u$ and $Q_{tc} = 0.7\ Q_{su}$ for displacement piles.

The adaptation of these rules to the system of safety of Eurocode 7 requires, in particular, having a precise knowledge of the scatter of the ratio of the predicted bearing capacity to the measured bearing capacity. Following the requirements and possibilities of Eurocode 7, the French pile standard will give the values for the correlation factors ξ for the bearing capacity of piles from static load test results or from measurements of ground properties (often referred to as the "model pile" approach), as well as the model factors γ_{Rd} for each design method (a MPM method and a CPT method will be offered). It will also allow the "alternative " approach mentioned in Eurocode 7 for the calculation of the bearing capacity from ground tests results, which does not make use of the correlation factors ξ, but assumes that a correct estimate of the characteristic values of end bearing and shaft friction can be made directly from the formulae or charts (called the "ground model" approach). In this latter approach (resembling the traditional or Fascicule 62-V approach), the French standard might give values of the model factors slightly different from the ones for the "model pile" approach.

It should be noted that the way Serviceability limit states are treated in Fascicule 62 – Titre V is different from Eurocode 7-Part 1. The former introduces requirements on the creep load, easily determined from bearing capacity approaches, the latter relies essentially on settlement (axial displacement) estimates.

Axial displacements

The determination of the load-settlement curve of a single pile under axial loading is based on the concept of skin friction mobilisation curves also known as t-z curves.

The Fascicule 62-V Code (MELT, 1993) suggests, in case a settlement estimate must be made, to use the τ-s curves (unit skin friction – local displacement curves) and q-s_p curve (end load – end displacement curve) proposed by Frank and Zhao (1982) and Frank (1984), as shown on Figure 5, with k_τ and k_q given as functions of the pressuremeter modulus E_M and the diameter B of the pile :

$k_\tau = 2.0\ E_M/B$ and $k_q = 11.0\ E_M/B$ for fine soils
$k_\tau = 0.8\ E_M/B$ and $k_q = 4.8\ E_M/B$ for granular soils

1 For micropiles, $-Q_{tu}/1.2$
2 For micropiles, $-Q_{tc}/1.1$
3 For micropiles, $-Q_{tc}/1.4$

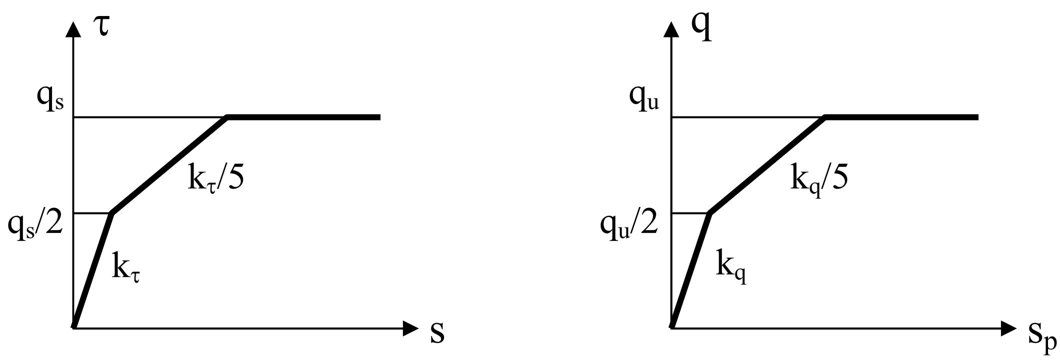

Figure 5. Fascicule 62-V : MPM τ-s and q-s_p curves after Frank and Zhao (1982).

Examples of the use of this MPM method for predicting load-settlement curves of piles are given, e.g. in Frank (1984), Bustamante et al. (1989) and Bustamante and Frank (1999).

Figures 6 and 7 are such examples of the use of the Frank and Zhao (1982) MPM method for the analysis of full scale static load tests.

The Koekelare pile of Figure 6 is a cased screw pile ϕ350mm/650mm constructed in an Ypresian clay. It can be seen that the prediction of the load-settlement curve is excellent.

Figure 7 shows all the results of the prediction exercise which was organised for the International Symposium ISP5-PRESSIO 2005, taking place at the occasion of the '50 years of pressuremeters' (Reiffsteck, 2006 and 2009). The pile is a CFA (continuous flight auger bored pile) with a diameter B = 0,5 m and a length D = 12 m. The pile is embedded in a 9,6 m thick clay layer, below a 2.4 m thick silt layer. The watertable is located 1.8 m below ground level. It is interesting to note that the predictions made by Robas and Kuder (2006) and by Said et al. (2006) – which are the closest predictions to the whole initial part of the measured load-settlement curve, both used the Frank-Zhao MPM method, and were established completely independently.

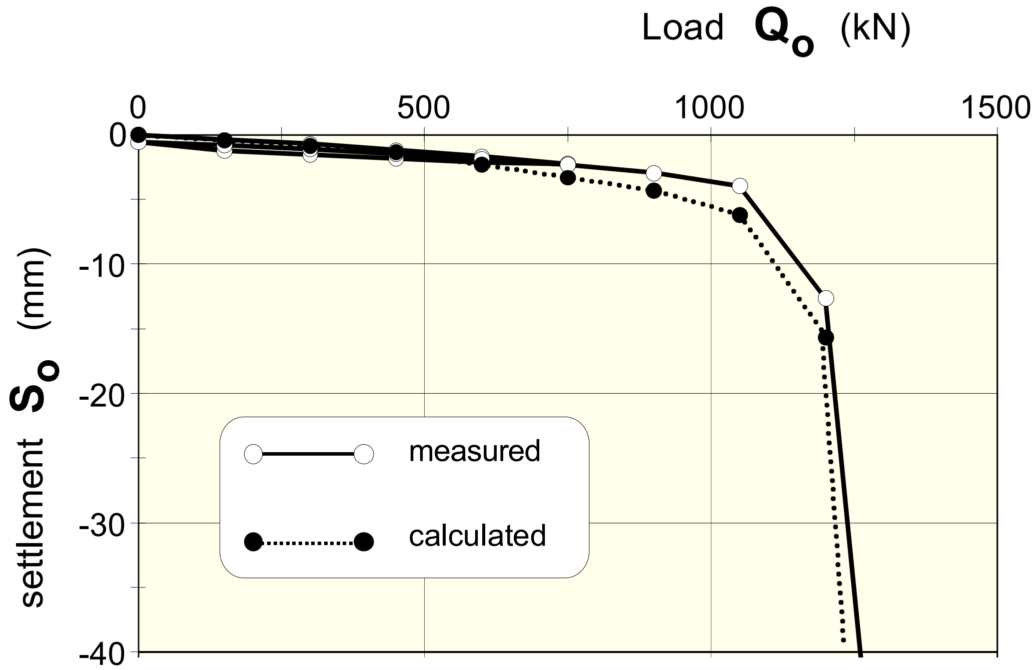

Figure 6. Comparison of measured and calculated load-settlement relationship for the Koekelare pile (Bustamante and Frank, 1999).

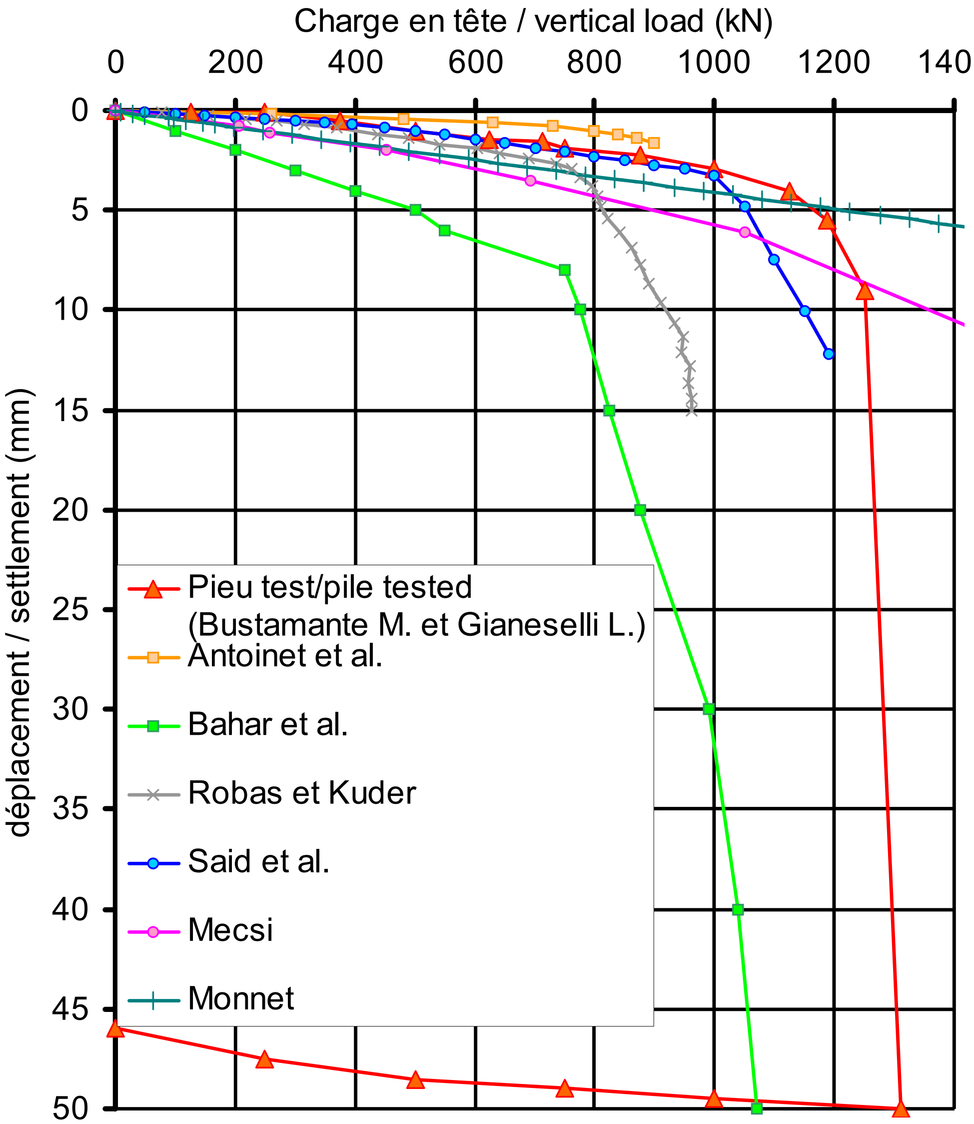

Figure 7. Comparison of the experimental curve with the participants' predictions (Reiffsteck, 2006)

Transverse displacements

The methods using the subgrade reaction modulus (or p-y reaction curves, p – reaction pressure, y – horizontal displacement) are now well known for the design of piles under lateral loads. These methods, which consider the pile as a beam on linear or non-linear elastic springs, are very much used in France, precisely because of the development of the MPM test which provides the soil engineer with both strength and deformation information about the soil.

The research carried out by the LPCs have concerned not only overturning loads at the head, but also transverse thrusts due to lateral soil movements (at the toe of an embankment, for instance). In this latter case, the pile soil movement y is replaced by the relative displacement y – g, where g is the displacement of the soil in absence of the pile – see below.

The basic method from MPM tests (Ménard, 1962-1969) is detailed in the book by Baguelin et al (1978). From the results of the test at the considered depth (E_M, pressuremeter modulus; p_f, creep pressure and p_l, limit pressure), the reaction curve (p, y) of a single pile at a given depth, is established, for long duration loadings, as shown on Figure 8.

The subgrade reaction modulus k_s = p/y of part OA is obtained from the Ménard settlement formula (equation 4) with $q-\sigma_v$ = p and s = y, and $E_c = E_d = E_M$ and making, for a strip footing, λ_c =1.5 and λ_d =2.65.

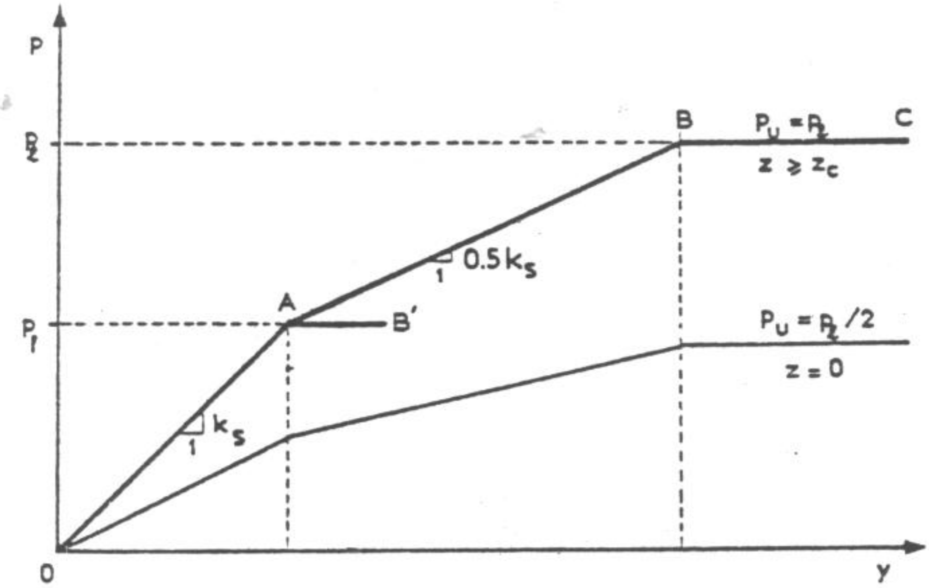

Figure 8. Reaction curve from Ménard pressuremeter MPM for long duration loadings (Baguelin et al., 1978)

Above the creep pressure p_f ($p_f = p_l/2$ can be used as an estimate), the non linear effect is taken into account by reducing the tangent reaction modulus by one half (segment AB on Figure 8). Finally, the ultimate pressure on the pile is taken as being the limit pressure measured with the MPM (segment BC). However, in current practice, the creep pressure should normally not be exceeded and the displacements should be determined using the law OAB', which is on the safe side in the case of loads at the head of the pile (it might be the contrary when the pile is submitted to lateral thrusts from the soil – see below).

The p-y curve is, in principle, modified for depth values z lower than a critical depth z_c, due to surface effect. For z = 0, the pressures are divided by 2 for the same displacement y (or y –g) and are then linearly interpolated until z = z_c. For cohesive soils z_c is taken equal to 2B (B is the diameter of the pile) and for granular soils it is taken equal to 4B.

The design of piles subjected to lateral soil thrusts, created by nearby slopes for instance, is based on the 'free soil displacement' concept. It is assumed that the lateral reaction curve now links the lateral reaction pressure p to the 'relative' displacement Δy = y - g, where y is the equilibrium soil-pile lateral displacement sought, and g is the free lateral soil displacement (or displacement in absence of the pile) – see e.g., Bigot et al.(1982), Frank (1984 and 1999). The Fascicule 62-V Code suggests a method for predicting g(z), as a function of depth z, of the characteristics of the slope, of the characteristics of the underlying soft soil and of the position of the pile.

It must be admitted that there are not as many cases of comparison of the prediction of the MPM method with full-scale test results for piles under transverse loadings, as in the case of the bearing capacity of piles under axial loading. However, a certain umber of such comparisons are available, in particular some experiments carried out by the LPC since the 1970s (see Baguelin et al., 1978).
As for those with determination of the reaction curves along the shaft, the experiment on Provins site (which will be briefly reported below) and different research projects at Plancoët on isolated piles, on a group of two piles and on a group of six piles (see e.g. Baguelin et al., 1985) must be mentioned. Also, the measurements taken during 16 years on a steel pipe driven through an unstable slope at Sallèdes (Puy-de-Dôme) are very valuable (Frank and Pouget, 2008). For the group of two piles at Plancoët it is interesting to note that the reaction measured on the trailing pile

is found to be reduced by a factor of 0.4 to 0.5 relatively to the leading (front) pile, the distance between the 2 axes being 3 times the frontal width.

From the various experimental evaluations, Baguelin et al. (1978) conclude that the standard MPM pressuremeter method (Figure 8) is, in general, pessimistic for quick monotonic loadings. It tends to overestimate the head displacements and maximum bending moments of piles submitted to loads at their head, and thus is conservative. In reality foundations must often sustain cyclic and/or long term loads and the soil can be severely damaged by the installation of the piles, all being parameters very difficult to quantify in everyday practice. These different facts allow one to think that the method is quite acceptable.

The experiment on the site of Provins in 1974 is interesting because the behaviour of an instrumented pile was examined under head loading, and also when being submitted to lateral thrusts due to the construction of an embankment. The pile is a steel instrumented pipe, of OD = B = 0.926 m and thickness e = 0.015 m. Furthermore, the 4 stages of the experiment (initial head loading to 120 kN, then embankment construction to a height of 3.80 m, to a height of 6.80 m and after 3 months of consolidation under this final height) have been analysed in detail by using the different pressuremeter prediction methods (Bigot et al., 1982). Here only the main results concerning the head loading and the Ménard MPM method are discussed for conciseness.

Figure 9 compares the measured values M of bending moments (left) and displacements (right) (M) for the last level of applied load at the head (120 kN shear load at 0.20 m from ground level) to the results of 3 prediction methods :
- method A, with Ménard (MPM) reaction curve (Fig. 8)
- methods B and C, with p-y reaction curves constructed on the basis of self-boring pressuremeter tests results (not discussed here).

In the surface layer (silt and clay), the predominant one for head loading, the use of the MPM method of Figure 8 yields a mean soil reaction modulus:
$E_{sM} = k_s B = 2900$ kPa (method A).

It is clear from Figure 9 that the MPM method (method A) is on the safe side for short duration head loadings: the maximum bending moment is slightly overestimated and the displacements are overestimated by a factor of 2. This is consistent with the conclusions of Baguelin et al. (1978). This also shows that for long duration loadings at the head, the MPM method is quite acceptable, given all the uncertainties…

Figure 10 compares the measured values M of bending moments (left) and displacements (right) (M) after 3 months of consolidation under the final height of the embankment to the results of the same 3 prediction methods (A, B and C). Here, the difficulty is the prediction of the bending moments, as it is a 'displacement-imposed' problem. The measured bending moment (curve M) in the upper part is well predicted by the MPM method (curve A). In the lower part the method overestimates the bending moment by a factor of around 1.8, which is largely on the safe side.

The full scale experiment of Sallèdes (steel pipe pile installed through an unstable slope) confirmed the great difficulty in predicting the long duration behaviour of piles undergoing lateral thrusts from a moving ground; it is clear that the MPM method overestimates the bending moments of such piles (see Frank and Pouget, 2008, for the extensive analysis of this unique experiment).

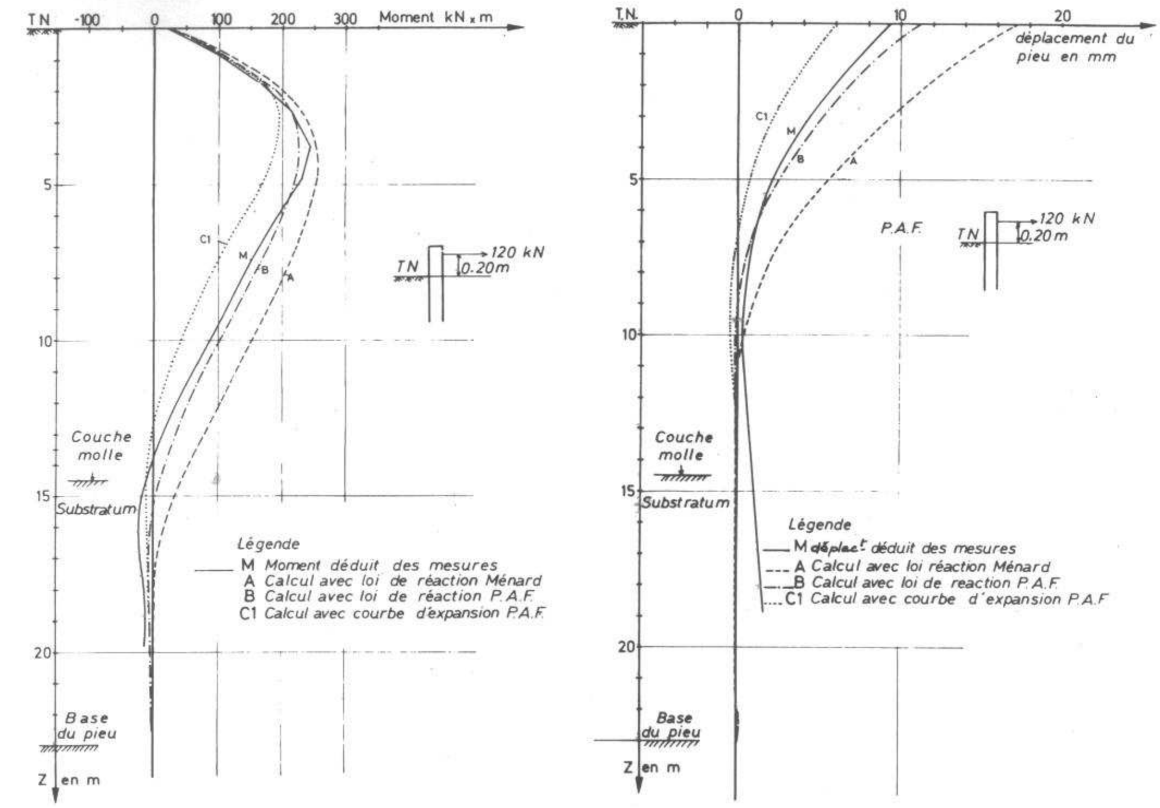

Figure 9. Provins pile. Comparison of measured and calculated bending moments and displacements for head loading (Bigot et al. 1982).

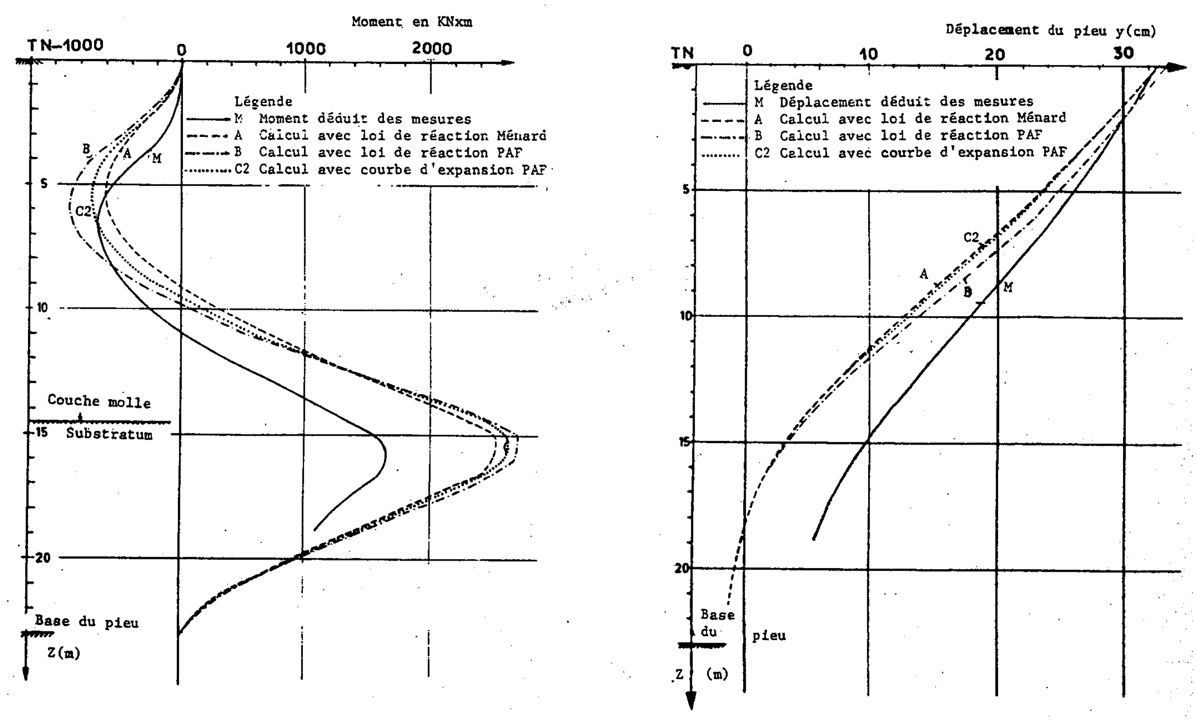

Figure 10. Provins pile. Comparison of measured and calculated bending moments and displacements after 3 months of consolidation under final height of embankment (Bigot et al. 1982).

CONCLUDING REMARKS

This paper has briefly explained the present design rules used in France for designing foundations.

In particular, the French code “Fascicule 62 – Titre V”, adopted in 1993 for the design of shallow and deep foundations of civil engineering works has been presented, as well as the revised method for the bearing capacity of piles (Bustamante et Gianeselli, 2006).

Attention has been focused on the use of Ménard Pressuremeter (MPM) as an efficient tool for designing foundations. The Ménard pressuremeter by providing both a failure parameter (the limit pressure) and a deformation parameter (the pressuremeter modulus E_M) allows to tackle not only bearing capacity problems, but also all the problems linked to the displacements of foundations.

At the present time, where the design rules (‘geotechnical models’) for foundations must be adapted in order to fulfil the requirements of Eurocode 7 (CEN, 2004), it is believed that the approach on which Fascicule 62-Titre V has been developed, especially with MPM, will be easily updated and converted into national standards complementing Eurocode 7. The MPM rules are not only flexible, in the sense that they can incorporate easily the new experimental findings, but also because they are a tool for checking all limit states, whether the ultimate ones or the serviceability ones. Eurocode 7 is a code with advocates explicitly the ‘displacement design’ of foundations (compared to the ‘capacity’ or traditional design), especially for serviceability checks. The MPM based design methods are obviously able to face this challenge!

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