[Geotech] SEMINAR MONDAY APRIL 29TH,2013

Georgia Tech, Atlanta

Pressuremeter testing and foundation design

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Contents

- Introduction to design rules. Why use the MPT ?
- Shallow foundations
- Axial capacity of piles
- Transverse capacity of piles
- Conclusions

Main references

- Work of Ménard (1954-1967), Cambefort, Gambin, Cassan, etc.
- Work of Laboratoires des ponts et chaussées (LPCs) from 1963-… by Amar, Baguelin, Bustamante, Canépa, Frank, Garnier, Jézéquel and others
- 3 synthetical papers:

- Gambin, M., Frank, R. (2009) "Direct Design Rules For Piles Using Ménard Pressuremeter Test", Proc Int Foundation Congress and Equipment Expo'09 (IFCEE'09), Orlando, Florida, ASCE
- Canépa, Y., Garnier J. (2004). "Etudes expérimentales du comportement des fondations superficielles - Etat de l'art", Proc. Int Symp. Fondsup 2003
- Bustamante M., Frank R. (1999) "Current French design practice for axially loaded piles". *Ground Engineering* (note : MPM & CPT)

- Foundation engng code for public works (Fascicule 62-V, 1993) and course taught at Ecole nationale des ponts et chaussées – under re-drafting to match Eurocode 7…

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- Introduction to design rules. Why use the MPM?
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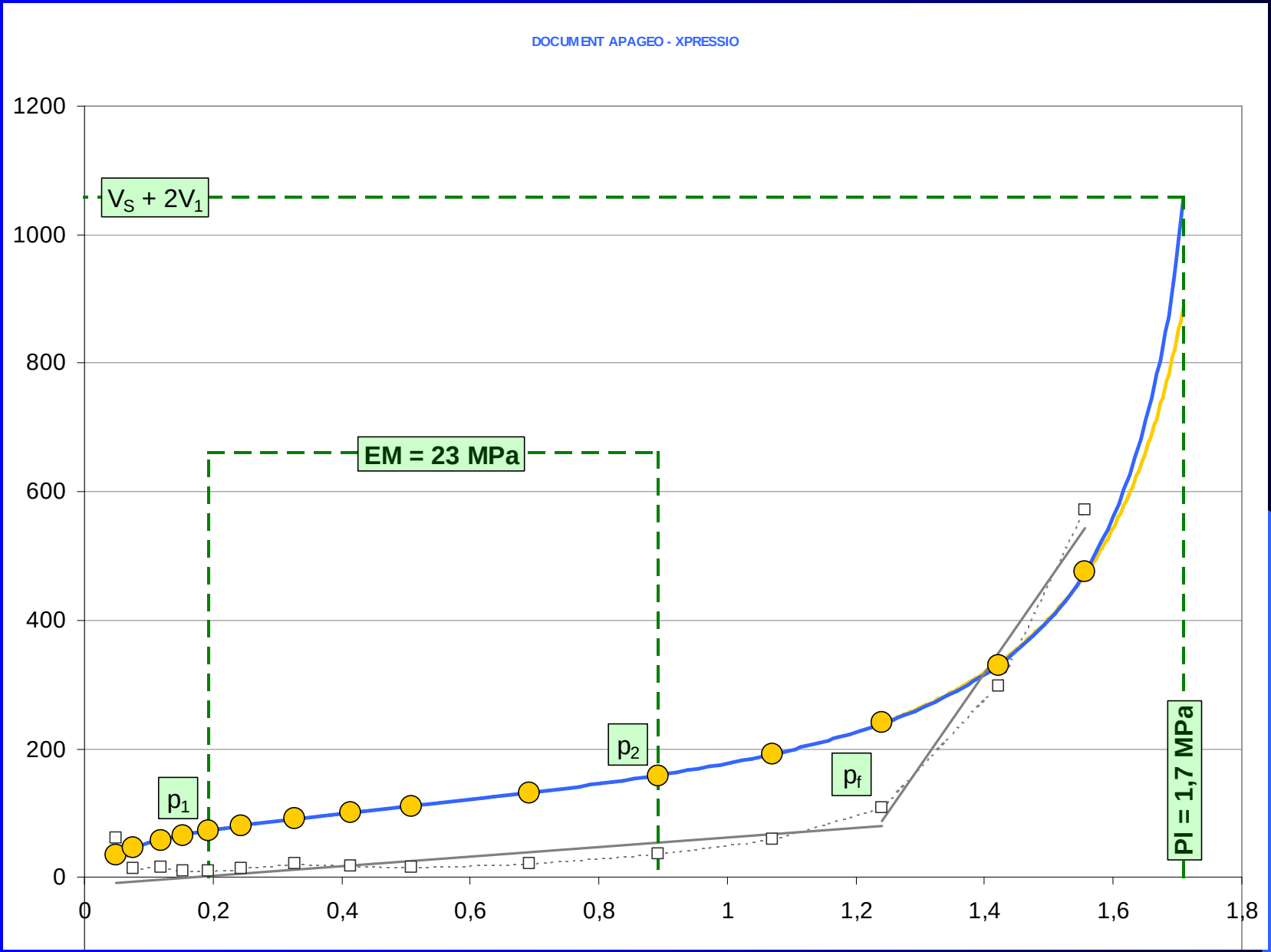
The Ménard pressuremeter test (MPT)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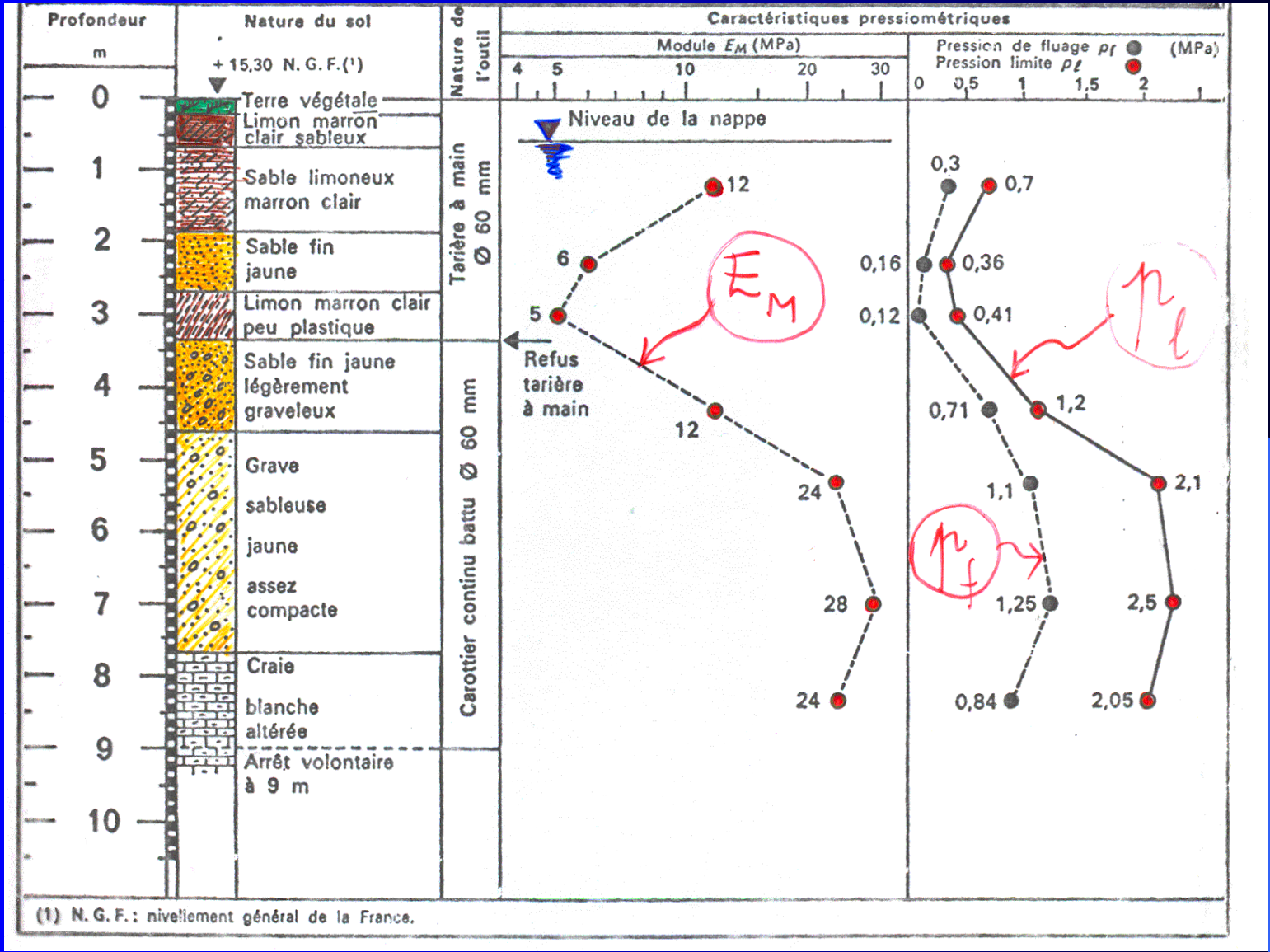
1,8
$V_S + 2V_1$
EM = 23 MPa
p_1
p_2
p_f
Pl = 1,7 MPa

Pressuremeter soil profile



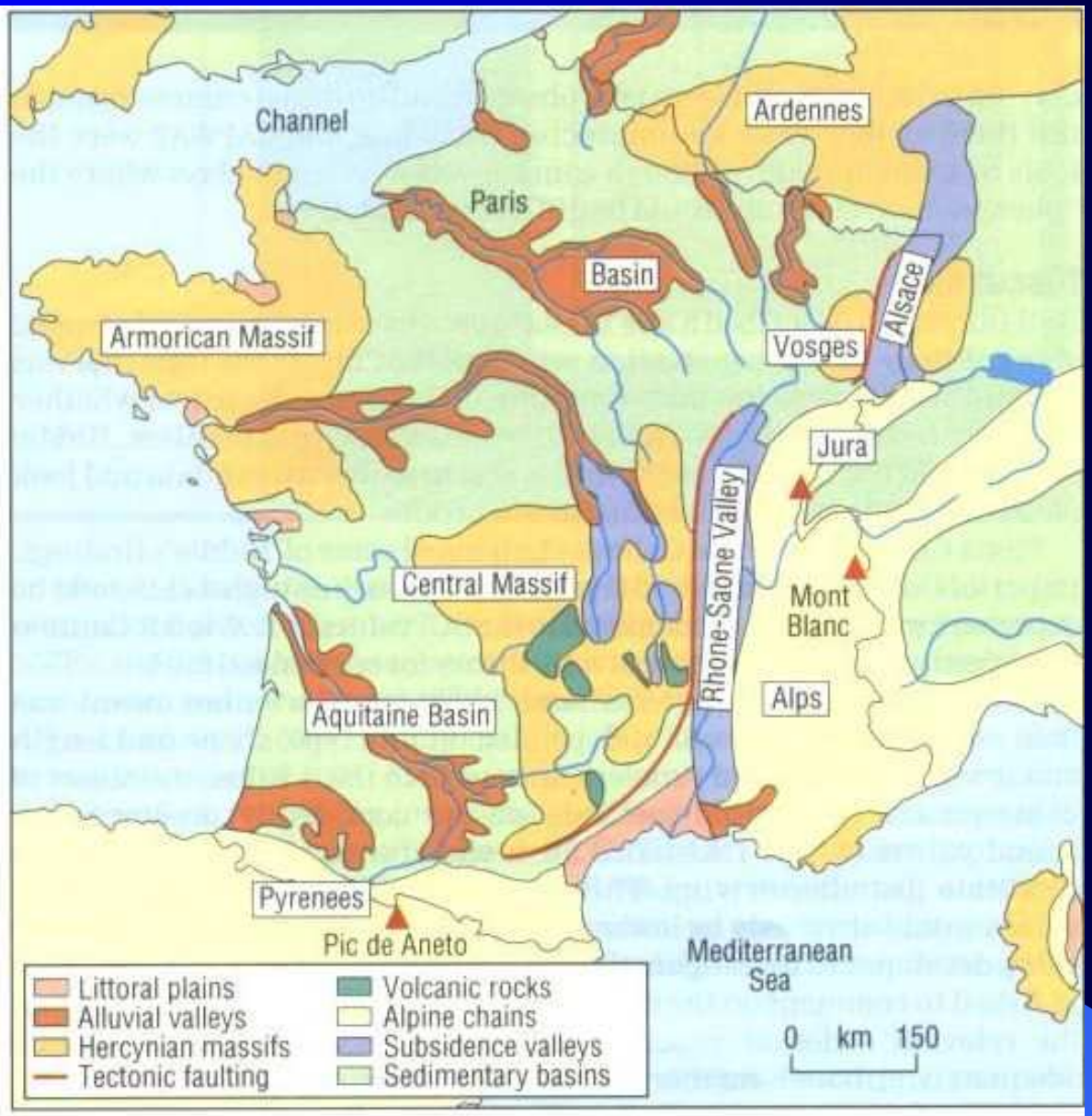


Figure 1: Main geological units of France (Frank and Magnan, 1996).

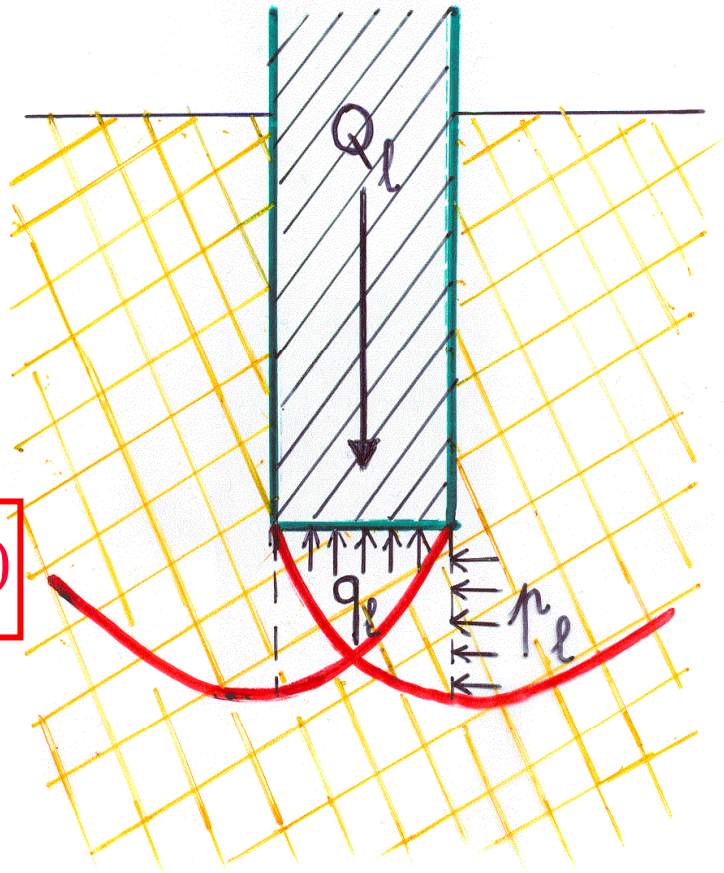
Table 3. Field and laboratory tests feasibility (Bustamante & Gianeselli, 2006).

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

Bearing capacity : the pressuremeter ‘faith’

$$q_u = q_o + k_p\,(p_{le} - p_o)$$



$$q_u = f(E, \nu, c, \varphi, \psi \ldots\ldots)$$

- The pressuremeter test can be performed in all types of soils and soft rocks

- It provides the soil engineer with both a failure parameter and a deformation parameter measured in situ

Use of in situ test results

- The **direct** method ('semi-empirical') should be preferred i.e direct rules for determining the bearing capacity of foundations from the test results :
  (and possibly the displacements)

R_d,N,q_c, etc.

Specific advantages of the Pressuremeter

with the limit pressure p_{LM} **:**

- bearing capacity of shallow foundations (q_p)
- axial bearing capacity of deep foundations (q_p and q_s)

With the pressuremeter modulus E_M **:**

- settlement of shallow foundations (s)
- analysis of the behaviour of deep foundations under transverse loading

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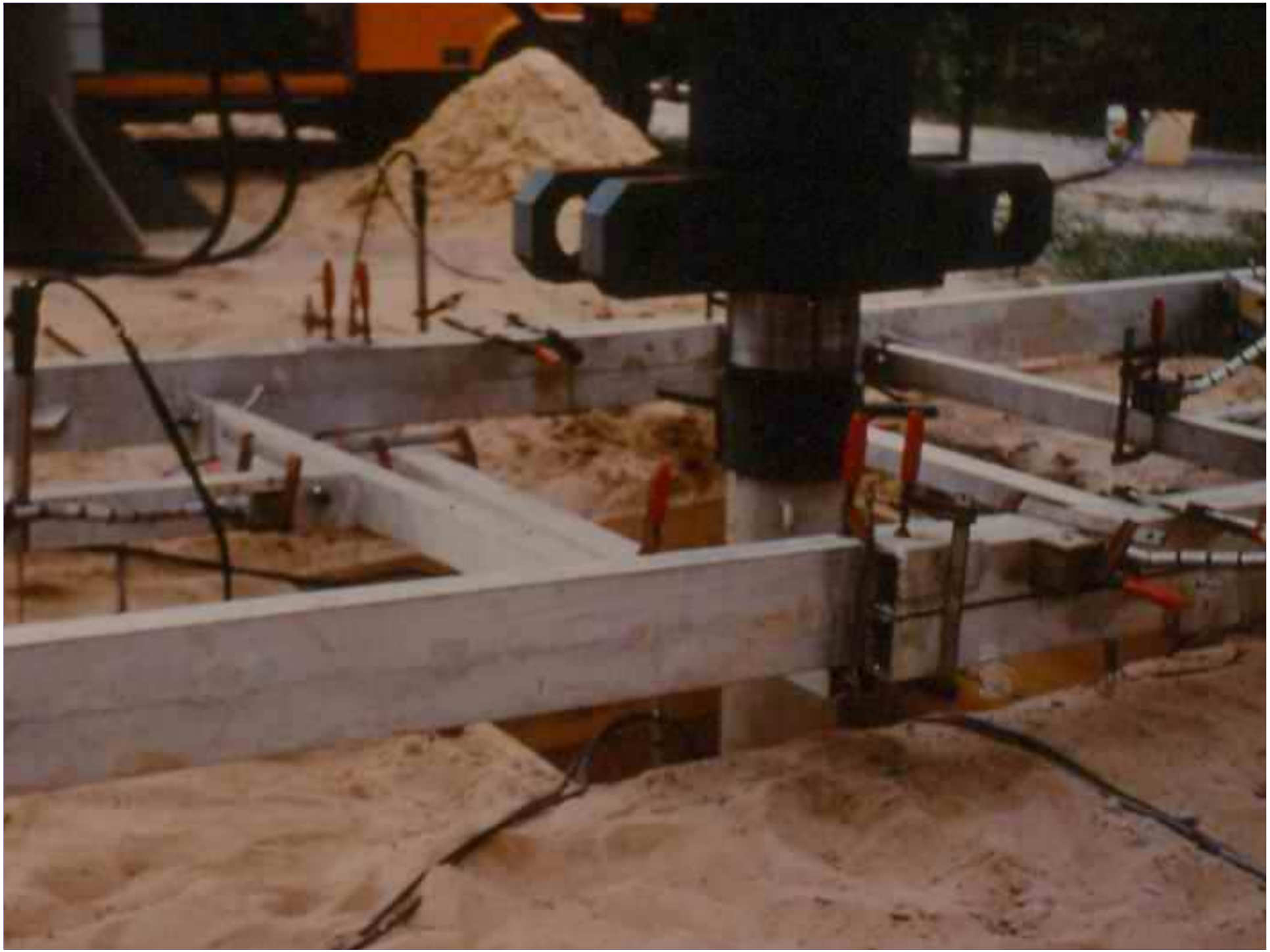
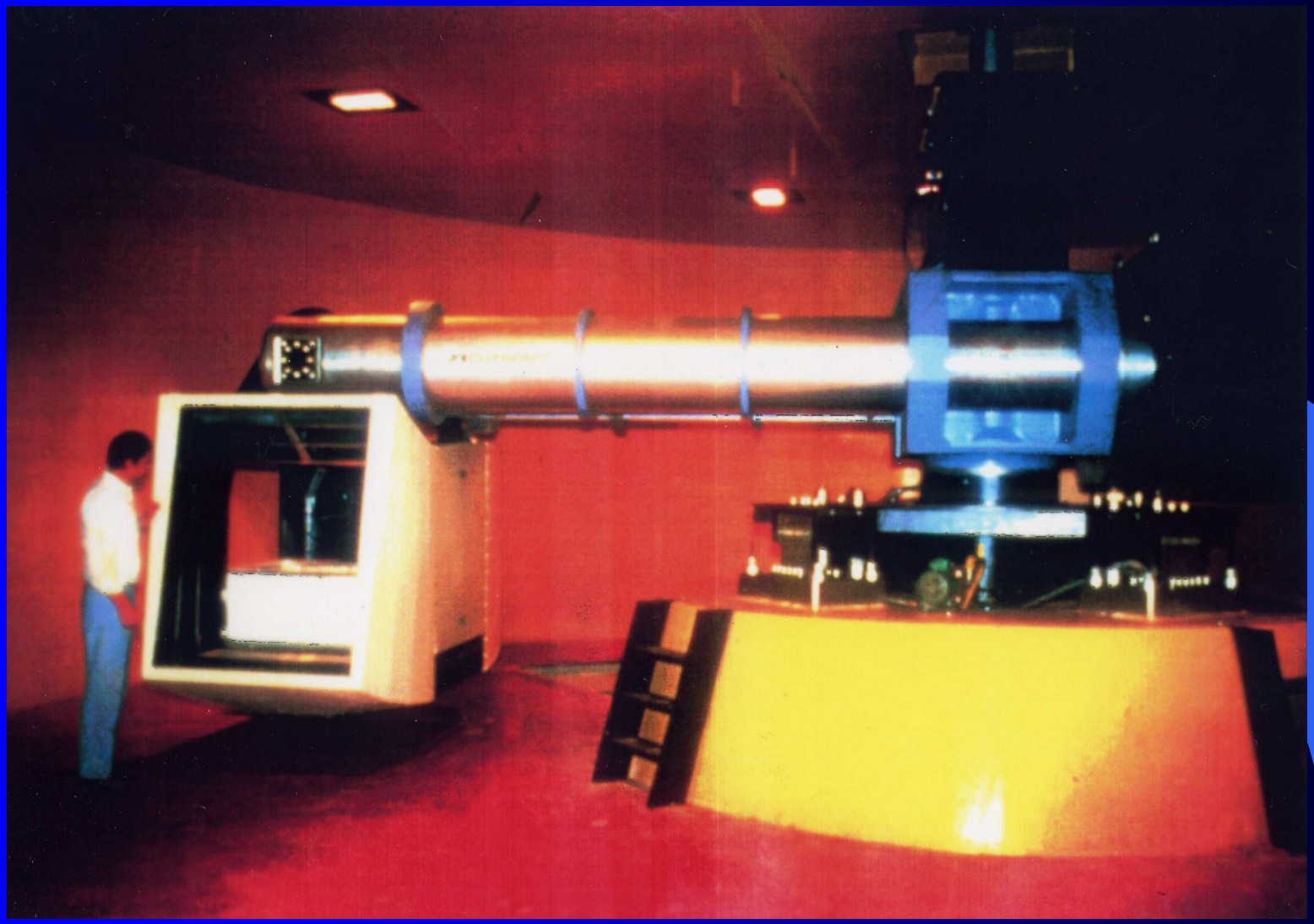
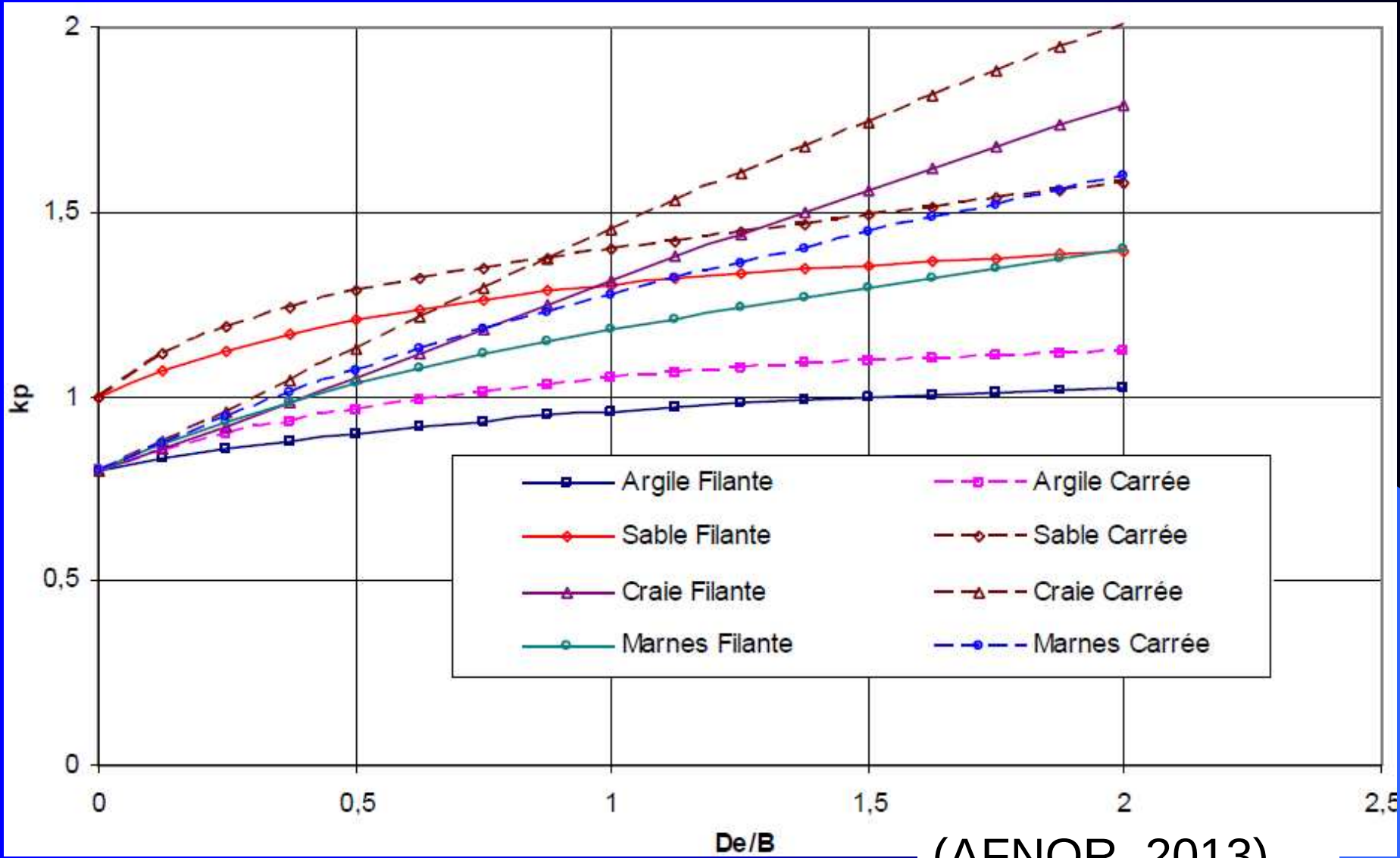
Tests at Jossigny & La Benne (1980s)



LCPC Centrifuge (Nantes) 1985 -…



Bearing factors - strip and square footings



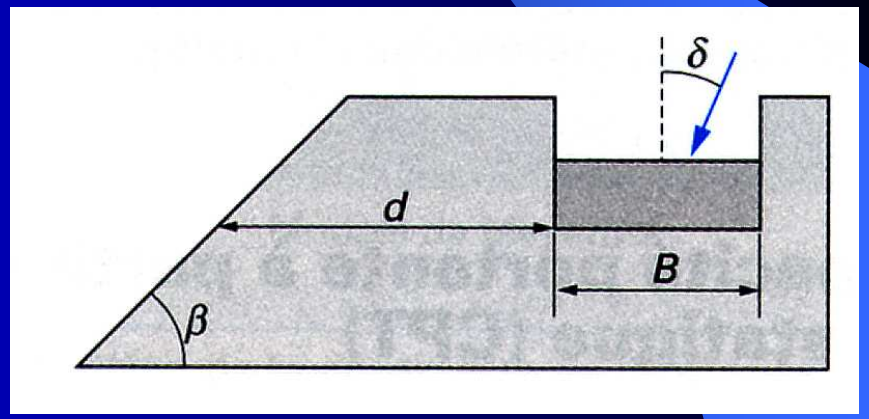
(AFNOR, 2013)

Design bearing capacity

Reference stress $q_{réf}$:

$$q_{réf} \leqslant \frac{1}{\gamma_q} k_p p^*_{\ell e} i_{\delta\beta} + q_0$$

$i_{\delta\beta}$ reduction factor depends on β, δ, d/B_e, D_e/B_e

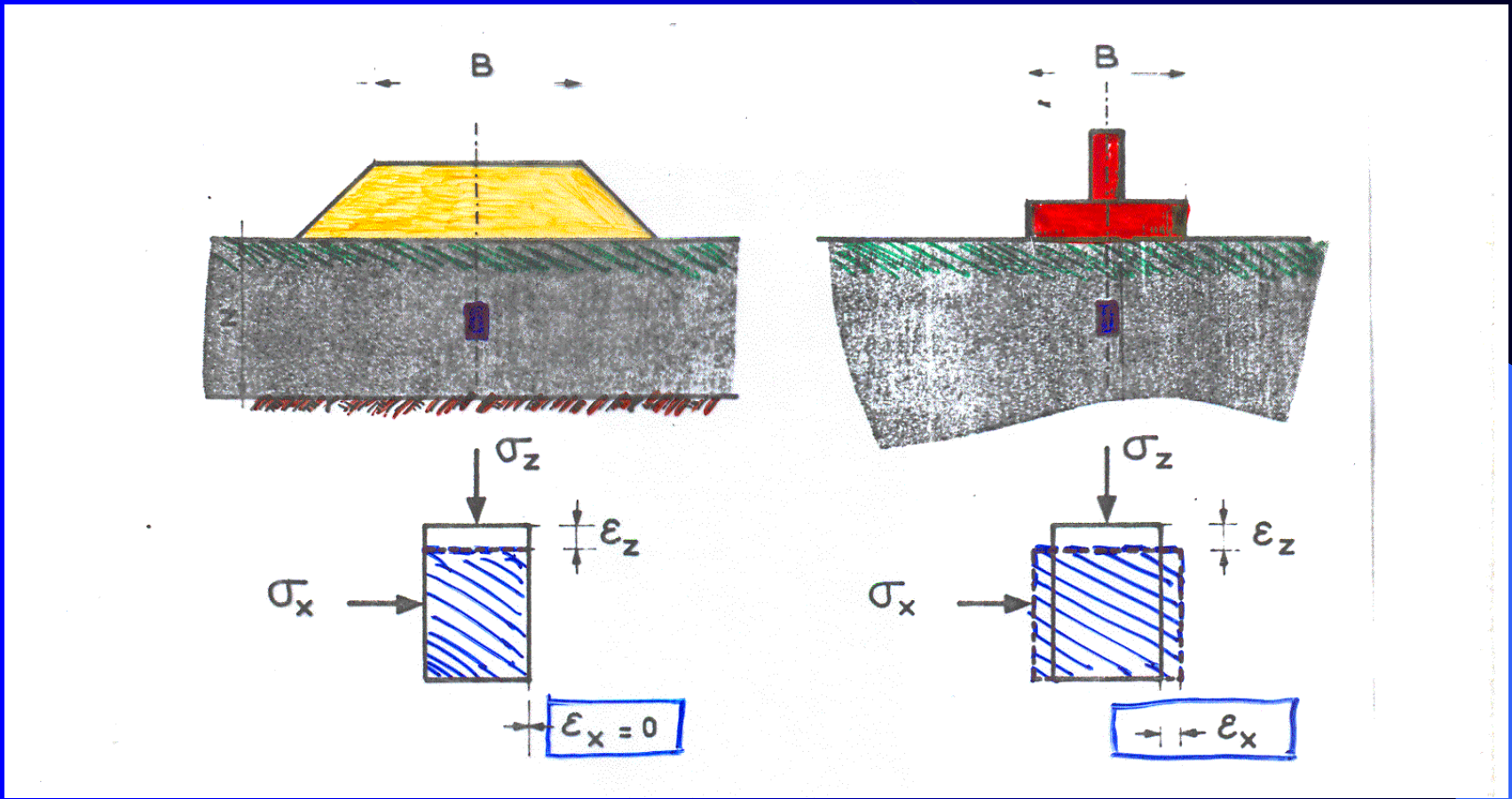


With (model factor) $\gamma_{RD} \approx 1.2$
and

$\gamma_{R;v} = 1.4$ for ULS

$\gamma_{R;v} = 2.3$ for SLS

Pressuremeter method for settlement prediction



Ménard's formula for settlement

$$s(10 \text{ years})\ s_c + s_d$$

$s_c = (q - \sigma_v)\,\lambda_c B\alpha/9E_c$ is the volumetric settlement

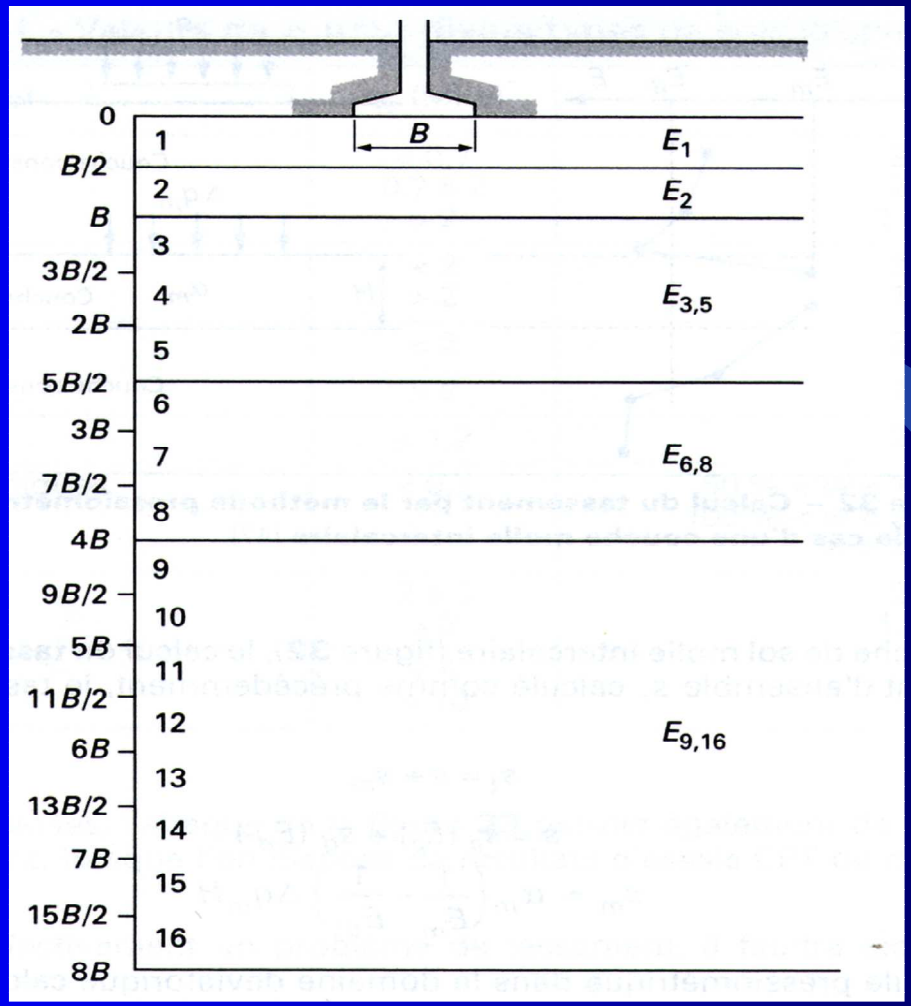
$s_d = 2\,(q - \sigma_v)\,B_0(\lambda_d B/B_0)^{\alpha}/9E_d$ is the deviatoric settlement

Rheological coefficient α

Type	Tourbe	Argile		Limon		Sable		Sable et gravier	
	α	E/p_ℓ	α	E/p_ℓ	α	E/p_ℓ	α	E/p_ℓ	α
Surconsolidé ou très serré		> 16	1	> 14	2/3	> 12	1/2	> 10	1/3
Normalement consolidé ou normalement serré.....	1	9 à 16	2/3	8 à 14	1/2	7 à 12	1/3	6 à 10	1/4
Sous-consolidé altéré et remanié ou lâche.....		7 à 9	1/2	5 à 8	1/2	5 à 7	1/3		

Type	Roche
	α
Très peu fracturé.....	2/3
Normal	1/2
Très fracturé ..	1/3
Très altéré	2/3

Selection of moduli E_c and E_d



$$E_c = E_1 \text{ and } \quad \frac{4}{E_d} = \frac{1}{E_1} + \frac{1}{0{,}85\ E_2} + \frac{1}{E_{3,5}} + \frac{1}{2{,}5\ E_{6,8}} + \frac{1}{2{,}5\ E_{9,16}}$$

LPCs 1978-1987

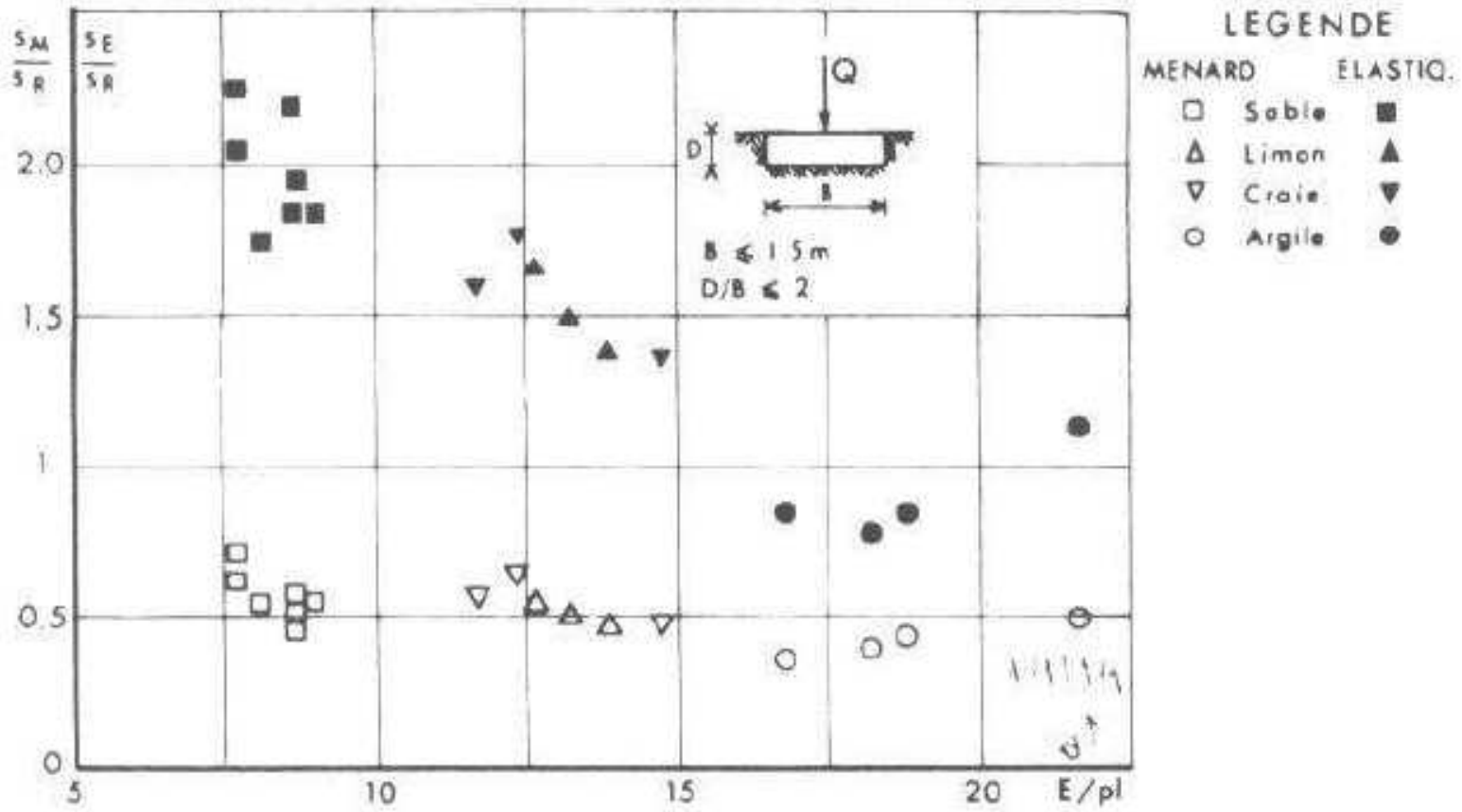


Figure 3. Comparisons between predicted and measured settlements at 10 years under $q_u/2$ (Amar et al., 1987).

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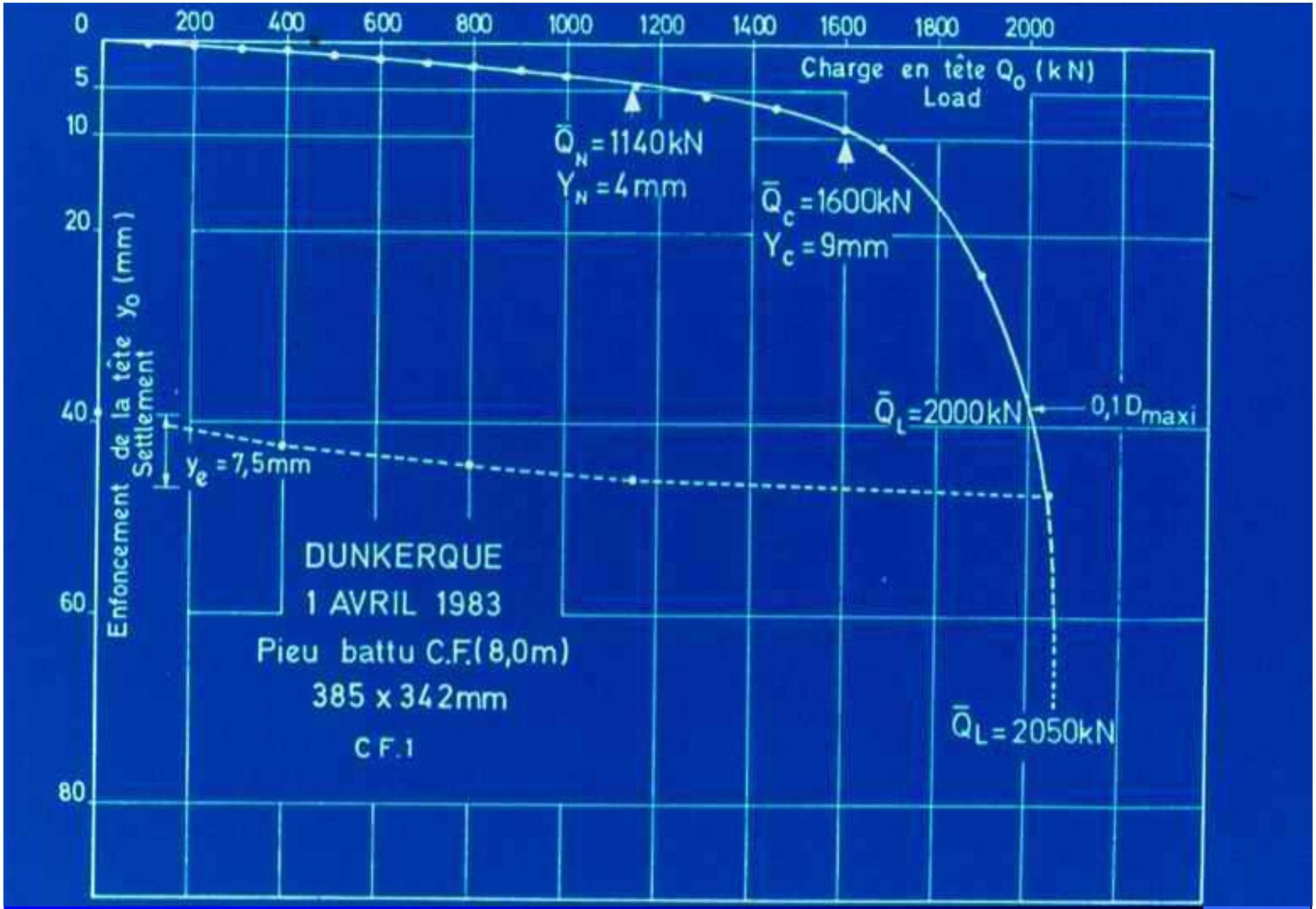
Pile load tests at Dunkirk (1985)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1200
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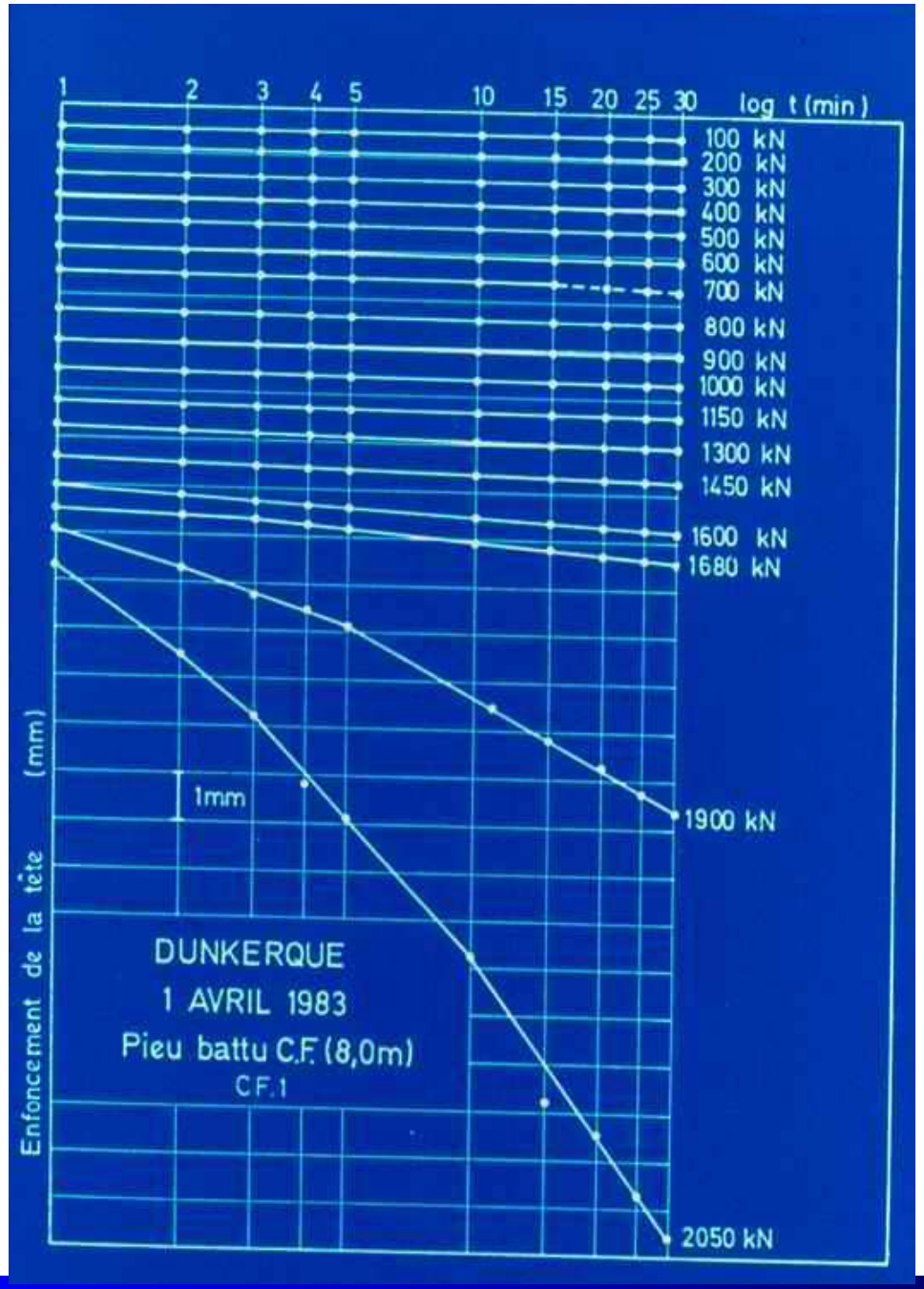
2000
5
10
20
40
60
80
Charge en tête Q_o (kN)
Load
Enfoncement de la tête y_o (mm)
Settlement
$\bar{Q}_N$ = 1140kN
Y_N = 4mm
$\bar{Q}_c$ = 1600kN
Y_c = 9mm
$\bar{Q}_L$ = 2000kN
0,1 D_{maxi}
y_e = 7,5mm
DUNKERQUE
1 AVRIL 1983
Pieu battu C.F.(8,0m)
385 x 342mm
C F.1
$\bar{Q}_L$ = 2050kN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25
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200 kN
300 kN
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600 kN
700 kN
800 kN
900 kN
1000 kN
1150 kN
1300 kN
1450 kN
1600 kN
1680 kN
1900 kN
2050 kN
1mm
Enfoncement de la tête (mm)
DUNKERQUE
1 AVRIL 1983
Pieu battu C.F. (8,0m)
C.F.1

DUNKERQUE
1 AVRIL 1983
Pieu battu C.F.(8,0m)
$\overline{Q}_c$ = 1600 kN
$\overline{Q}_L$ = 2000 kN
CF.1
Pente α mm (10 - 30min)
Load
Charge en tête Q_0(kN)
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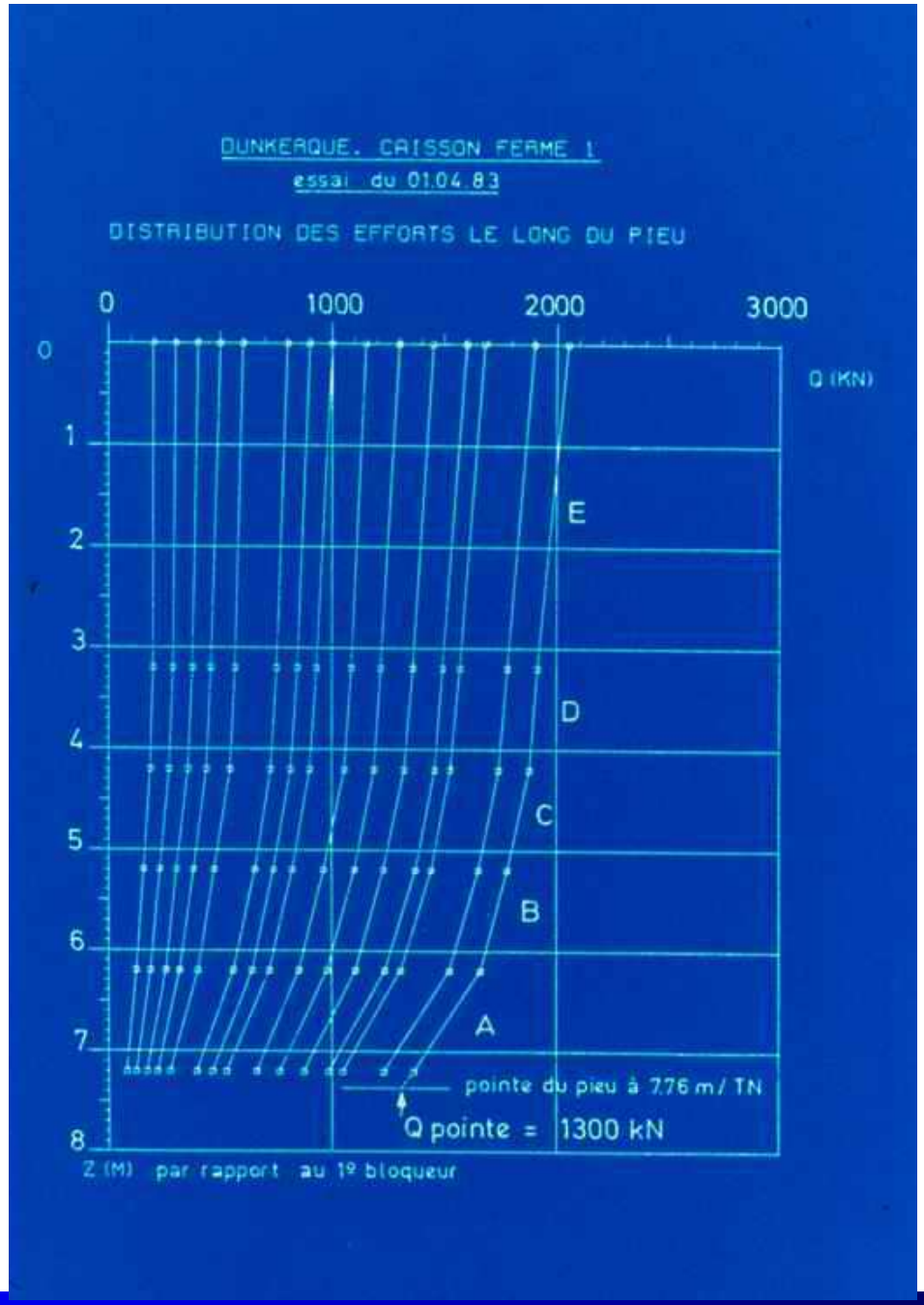
DUNKERQUE. CAISSON FERME 1
essai du 01.04.83
DISTRIBUTION DES EFFORTS LE LONG DU PIEU
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2000
3000
Q (KN)
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2
3
4
5
6
7
8
E
D
C
B
A
pointe du pieu à 7.76 m / TN
Q pointe = 1300 kN
Z (M) par rapport au 1º bloqueur

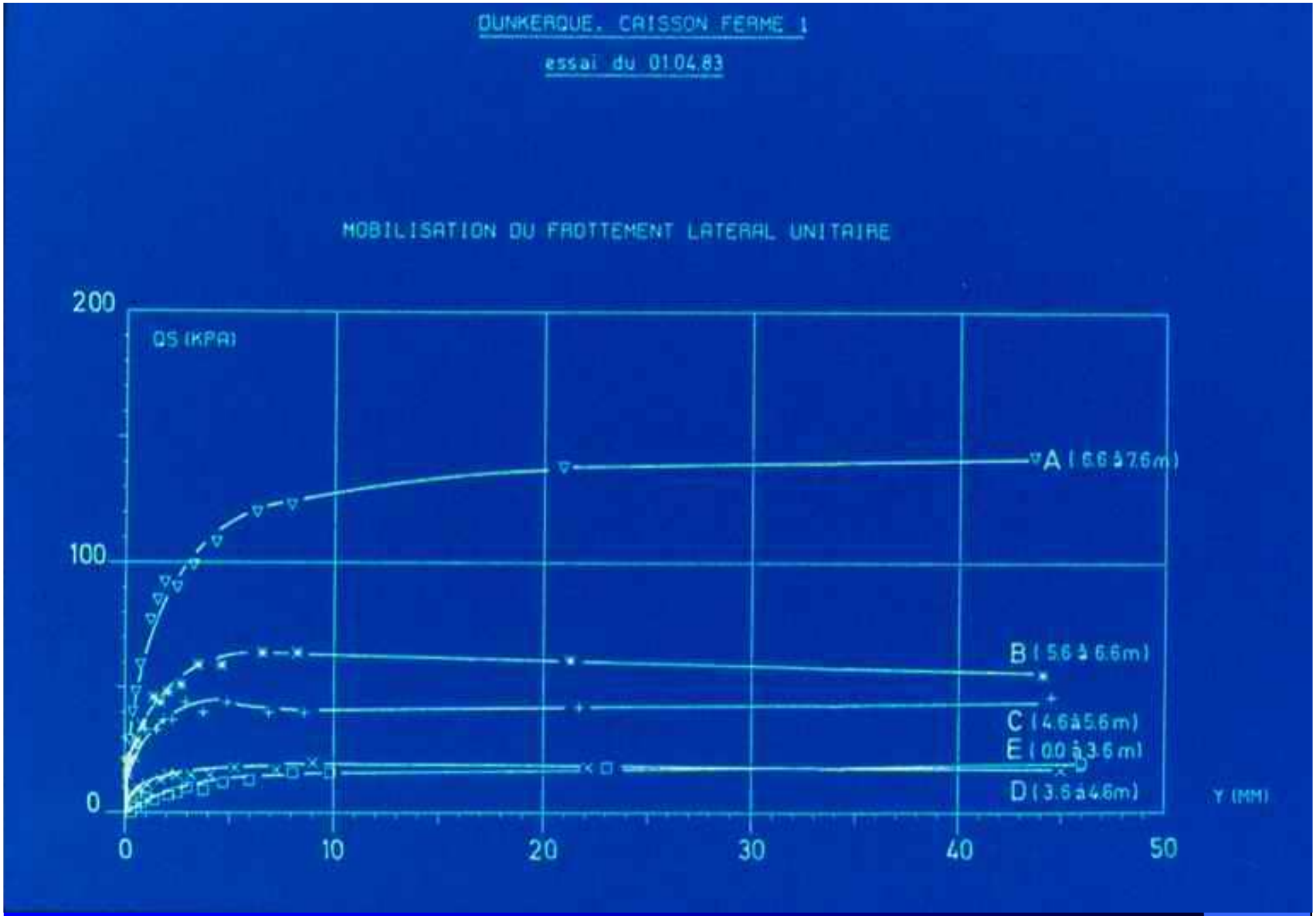

DUNKERQUE. CAISSON FERME 1
essai du 01.04.83
MOBILISATION DU FROTTEMENT LATERAL UNITAIRE
QS (KPA)
200
100
0
0
10
20
30
40
50
Y (MM)
A (6.6 à 7.6 m)
B (5.6 à 6.6 m)
C (4.6 à 5.6 m)
E (0.0 à 3.6 m)
D (3.6 à 4.6 m)

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).

Revised MPM rules for bearing capacity : k_p

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.

Revised MPM rules for bearing capacity : q_s

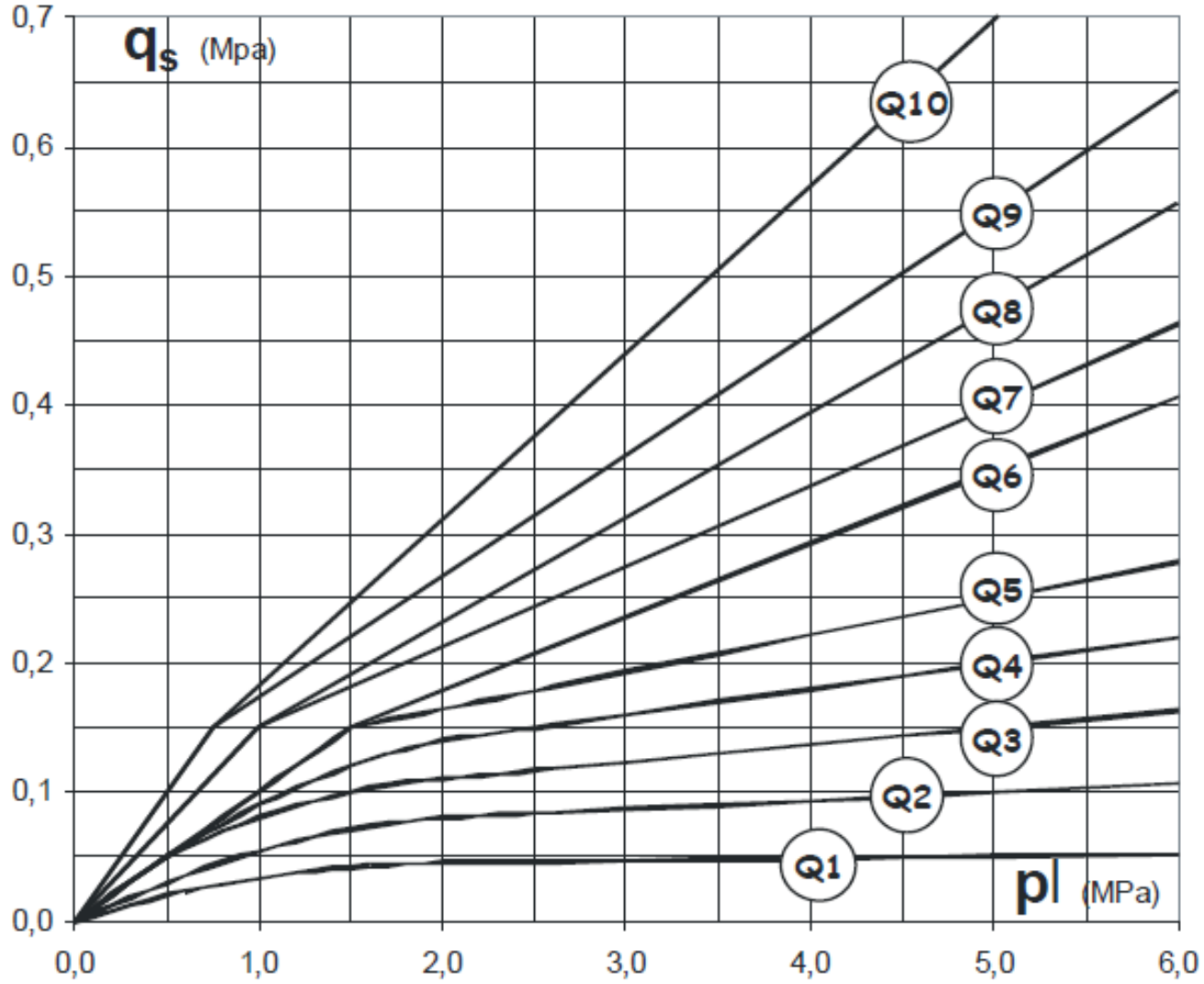


Figure 4. Direct design with MPM data. Chart for skin friction q_s (Bustamante et al., 2009).

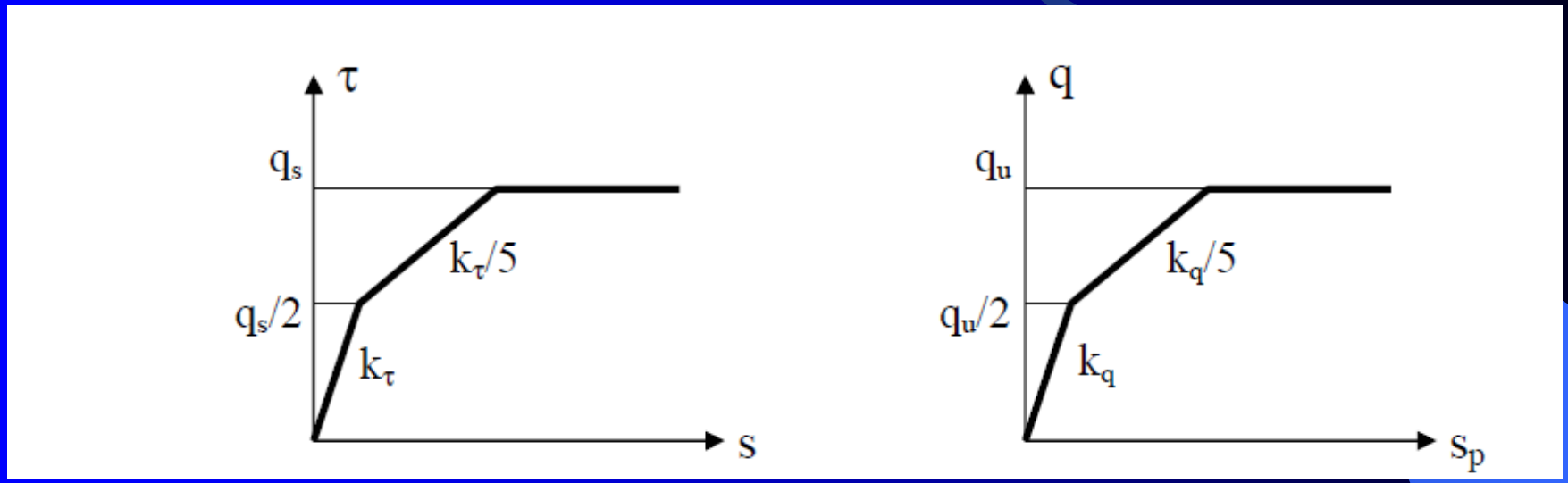
Table 6. Direct design with MPM data. Selecting the Qi line to obtain the limit unit skin friction values q_S (Bustamante et al., 2009).

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.

Settlement of piles

t-z curves from Ménard pressuremeter modulus E_M
(laws of Frank-Zhao, 1982)



for fine grained soils :

$k_\tau = 2.0\ E_M/(2r_o)$ and $k_q = 11.0\ E_M/(2r_o)$

for granular soils

$k_\tau = 0.8\ E_M/(2r_o)$ and $k_q = 4.8\ E_M/(2r_o)$

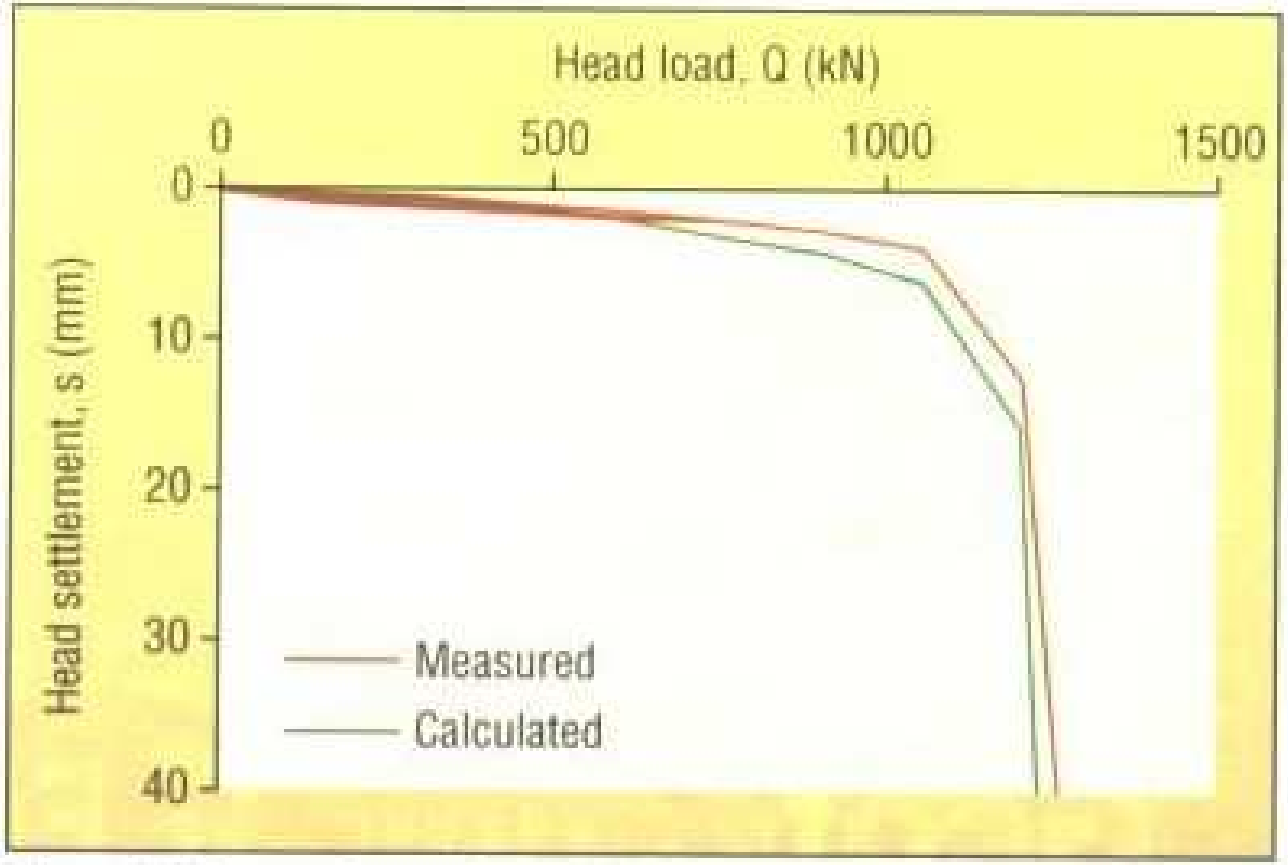


Figure 5. Comparison of measured and calculated load-settlement relationship for the Koekelare pile (Bustamante & Gianeselli 1993).

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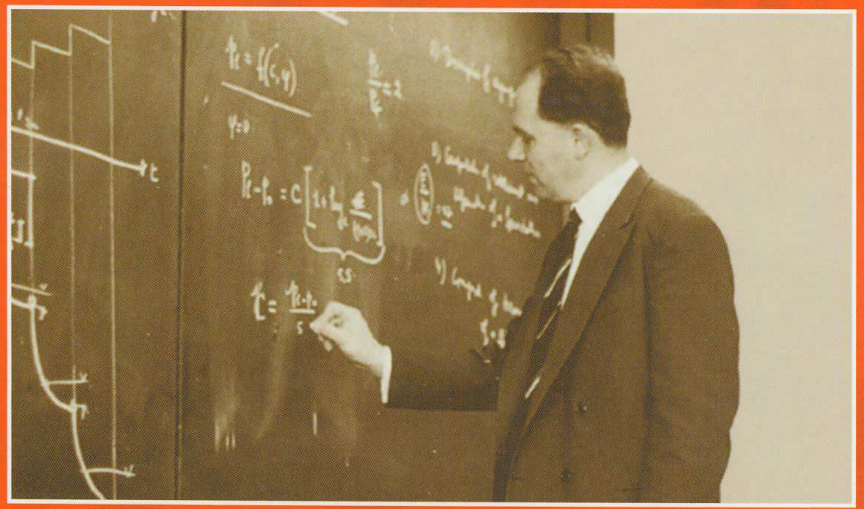

Symposium international
50 ans de pressiomètres

International Symposium
50 years of pressumeters

Sous la direction de / *Edited by*
Michel GAMBIN, Jean-Pierre MAGNAN et Philippe MESTAT

Marne-la-Vallée, 22-24 août 2005 / *22-24 august 2005*

LCPC Laboratoire Central des Ponts et Chaussées

Presses de l'école nationale des Ponts et chaussées

Volume 2

ISP5 - PRESSIO 2005

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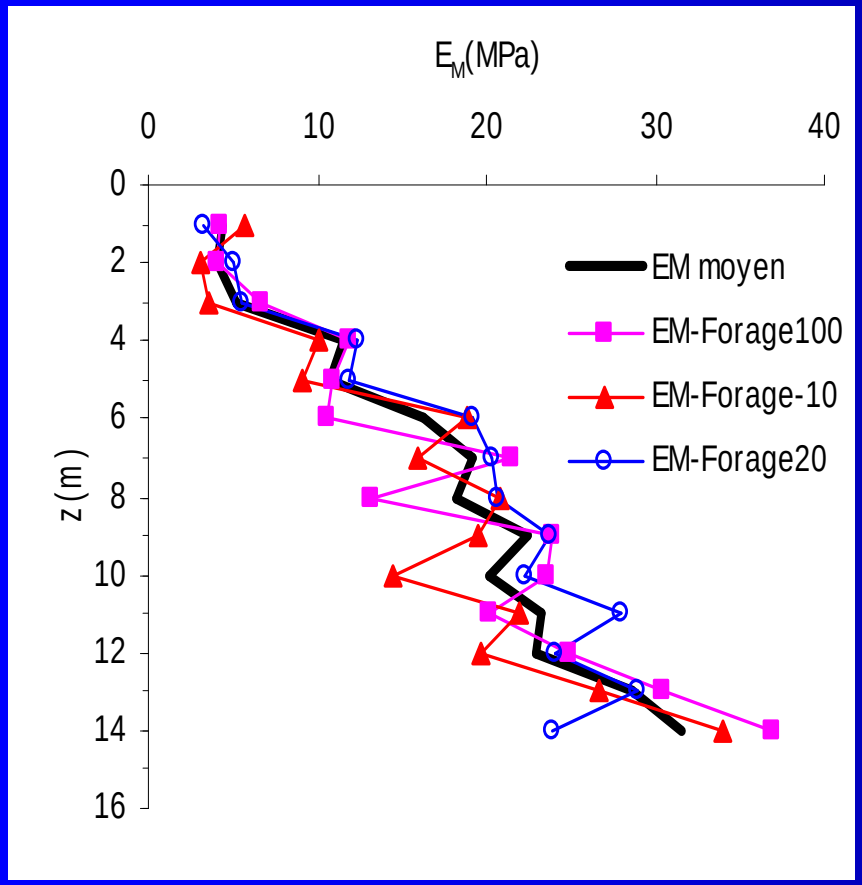
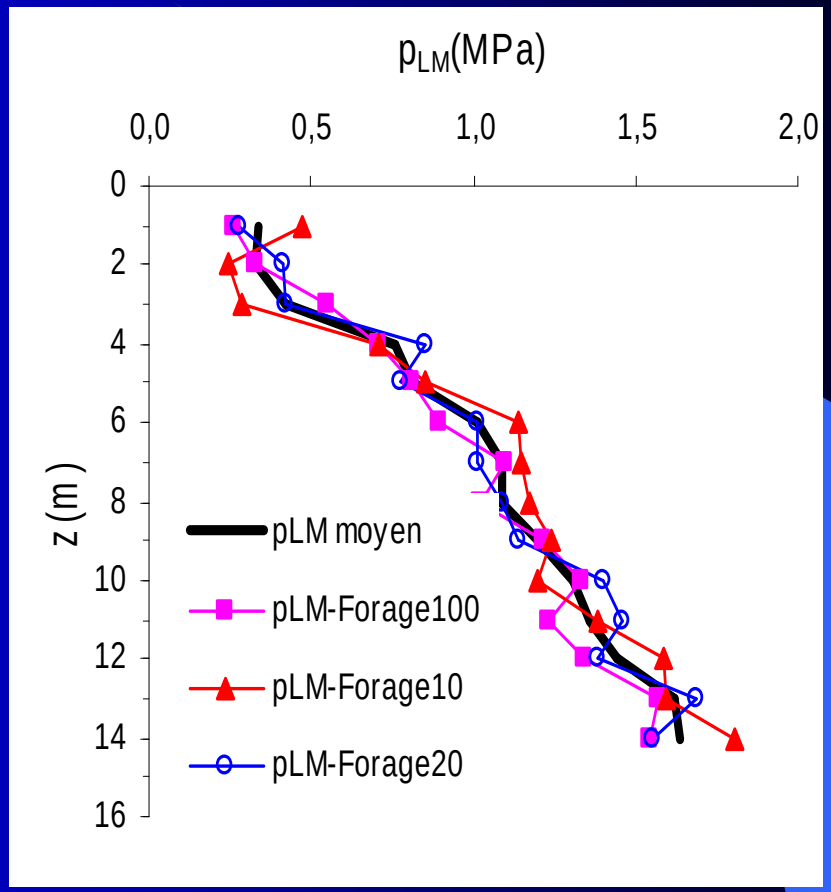
LCPC Laboratoire Central des Ponts et Chaussées

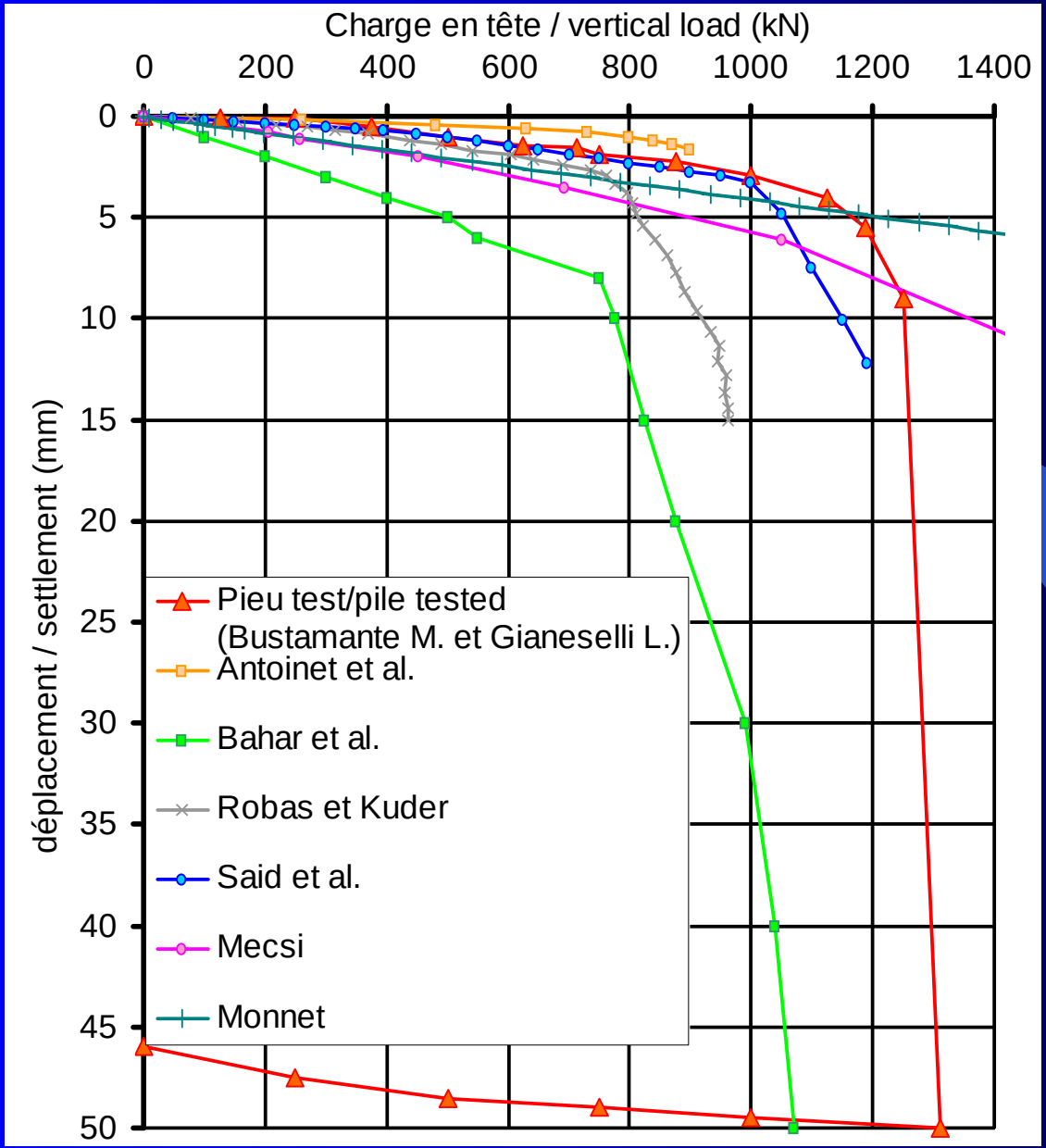
Presses de l'école nationale des Ponts et chaussées

2nd example : CFA pile in silt and clay (Northern France)

B = 0.5 m ; D = 12 m






Charge en tête / vertical load (kN)
0
200
400
600
800
1000
1200
1400
déplacement / settlement (mm)
0
5
10
15
20
25
30
35
40
45
50
Pieu test/pile tested
(Bustamante M. et Gianeselli L.)
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Bahar et al.
Robas et Kuder
Said et al.
Mecsi
Monnet

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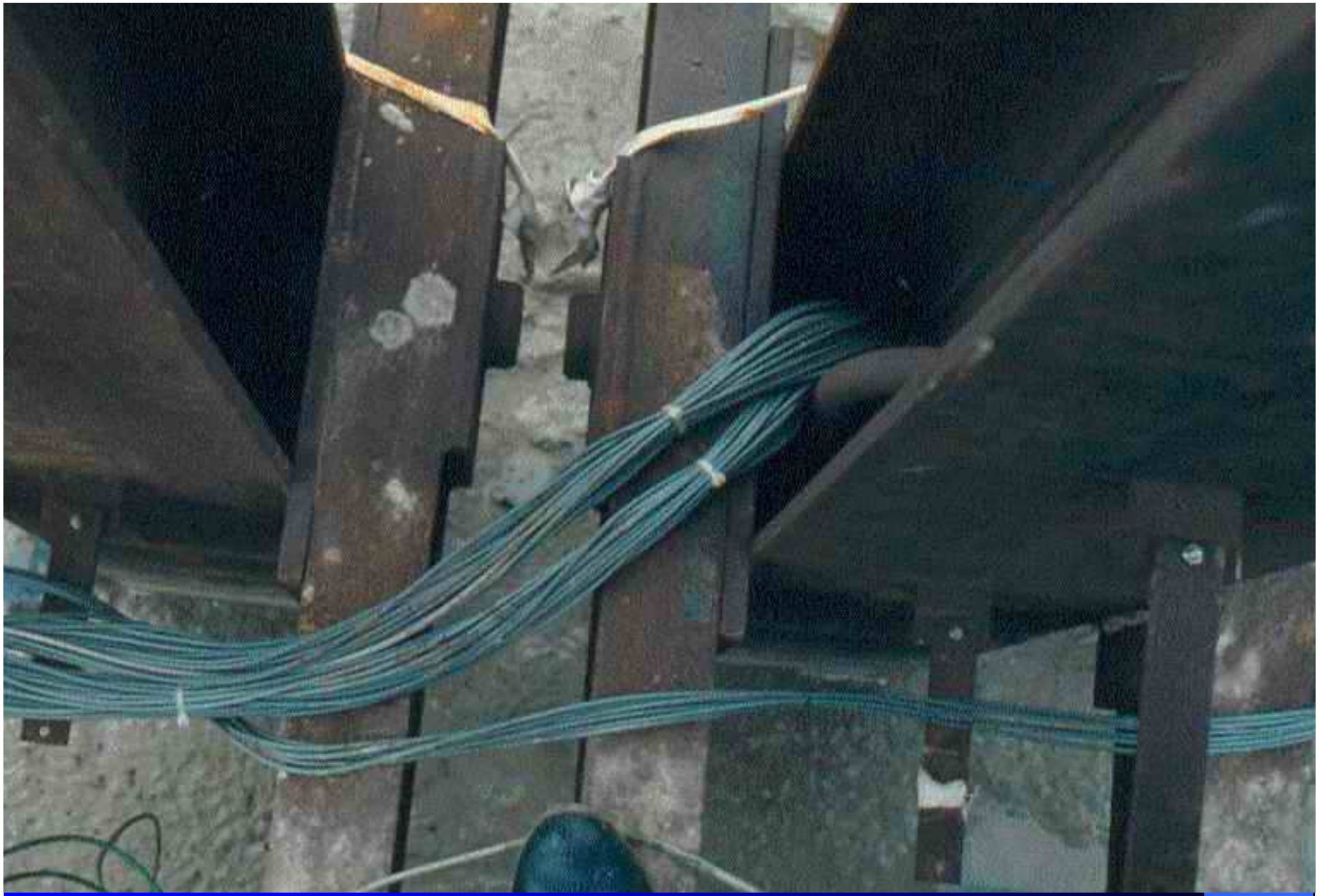
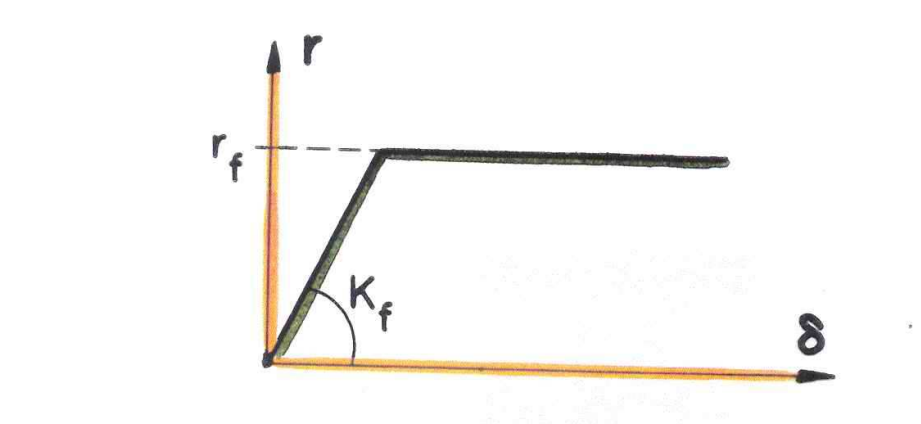
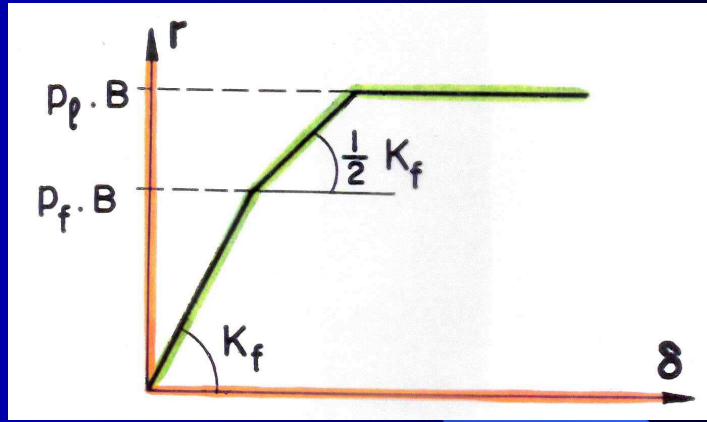
- Conclusions

Transverse load tests at Plancoët (1975-…)



P-y curves





Short duration

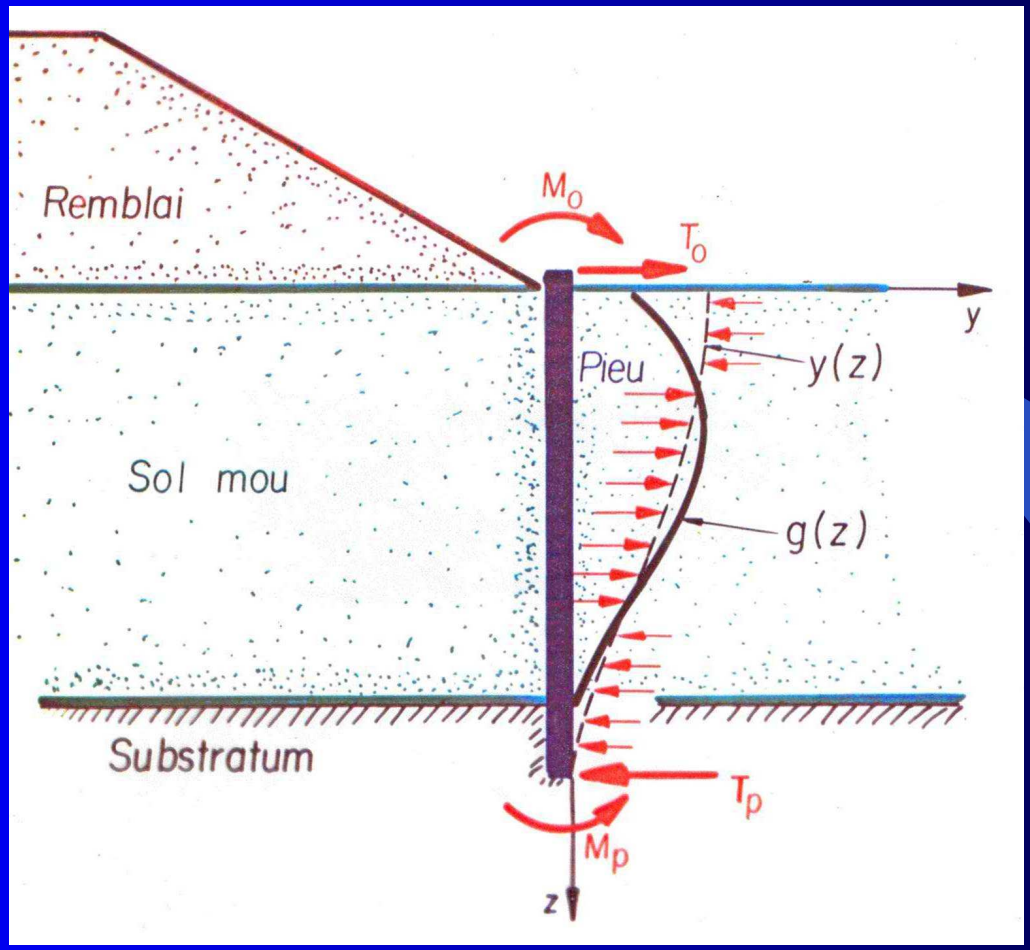
Accidental actions

with $K_f = 2E_{SM}$

$$E_{sM} = E_M \frac{18}{4(2{,}65\ B/B_o)^\alpha\ B_o/B + 3\alpha} \quad \text{or} \quad E_M \frac{18}{4(2{,}65)^\alpha + 3\alpha}$$

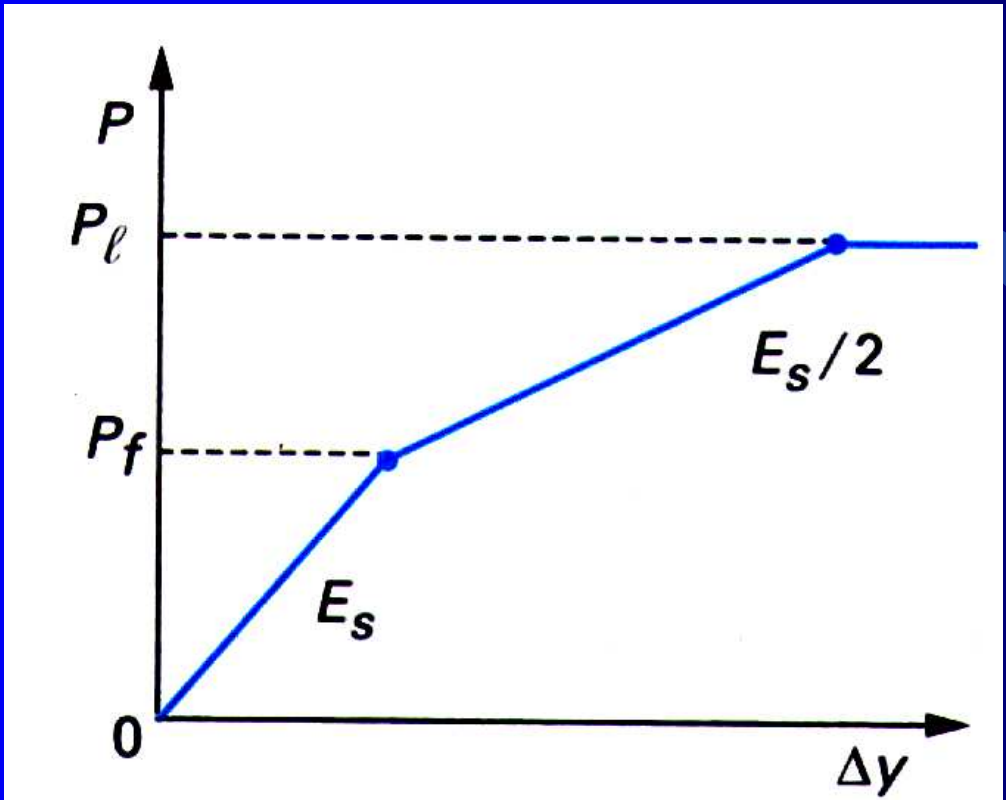
E_{sM} = Ménard reaction modulus

g(z) method

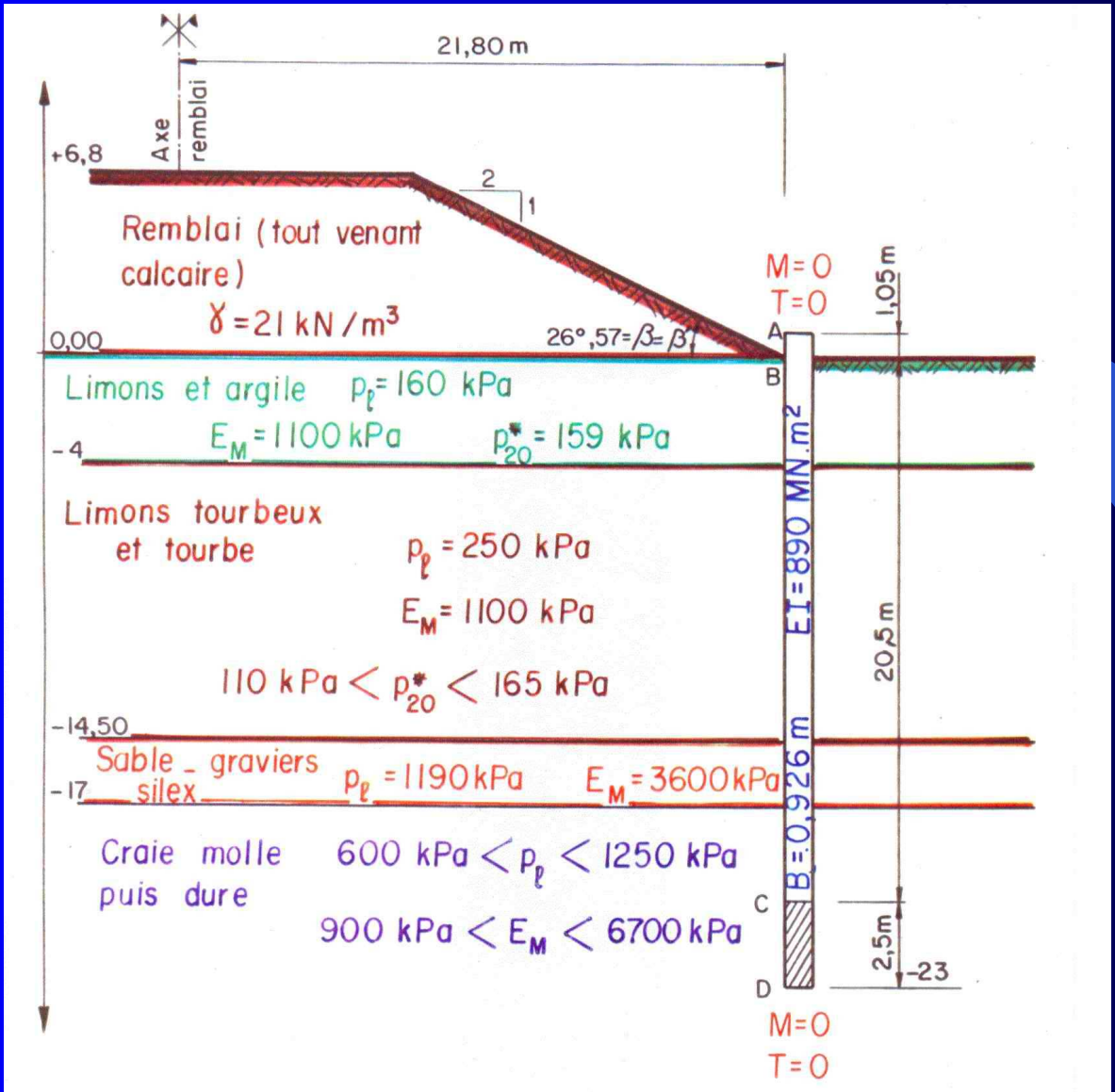


$$EI \frac{d^4y}{dz^4} + P(\mathbf{y} - \mathbf{g}) = 0$$

Reaction curve for long duration loads



Pile of Provins (1974)



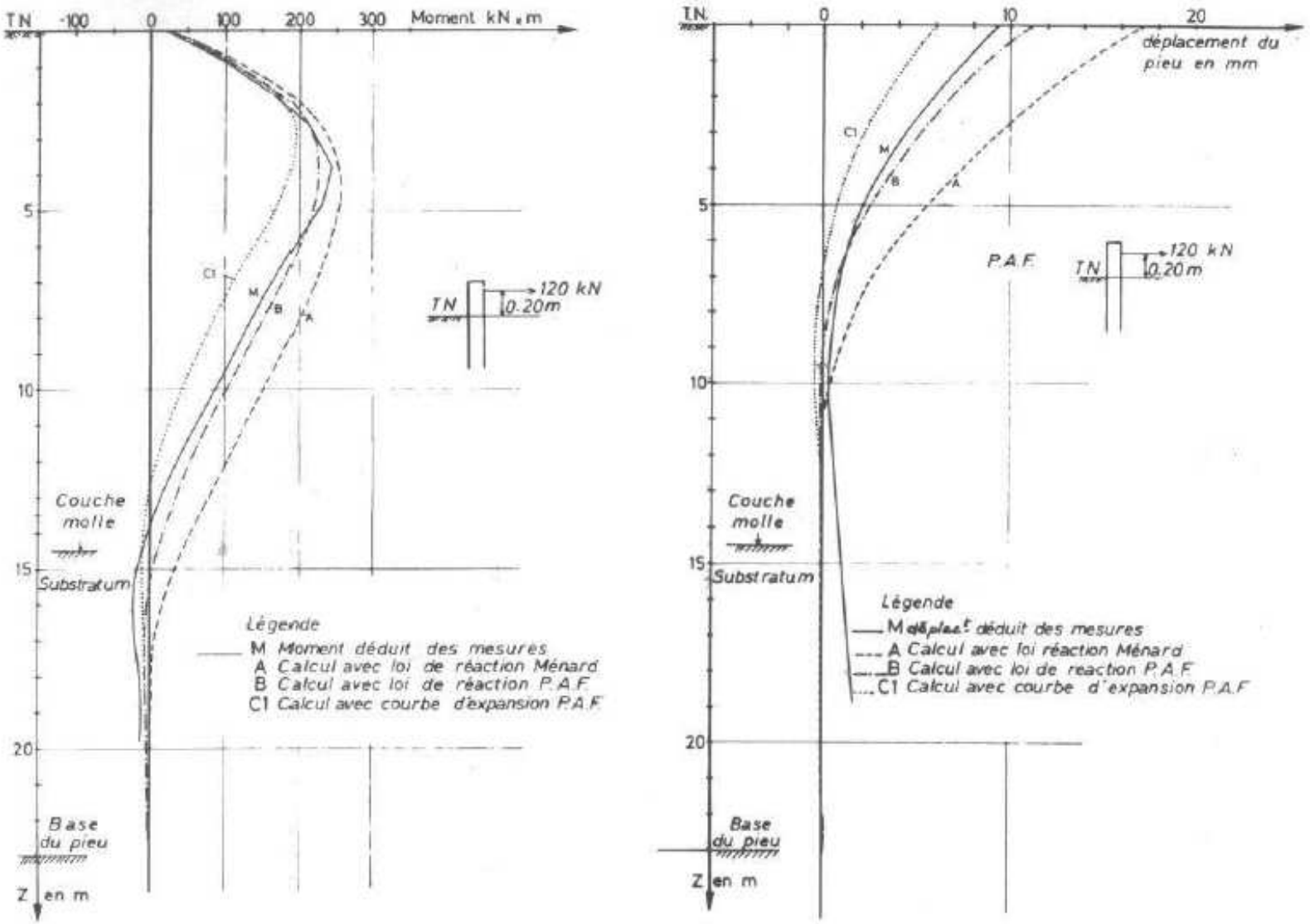


Figure 9. Provins pile. Comparison of measured and calculated bending moments and displacements for head loading (Bigot et al. 1982).

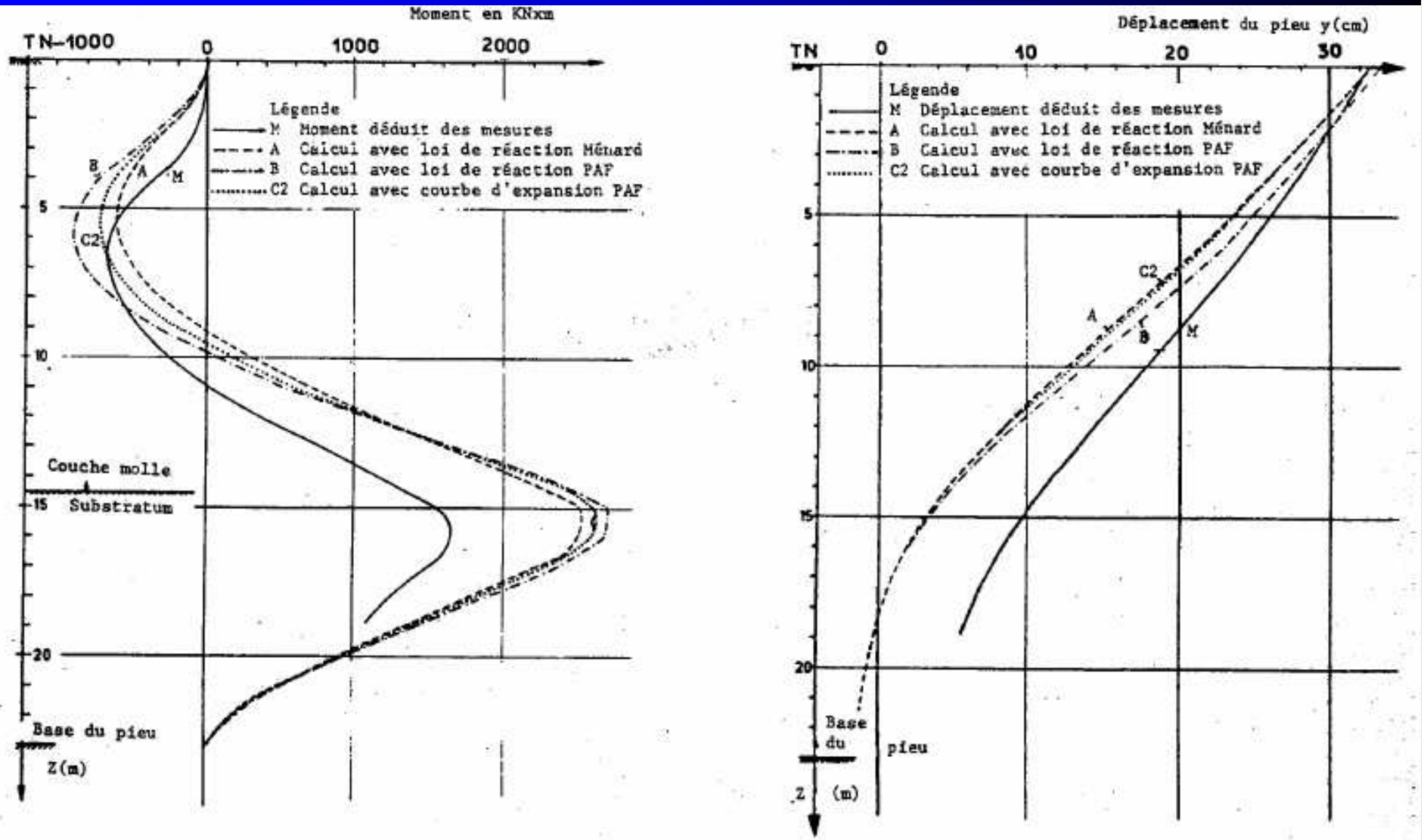


Figure 10. Provins pile. Comparison of measured and calculated bending moments and displacements after 3 months of consolidation under final height of embankment (Bigot et al. 1982).

Contents

- Introduction to design rules. Why use the MPM (and CPT) ?
- Shallow foundations
- Axial capacity of piles
- Transverse capacity of piles
- Conclusions

- The pressuremeter allows the determination of the bearing capacity and of the displacement of foundations
- The base capacity is based on the analogy with the cavity expansion theory (sounder than limit equilibrium approaches)
- The shaft friction of piles is determined essentially in an empirical manner (numerous full scale load test results)
- MPM results yield easily p-y and t-z curves.

Eurocode 7 issues/challenges …

- All design rules compatible with Eurocode 7 should be based on pile load test results
- Important role of displacement of foundations of structures (more than safety with regard to ground failure)
- Are we ready of SLS checks only based on displacements? … and is the structural engineer ready himself ?

Thank you for your attention !