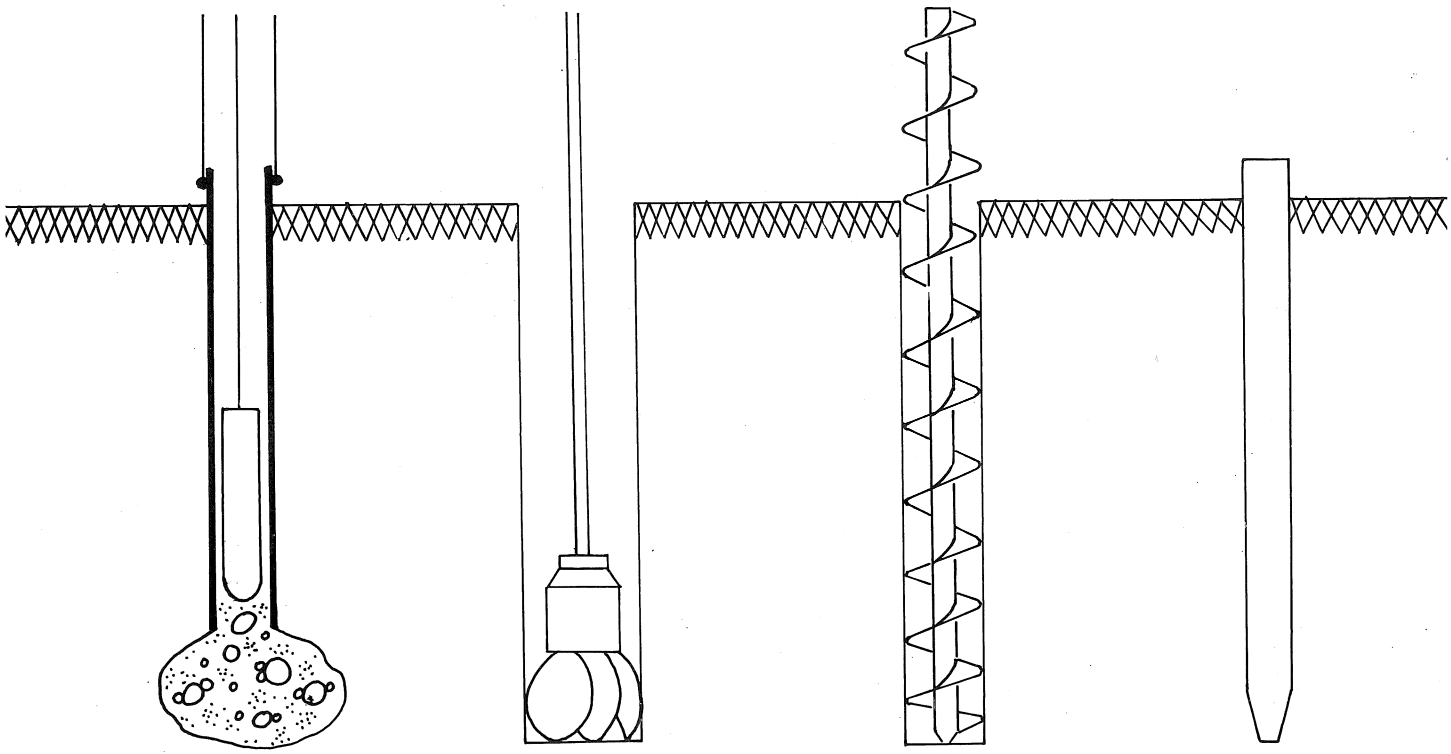
INFLUENCE OF LATE DEVELOPMENT IN FOUNDATION TECHNIQUE ON THE DESIGN OF PILES



BY

A. HOLEYMAN, IR., M.Sc., DR., (MANAGER FRANKI INTERNATIONAL)

AND

M. BOTTIAU, IR., (ENGINEER FRANKI INTERNATIONAL)

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1. INTRODUCTION

When one searches to estimate the bearing capacity of piles, three elements at least have to be taken into account :

- the physical caracteristics of the pile,
- the nature and resistance of the encountered soil,
- the method of execution.

In this note, we will emphasize this last element. The different piles will be classified according to their execution and installation procedure. We will then try to establish the bearing ratio between them in the light of the in-situ tests currently used to determine the bearing capacity of piles.

2. GROUPS OF PILES

We will distinguish three groups of piles, according to their execution procedure :

2.1. GROUP I : Piles introduced into place by displacement of the in-situ soil.

This definition implies that two conditions must be satisfied :

a- The soil volume displacement is equal to the pile volume
b- The soil displacement vector at the pile interface can only be directed outwards the pile and downwards.

These two conditions induce a maximal compression stress state near by the toe of the pile; this stress state implies a high bearing capacity.

Examples of piles pertaining to this group I are :

* Cylindrical or prismatic prefabricated jacked on driven piles, which include

  - Miga piles and
  - Prefabricated driven piles

* Piles cast-in-situ with the help of a recoverable sleeve

  which include,
  - Cast-in-situ piles without formation of an enlarged base and
  - Cast-in-situ piles like the Franki-pile with its enlarged base.

  This last one will be chosen as example of this first group.

Execution of the Franki-pile

The Franki-pile is a driven cast-in-situ concrete pile. Its enlarged base and rough shaft allow it to make maximum use of the soil bearing capacity.

The various phases of the installation of the Franki-pile are detailed in fig. 2.1

2.2. GROUP II : Piles which are constructed by excavation

This group mainly includes the bored piles.

* Bored piles

  The compression stress state is here no longer maximum in the soil surrounding the pile. The possible stress release at the bore wall resulting from the excavation process can reduce the bearing capacity of the bored piles in comparison to the driven piles.

  We can distinguish e.g.:
  - large diameter bored piles
  - small diameter drilled piles (continuous flight auger-piles)

Execution of bored piles

Large diameter bored piles

They are bored cast-in-situ piles. Several drilling methods can be used according to the ground conditions :

- recoverable steel casing,
- permanent thin wall lining,
- bentonic mud.

The installation procedures are described in fig. 2.3

Small diameter drilled piles

These small diameter piles are executed by extracting soil with a continuous hollow flight auger, plugged at the bottom during drilling.

The execution scheme is shown in fig. 2.5

2.3. GROUP III : Piles introduced into place by an intermediate process (no pure displacement no pure excavation)

Examples of these piles are :
- prefabricated piles with an enlarged base
- bored and grouted piles :
  - injected micro-piles
  - large diameter piles for foundations of pylons for high-voltage lines.

As these two last types of piles are actually new techniques, we will detail a little more their execution.

Execution of micro-piles

These piles are executed by soil extraction using varied tools chosen in functon of the encountered soil conditions. Their diameter will not exceed 250 mm.

We will see further that two types of injected micro-piles have to be differentiated according to the injection procedure.

The execution procedure is shown in fig. 2.7

Execution of large diameter piles for foundations of pylons for high-voltage lines

This concerns bored piles of large diameter where the common filling with concrete is replaced by a lost steel tube provided on the outside with 6 injection tubes ("tubes à manchettes"). This device allows to inject in several phases the annular void around the tube to ensure a perfect contact with the soil.

This system is introduced into place without vibrations and allows a good penetration of stiff layers, impossible with other execution procedures.

The figure 2.9 gives an example of this new type of piles which has been developped in Belgium by Franki.

3. IN-SITU TESTS CONSIDERED FOR PILE DESIGN

We will base our comparison on the following methods which are currently used and have known some recent development and adjustment in the choice of the pile design parameters. In our review of the literature, we have examined the methods based on the cone penetration test (CPT) and those based on the pressuremeter test (PMT).

3.1. CONE PENETRATION TEST

The well known cone penetration test yields the following basic results :

- point resistance q_c [MPa]
- local skin friction f_s [MPa]
- total skin friction L [MN or kN]

We will examine three different methods and give the pile bearing coefficients resulting from the latest developments :

- method of professor E. De Beer, (1971-72/1984)
- LPC - cone method (Bustamante - Gianeselli), (1983)
- French proposition for the establishement of the "Eurocode 7", (1985)

3.2. PRESSUREMETER TEST

In the methods based on this test, one basically expresses the "limit" (yield) stress at the base q_p and the unit limit skin friction q_s from the measured limit pressure p_ℓ.

We will examine two different methods and also give the coefficients resulting from the latest developments :

- LPC - method (Bustamante - Gianeselli, 1983)
- French proposition for the establishement of the "Eurocode 7", (1985)

All methods based on either of these two in-situ tests require the introduction of more or less empiric coefficients depending on :

- the nature and resistance of the encountered soil
- the execution procedure

<u>Preliminary remarks</u>

* Only three categories of soils will be taken into account :

  - clay and silt, (cohesive soils)
  - sand and gravel, (granular soils)
  - chalk, (soft rock).

  excluding dry and lateritic soils, marl, limestone.. .

It seems to be the rule among the considered authors (almost all of them from the French School) to use one single category for both clayey and silty soils. As this is a review of existing data, this single category will also be found in this paper. However, we think that a distinction should be to be made between clay and silt because they do exhibit a different behaviour under the process of excavation and compaction.

* It is also worthwile to note that the choice of a method for the estimation of the bearing capacity at the base implies the use of the same method to estimate the lateral resistance.

4. ASSESSMENT OF THE BEARING CAPACITY AND SKIN FRICTION OF GROUP I PILES (DISPLACEMENT PILES)

4.1. Cone penetration test

4.1.1. De Beer's Method

* The unit rupture load at the toe of the pile can be written as :

$$q_b = \varepsilon \, d_g$$

where d_g = unit rupture load following De Beer (1971-1972)

$$d_g\ (z) = f_{DB}\ (z,\ q_c(z),\ \frac{d_b}{d_c})$$

with
- z : depth
- d_b : diameter of the pile base
- d_c : diameter of the penetration cone.

De Beer's method is a method which scales the q_c diagram according to the size of the failure mechanism of a given base relative to the failure mechanism of the cone.

ε is a coefficient ≤ 1 :
- $= 1$ granular soils
- < 1 clayey soils

for example, in the Boom Clay (tertiary overconsolidated and fissured)

$$\varepsilon = 1 - 0{,}01\ (\frac{d_b}{d_c} - 1)$$

* The skin friction can be calculated by :

$$q_s = \alpha_{sn} \ \alpha_{sd} \cdot \frac{d_b}{d_c} \cdot L$$

where L is the total friction (MN or kN) acting along the rods up to the depth of penetration. This value is obtained by difference between the total penetration force and the cone resistance. Due to the wear of the skin friction at one particular level with the travel of the rods, L is systematically lower than the integral of the local skin friction as measured with a friction sleeve.

α_{sn} depends on the difference between the nature of the lateral surface of the pile and the penetrometer.

α_{sd} is the scale factor representing the effect of the difference between the respective diameters of the pile and the penetrometer d_b and d_c

The ($\alpha_{sn} \cdot \alpha_{sd}$) - values are not known for all cases but we can give the following values:

Soil type	$\alpha_{sn} \cdot \alpha_{sd}$
Granular soils	1.6
Stiff and fissured clay	1.15

4.1.2. LPC Cone Method (Bustamante - Gianeselli)

* The unit point resistance q_p is written here

$$q_p = K_c^{pc} \ q_c$$

where q_c is the cone resistance at the base level

K_c^{pc} is a point bearing factor which depends on soil type and pile installation procedure.

K_c^{pc} is given for the here considered driven piles in table 1.1. :

Table 1.1.

type of soil	point bearing factor K_c^{pc}
clay/silt	0.600
sand/gravel	0.375
chalk	0.400

* The unit friction resistance of the pile is obtained from the appropriate curve, f_s versus q_c following
  - soil type
  - way of execution.

The appropriate curve in fig. 4.1. is selected according to the soil type and q_c range using table 1.2.

Table 1.2

Soil type	q_c (MPa)	Curve #
clay and silt	< 0.7	I 1
	> 1.2	I 2
sand and gravel	< 3.5	I 3
	3.5 < < 7.5	I 4
	> 7.5	I 5
chalk	< 3.0	I 6
	> 3.0	I 7

Note : the investigation of this method brings us to the following remark : the mentioned curves are actually given according to
- the soil type
- the q_c value.

This means that for a given pile type, the unit mantle resistance jumps when one switches from one range of cone resistance to an other range.

example : in sand/gravel, when q_c moves over the 3.5 MPa pivot value, the unit friction resistance jumps from curve 3 to curve 4, thus from 0.055 MPa to 0.070 MPa.

This jump is, of course, not realistic. So, it could be worthwhile to note that these curves would preferably be given for a type of pile and valid over the whole range of q_c.

4.1.3. French proposition

* The total limit point stress at the pile base q_b is given by :

$$q_p = q_{ca} K_c^7$$

with q_{ca}: equivalent point resistance calculated as explained in the French proposition for Eurocode 7. It is basically a safe average of the q_c diagram over 1.5 d_b above and below the base level.

K_c^7 : bearing capacity factor depending on :
- soil type
- pile type

The K_c^7 values are given in table 1.3

Table 1.3

Soil type	Bearing capacity factor K_c^7
clay/silt	0.50 to 0.60
sand/gravel	0.40 to 0.50
chalk	0.40

* The unit skin friction is obtained by :

$$q_s = \frac{q_c}{\alpha}$$

where α is a coefficient which depends on :
- the soil type
- the pile installation process.

The α - values are given in table 1.4

Table 1.4

Soil type	α - value
clay - silt	60 - 100
sand - gravel	100 - 175
chalk	70 - 110

The higher values of α are associated with denser or more consistent soils.

4.2. Pressuremeter test

4.2.1. The LPC - Setra Method

* The total limit point pressure q_p is obtained from the equivalent limit pressure $p_{\ell e}$ and the geostatic horizontal and vertical stresses, respectively p_o and q_o by means of the classic formula :

$$q_p = k_p\,(p_{\ell e} - p_o) + q_o$$

where :

$p_{\ell e}$ is the equivalent limit pressure deduced from the measured limit pressure p_ℓ values,

k_p is a bearing capacity factor depending on :

- the soil type
- the type of execution

kp - values are given in table 1.5

Table 1.5

Soil type	k_p- value
clay - silt	1.8
sand - gravel	3.2 to 4.2
chalk	2.6

* The unit limit friction resistance q_s is known from the diagram q_s vs p_ℓ in figure 4.2.

The choice of the appropriate curve is made by use of table 1.6, according to the soil type.

Table 1.6

Soil type	Curve to be used
clay - silt	I 1
sand	I 2
gravel	I 2
chalk	I 3

4.2.2. Eurocode 7

The method is similar to the LPC-Setra method, the kp - values being given in table 1.7 and the curves for the skin friction by fig. 4.3 (the appropriate curve is chosen by table 1.6, already given above).

Table 1.7

Soil type	k_p
clay - silt	1.8 to 2.2
sand - gravel	3.2 to 4.2
chalk	2.4 to 2.8

5. ASSESSMENT OF THE BEARING CAPACITY AND SKIN FRICTION OF GROUP II PILES (BOREd PILES) AND COMPARISON WITH GROUP I PILES (DISPLACEMENT PILES)

5.1. Cone penetration test

5.1.1. De Beer's Method

* Because of the possible expansion of the soil around the bored pile compared to the compression of the soil influenced by the failure mechanisms of the cone during penetration and of the displacement pile, we have to apply a reduction coefficient to the values of the unit rupture load calculated as explained in the previous section.

  We can thus write here :

  $$q_{cb} = \beta \cdot q_b$$

  where q_b represents the point resistance of the driven pile with the same geometrical caracteristics

  β in a reduction coefficient depending on the stress state.

  The β - values depend on
  - the soil type
  - the depth of embedment
  - the allowance for soil decompression during the installation of the considered pile.

  one can adopt with De Beer's method :

  β = 0.8 for Boom clay
  β < 0.8 for granular/ slightly cohesive soils

5.1.2. LPC - Cone Method

* Table 2.1 gives the K_c^{Pc}- values for the bored piles in the application of formula $q_p = K_c\ q_c$.

  It also gives the values of ζ defined as the ratio of the ultimate bearing capacity at the base of a bored pile to the one of a driven pile, both piles being otherwise identical.

  We will thus write : $\zeta = \dfrac{(K_c^{pc})B}{(K_c^{Pc})D}$ (B : bored - D = driven)

Table 2.1.

Soil	K_c^{Pc}	$\zeta = \dfrac{(K_c^{Pc})B}{(K_c^{Pc})D}$
Clay - Silt	0.375	0.625
Sand - Gravel	0.150	0.400
Chalk	0.200	0.500

* Table 2.2. gives the curve in fig. 4.1. to be used entering both soil type and q_c to estimate the friction resistance.

Table 2.2.

Soil type	q_c (MPa)	curve #
clay ans silt	< 0.7	II 1
	> 1.2	II 2
sand and gravel	< 3.5	II 3
	> 5.0	II 4
	> 7.5	II 5
chalk	< 3.0	II 6
	> 4.5	II 7

5.1.3. French proposition

* The K_c^7 values to introduce in the previously defined equation $q_p = K_c \cdot q_{ca}$ are here given by table 2.3. This table also includes the $\zeta = \dfrac{(K_c^7)\ B}{(K_c^7)\ D}$ - values

Table 2.3.

Soil type	k_c	$\zeta = \frac{(k_c^7)B}{(k_c^7)D}$
clay-silt	0.35 to 0.45	0.58 to 0.9
sand - gravel	0.15 to 0.25	0.3 to 0. 3
chalk	0.20 to 0.30	0.5 to 0.75

* For the calculation of the friction resistance q_s, the α-value of formula $q_s = \frac{q_c}{\alpha}$ can be considered to be the same as those given in 1.1.3. for the driven piles (ζ = 1), except for sand and gravel :

  Loose sand and gravel ζ = 0,9
  Medium compact sand and gravel ζ = 1
  Compact sand an gravel ζ = 0,78.

5.2. Pressuremeter methods

5.2.1. The LPC - SETRA Method

The values of the bearing capacity factor K_p^{pc} are given in table 2.4.

Table 2.4.

Soil type	k_p^{pc}	$\zeta = \frac{(k_p^{pc})B}{(k_p^{pc})D}$
Clay/silt	1.2	0.67
Sand/gravel	1.1	0.26 to 0.34
Chalk	1.8	0.70

* Table 2.5 gives the curves to use for the assessment of the unit lateral resistance q_s with regard to the soil type. The curves themselves are given in fig. 5.1.

Table 2.5.

Soil type	curve to use (*)
Clay/silt	II 1^+
Sand	II 2
Gravel	II 3
Chalk	II 3^+

* The (+)- mark indicates that larger values are probable but they need to be confirmed by full-scale tests.

5.2.2. French propostion to Eurocode 7

* The k_p^7 values are given by table 2.6

Table 2.6.

Soil type	k_p^7	$\zeta = \frac{(k_p^7)B}{(k_p^7)D}$
Clay - silt	1.2 to 1.4	0.55 to 0.78
Sand - gravel	1.0 to 1.2	0.24 to 0.38
Chalk	1.8	0.64 to 0.75

* The curves to use for the assessment of the lateral resistance are given in fig. 5.2.
The appropriate curve is chosen using table 2.5

5.3. Comparative summary

The following tables 2.7 and 2.8 summarize the values of the bearing ratios between displacement and bored piles ratios, respectively for the base resistance and for the mantle resistance, deduced from the reviewed literature.

Table 2.7. Summary table of $\zeta = \frac{(q_p)^B}{(q_p)^D}$ values.

Method / Soil type	CPT TEST			PMT TEST	
	De Beer	LPC	French prop.	LPC	French prop.
Clay/Silt	0.8 (tertiary clay)	0.625	0.58 (0.73)0.90(**)	0.7	0.55 (0.65)0.78
Sand/gravel	0.59 - 0.72 (*)	0.400	0.30 (0.44)0.63	0.3	0.24 (0.29)0.38
Chalk	N.A.	0.500	0.50 (0.625)0.75	0.7	0.64 (0.69)0.75

(*) The values given are relative to a global coefficient (Mantle + predominant base resistance)
They result from full-scale tests performed in Belgium.

(**) The given values result from the ratio between the values given in 5.2.2 and those given in 4.2.2. We give in the order : minimum ratio (ratio between the means of both series) maximum ratio.

Table 2.8. Summary table of $= \frac{(q_s)_B}{(q_s)^D}$ values.

Method / Soil type	CPT TEST				PMT TEST			
	De Beer	LPC	French prop. (*)		LPC		French prop.	
					pℓ [MPa]		pℓ [MPa]	
Clay/silt	N.A.	1	1		< 0.35	1	> 0.6	+ 0.42 to
					> 2	0.5		0.58+
Sand/gravel	N.A.	1	q_c < 12 MPa	1	sand < 0.55	1		0.70 to 0.76
					> 2.2	0.67		
			q_c > 12 MPa	0.8			> 0.8	
					gravel	1		1
Chalk	N.A.	1	1		< 0.75	1	> 1.15	0.81 to
					> 2.9	0.75		0.83

(*) For a special type of "silt and loose sand", with q_c < 5MPa, ξ = 0.9

6. ASSESMENT OF THE BEARING CAPACITY AND SKIN FRICTION OF GROUP III PILES (PILES INTRODUCED INTO PLACE BY AN INTERMEDIATE PROCEDURE AND COMPARISON WITH GROUP I PILES (DISPLACEMENT PILES)

We shall distinguish

* Category A

This category mainly concerns the injected micropiles (drilled piles with a diameter ≤ 250 mm). The Injection is realised Globally and in an Unique phase (GUI for short)

* Category B

This category includes :

- drilled piles < 250 mm in diameter for which the Injection is Repetitive and Selective and repetitive injection (micropiles of RSI type),
- large diameter grouted bored piles also executed by a selective and repetitive injection (RSI).

6.1. Cone penetration test

6.1.1. LPC - Cone Method

* The unit lateral resistance q_s can be determined as previously defined by use of table 3.1. entering :
  - soil type
  - q_c - value.

Fig. 4.1 and 6.1 give the appropriate curves

Table 3.1.

Soil type	q_c (MPa)	Cat. III A	Cat. III B
Clay/silt	< 0.7	III A 1	III B 1
	> 1.2	III A 2	-
	> 2	-	III B 3
Sand/gravel	< 3.5	III A 3	-
	> 3.5	III A 4	-
	> 5.0	-	III B 3 (or more)
	> 7.5	III A 5	
Chalk	< 3.0	III A 6	-
	> 4.5	III A 7	III B 2 (or more)

6.1.2. French proposition to Eurocode 7

* The ultimate bearing capacity and the unit skin friction can be calculated as previously explained :
  the micropiles of category A are to be linked with bored piles wherease the piles of category B are to be linked with displacement piles.

6.2. Pressuremeter test

6.2.1. LPC - Method

* The unit point bearing load is calculated like above, linking the category A piles with bored piles and category B piles with driven piles.

* To know the unit lateral resistance, one have to use table 3.2. giving the curve in fig. 6.2 to use with regard to the soil type.

Table 3.2.

Soil type	III-A	III-B
Clay/silt	III-A 1	III-B 1
Sand	III-A 2	III-B 1
Gravel	III-A 2	III-B 2
Chalk	III-A 3	III-B 2

A larger limit skin friction can be accounted for but it can only be adopted on the basis of results of loading tests performed on identical piles.

6.2.2. French proposition to Eurocode 7

* The determination of both bearing capacity and skin friction is made following the same developments than in 6.2.1. However, the values of the k_p coefficient and the curves (given in fig. 6.3) to be used are different.

6.2.3. Bustamante (Nov/Dec. 1985)

* This method is also based on the pressuremeter test and its application is similar to the methods developped above.

* The unit point resistance is determined by using the bearing capacity factor k_p given in table 3.3. valid for both categories A and B.

Table 3.3.

Soil type	k_p value
Clay	1.6
Sand/gravel	1.2
Clay/Marl	1.8

* The unit limit resistance is evaluated from the curves, given in fig. 6.4, following the soil type.

The 1- mark is relative to the type III A
The 2- mark is relative to the type III B

6.3. Comparative summary

The values of the bearing ratios deduced from the analysis of the various methods are summarized on the following tables :

- table 3.4 relates to the base bearing ratio of a bored injected pile (group III) to a displacement pile (group I),
- table 3.5 and 3.6 relate to the mantle bearing ratio of bored injected piles (group III) of, respectively category A (GUI) and category B (RSI) to a displacement pile group I),
- table 3.7 allows to establish the mantle resistance enhancement when switching from category A to category B.

Table 3.4.

Method / Soil type		CPT test: LPC	CPT test: French prop.	PMT test: LPC	PMT test: French prop.	PMT test: Bustamante/Doix
Clay and Silt	III-A	0.625	(*) 0.58(0.73)0.90	0.7	0.55(0.65)0.78	(**) 0.89
	III-B	1	(*) 1	1	1	
Sand and Gravel	III-A	0.40	0.3(044)0.63	0.26(0.3)0.34	0.24(0.29)0.38	0.285 to 0.375
	III-B	1	1	1	1	
Chalk	III-A	0.50	0.5(0.625)0.75	0.7	0.64(0.69)0.75	0.69
	III-B	1	1	1	1	

Summary table of $\zeta = \frac{q_p^{Bi}}{q_p^{D}}$ - values for bored-injected piles type III-A/III-B

Remarks and comments :

(*) The values of category III-A are the same as those of bored piles and the values of category III-B the same as those of driven piles. This results from the classification established by both LPC and French proposition methods.

(**) The values given by Bustamante/Doix (Nov-Dec. 1985) make no distinction between III-A and III-B categories. One sees that the suggested values are in agreement with the means of the values given by the other methods for both III-A and III-B categories.

(***) The given values show that the bearing capacity at the base obtained with bored and injected piles will at most reach the one obtained with driven piles. In fact, the injection is realised along the shaft of the pile. The performances are thus enhanced for what concerns the friction resistance but there is no perceivable improvement for the point resistance.

Table 3.5

Method / Soil type	CPT test LPC	CPT test French prop.	PMT test LPC p_ℓ (MPa)	PMT test LPC	PMT test French prop.	PMT test Bustamante/Doix p_ℓ (MPa)	PMT test Bustamante/Doix
Clay and Silt	1	1		1^+	1^+	> 0.6	1.67 to 2
Sand and Gravel	1	q_c < 12 MPa 1 q_c > 12 MPa 0.8	sand gravel	1 1+	1 1+	> 0.8	1.28 to 3.78
Chalk	1	1	< 1 > 1.6	1 1.19 to 1.63	p_ℓ >1.6 MPa 1.2 to 1.55	>1.15	1.58 to 2.15

Summary table of $\xi = \frac{q_s^{Bi}}{q_s^{D}}$ - values for bored - injected piles of type III-A

<u>Table 3.6.</u>

Method / Soil type	CPT test: LPC		CPT test: French prop.	PMT test: LPC		PMT test: French prop.		PMT test: Bustamante/Doix	
	q_c (MPa)			p_ℓ (MPa)		p_ℓ (MPa)		p_ℓ (MPa)	
Clay and Silt	< 0.7	1	1	< 0.55	1	> 1.6	2.06 to 2.73	> 0.6	3.25 to 3.33
	> 2	1.89 to 2.11		> 1.6	2 to 3.25				
Sand and Gravel	5 <<7.5	1.77 to 2.41	1	sand < 0.75	1	> 1.6	1.44 to 1.91	> 0.8	1.92 to 4.09
	> 7.5	1.29 to 1.73		> 2	1.48 to 2.17				
				gravel < 0.75	1				
				> 2	1.75 to 2.5	> 2	1.69 to 1.93		
Chalk	> 4.5	1 to 1.51	1	< 1	1	> 2	1.43 to 2.08	> 1.2	2.00 to 2.92
				> 2.5	1.44 to 1.88				

Summary table of $\xi = \dfrac{(q_s^{Bi})}{q_s^{D}}$ - values for bored - injected piles of type III-B

Table 3.7.

Method	CPT test			PMT test					
Soil type	LPC		French prop.	LPC		French prop.		Bustamante/Doix	
	q_c (MPa)			p_ℓ (MPa)		p_ℓ (MPa)		p_ℓ (MPa)	
Clay and Silt	< 0.7	1	1	< 0.55	1	> 1.6	2.06 to 2.73	> 0.6	1.67 to 1.94
	> 2	1.89 to 2.11		> 1.6	2 to 3.25				
Sand and Gravel	5 <<7.5	1.77 to 2.41		sand < 0.55	1				
	> 7.5	1.29 to 1.73	q_c< 12MPa 1	> 2.30	1.48 to 2.17	> 1.6	1.44 to 1.91		
				gravel < 0.75	1				
			q_c> 12MPa 1.25	> 2	1.75 to 2.50	> 2	1.75 to 2.57	> 0.8	1.08 to 1.5
Chalk				< 1	1				
	> 4.5	1 to 1.51	1	> 1.5	1.15 to 1.21	> 2	1.19 to 1.34	> 1.15	1.6 to 1.36

Summary table of $\rho = \frac{(q_s)_B^{Bi}}{(q_s)_A^{Bi}}$ - values for bored - injected piles.

7. GENERAL COMMENTS AND CONCLUSIONS

7.1. We insist on the fact that if a method is adopted, it has to be followed from the beginning to the end; thus combination of methods is to be avoided.

7.2. In this note, a single group was made for the piles for which the installation procedure involves no pure soil displacement and no pure excavation namely the grouted bored piles. The only proposition examined in this specific way is Bustamante's (1985).
The values given in this proposition seem to be the most precise and specific to this group. On the other hand, it makes no distinction between the two categories of injected piles as far as the base resistance is concerned.

7.3. From the examination of the several methods presented in this paper, it appears that the methods based on the PMT test are more detailed and precise than those based on the CPT-test. This comment is only valid for the French literature examined.

7.4. The installation procedure influences the bearing capacity in very large proportions, as mentioned at the beginning.
We suggest here a summary of general bearing coefficients both for the point resistance and the friction resistance. These coefficients result from the means of the values given in the French literature and are thus representative of the French industry of deep foundations. The values given by De Beer were not taken into account, the given coefficients being global (base + mantle resistance) and relating only to dense sands.

Table 3.7

Soil type	Group I (displacement piles)	Group II (bored piles)	Group III (injected bored piles)		
			A	A+B	B (*)
Clay/silt	1	0.68	0.68	0.86	1
Sand/gravel	1	0.36	0.36	0.56	1
Chalk	1	0.63	0.63	0.77	1

General ζ - base resistance coefficients valid for the French industry of deep foundations.

*) The values given in category A and B are calculated from LPC and French proposition for both CPT and PMT tests. The intermediate column (A+B) averages the coefficients of these two methods and those of Bustamante and Doix (1985).

Table 3.8

Soil type	Group I	Group II	Group III	
			A	B
Clay-Silt	1	0.75	1.17	2.34
Sand-Gravel	1	0.83	1.29	1.80
				1.89
Chalk	1	0.89	1.34	1.63

General ξ - mantle resistance coefficients valid for the French industry of deep foundations.

The values given above are general coefficients. They represent the mean of the values given by the different French methods. This mean is calculated for compact soils (normally safe average)

Bored piles exhibit lower point and friction resistances. The base resistance ratio ζ with regard to the displacement piles is of the order of 0.35 to 0.65, increasing as density decreases. The friction resistance ratio ξ goes from 0.75 to 0.89 increasing on the other hand with density.

As far as the base resistance is concerned, the LPC and the French proposition to Eurocode 7 methods link the bored and grouted piles of A category (Global and Unique Injection) with the bored piles and the piles of B category (Repetitive and Selective Injection) with the displacement piles. The resulting coefficients are thus those of group I and II. We also give an average coefficient for this third group, including then the values suggested by Bustamante. These values show that a global enhancement exists with regard to bored piles but that driven piles will always present a higher base resistance. On the other hand, the friction resistance ratio is systematically higher than 1 with values of 1.2 to 1.35 for category A and of 1.63 to 2.24 for category B. A real enhancement is thus observed for what concerns the friction resistance, with special emphasis for the Repetitive and Selective injected bored piles.

8. REFERENCES

[1] Bustamante, Doix (1985)
Une méthode pour le calcul des tirants et des micropieux injectés. Bull. liaison labo. P. et Ch. - 140 - Nov.-Déc. 1985. (pp 75 - 92)

[2] Bustamante, Haiun, Millan (1985)
Règles provisoires de justifications des fondations sur pieux à partir des résultats des essais pressiométriques. Document réalisé et diffusé par le LCPC et le SETRA.

[3] Bustamante, Gianeselli
Réajustement des Paramètres de calcul des Pieux.
Proceedings of the 10 th ICSMFE, Vol. 2, Session 8, Stockholm, 1981, pp 643-646.

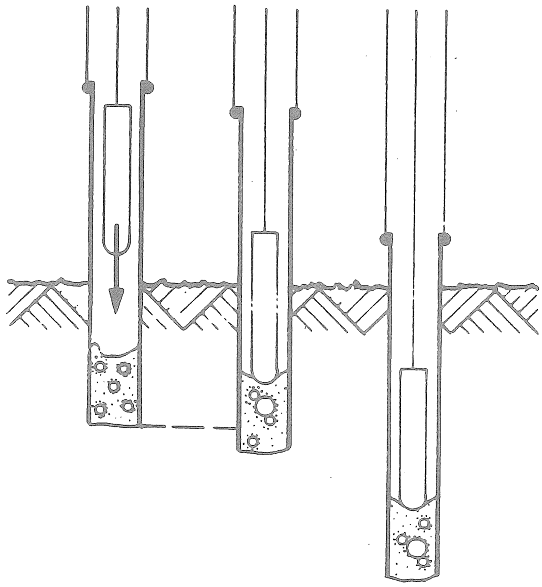
[4] Commission Nationale Belge (1985)
Projet de Norme : "Fondations sur Pieux" - non publié 1985.

[5] E. De Beer
Different behaviour of bored and driven piles. Proc. 6th Conference on Soil Mechanics and Foundation Engineering, Budapest, 1984 (G. Petrasovits ed.) pp 307-318

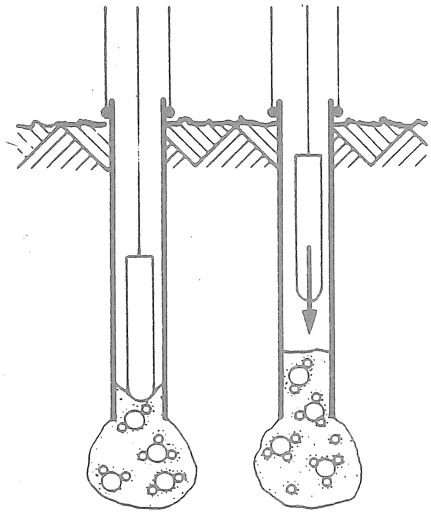
[6] Fascicule 68 (1983)
Execution des travaux de fondations d'ouvrages - Guide pour le calcul des fondations profondes soumises à charges axiales - (projet) - Juin 1983.

[7] Proposition française pour l'établissement de l'Eurocode n° 7 : Fondations - Dimensionnement des pieux à partir des essais in situ : - pressiomètre
- pénétromètre statique.

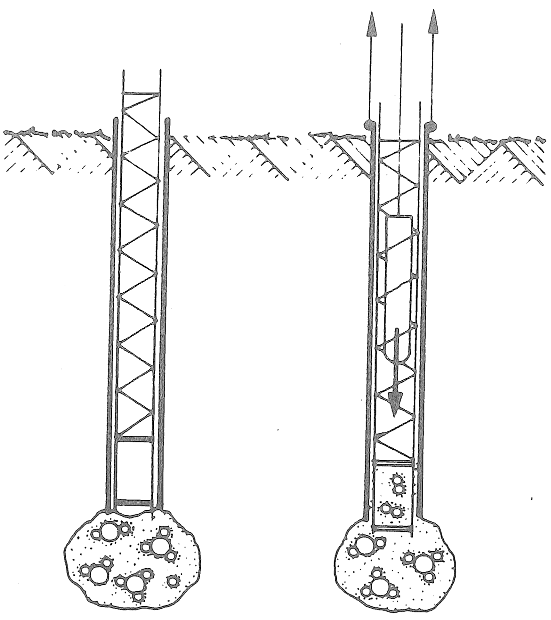
Fig. 21 Execution of the Franki pile

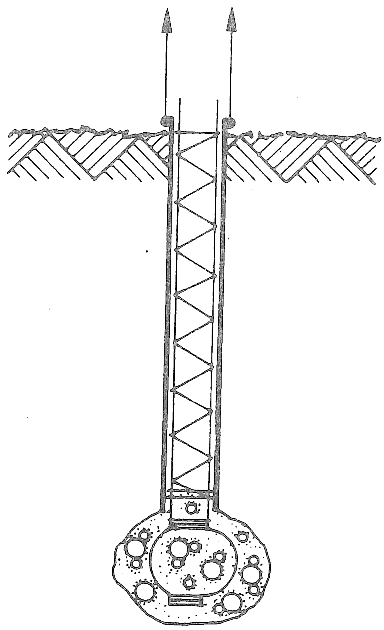


a. Bottom driving with an internal hammer



b. Formation of the Franki base

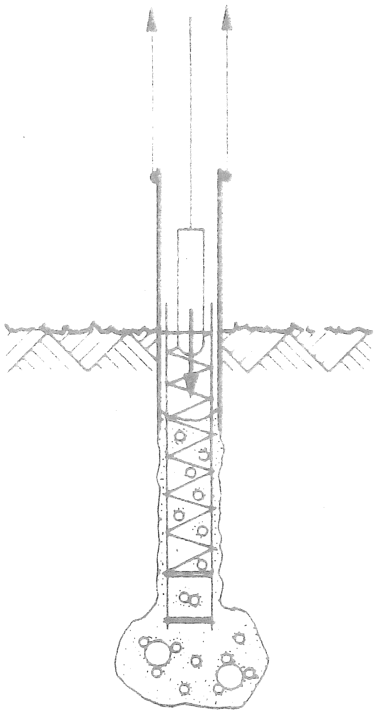


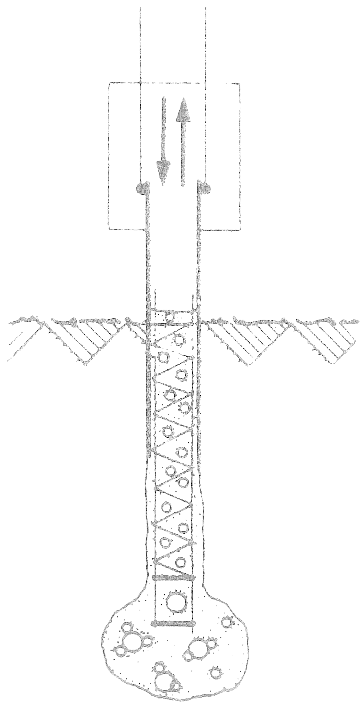


c. Anchoring and reinforcement

Compression pile

Traction pile

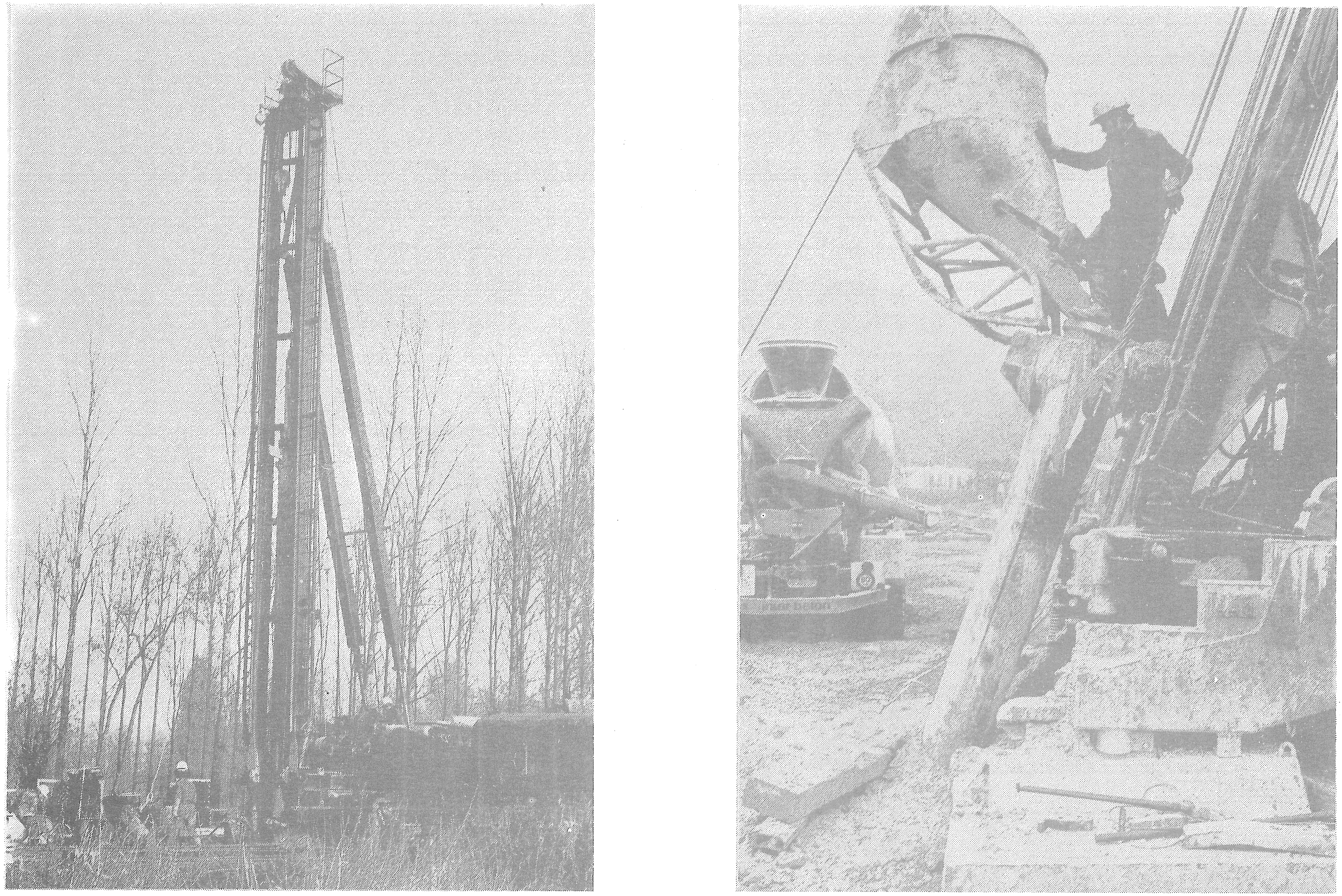




d. Concreting of the shaft

Successive charges of zero concrete are rammed into the soil, simultaneously withdrawing the tube.

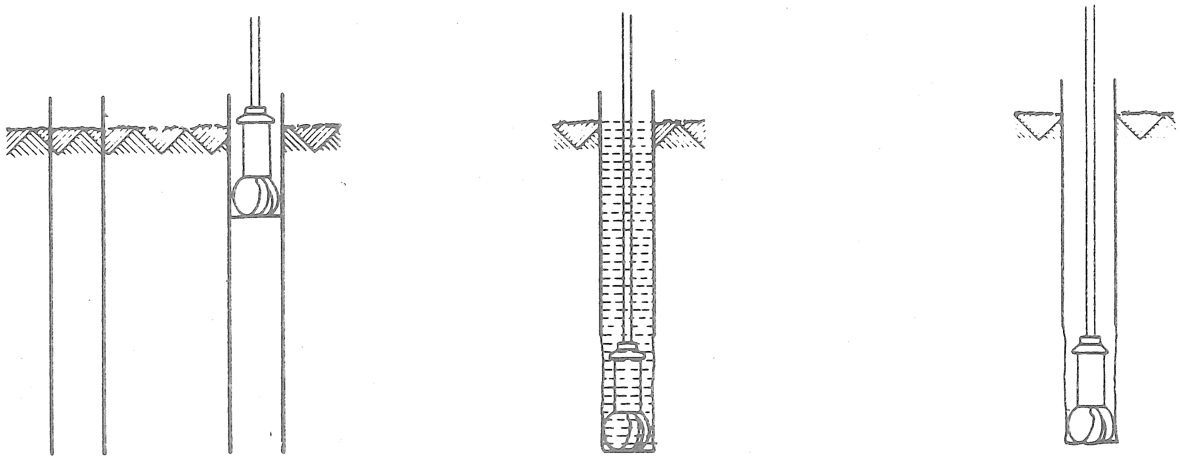
Filling of the casing with high (6-13 cm) slump concrete and extraction of the tube by vibration.



2a 2b

Fig. 2 Example of execution of Franki cast-in-situ piles

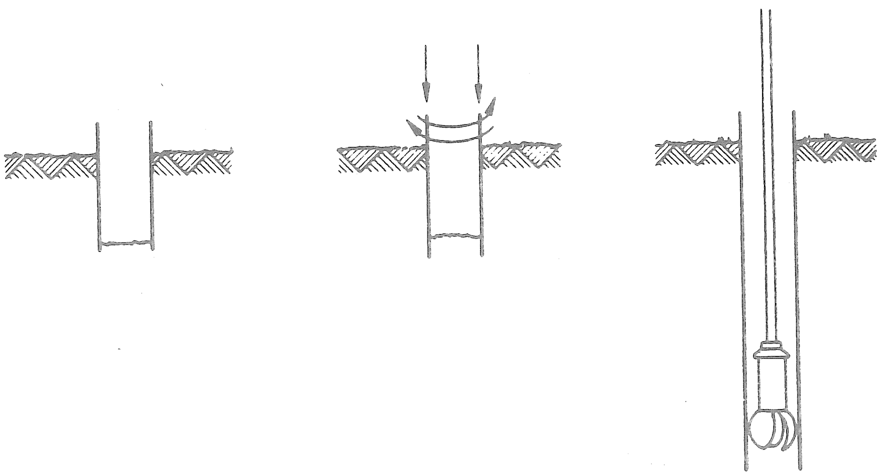
Fig. 2.3 Execution of bored piles



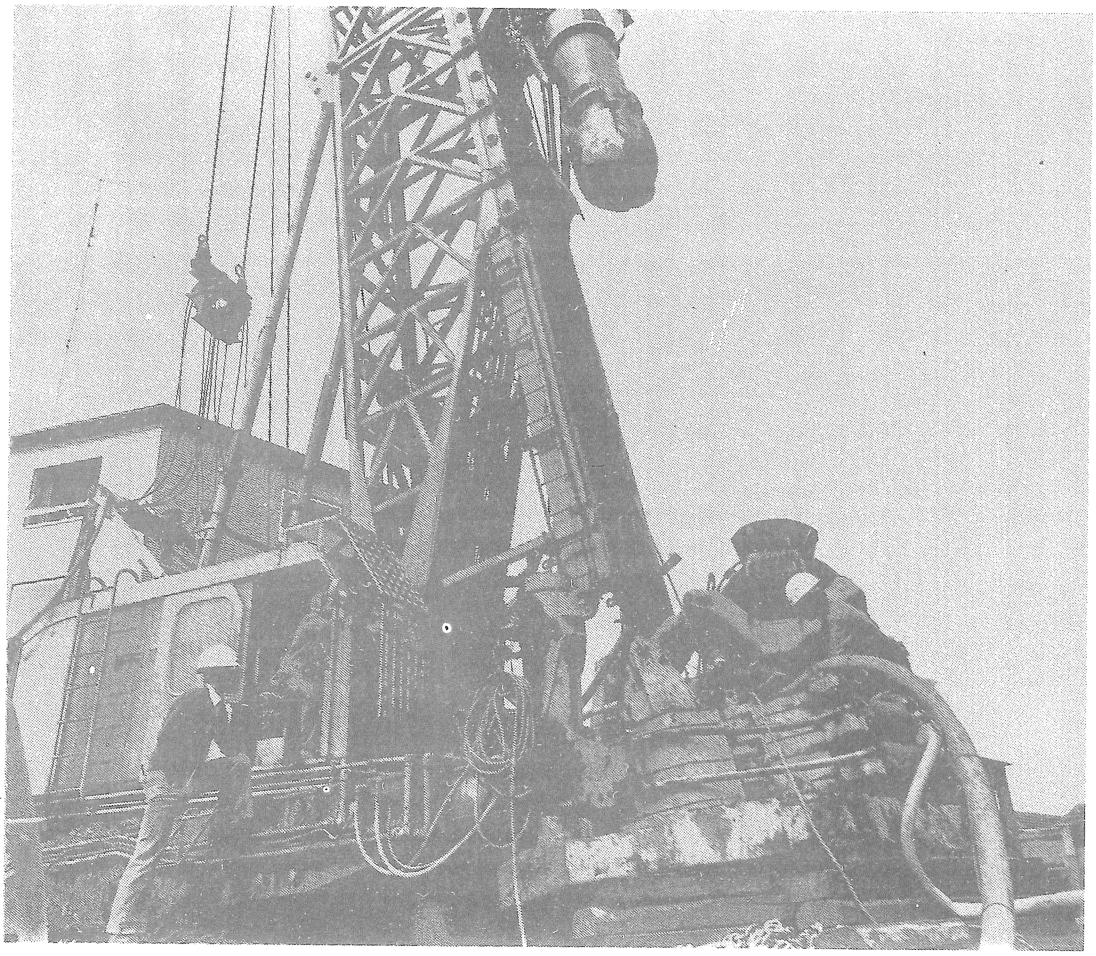
a
Driving of the tube before boring

b
Boring with temporary casing and use of bentonitic mud

c
Dry boring with temporary casing



d Boring with temporary casing executed by oscillation and thrust



4a

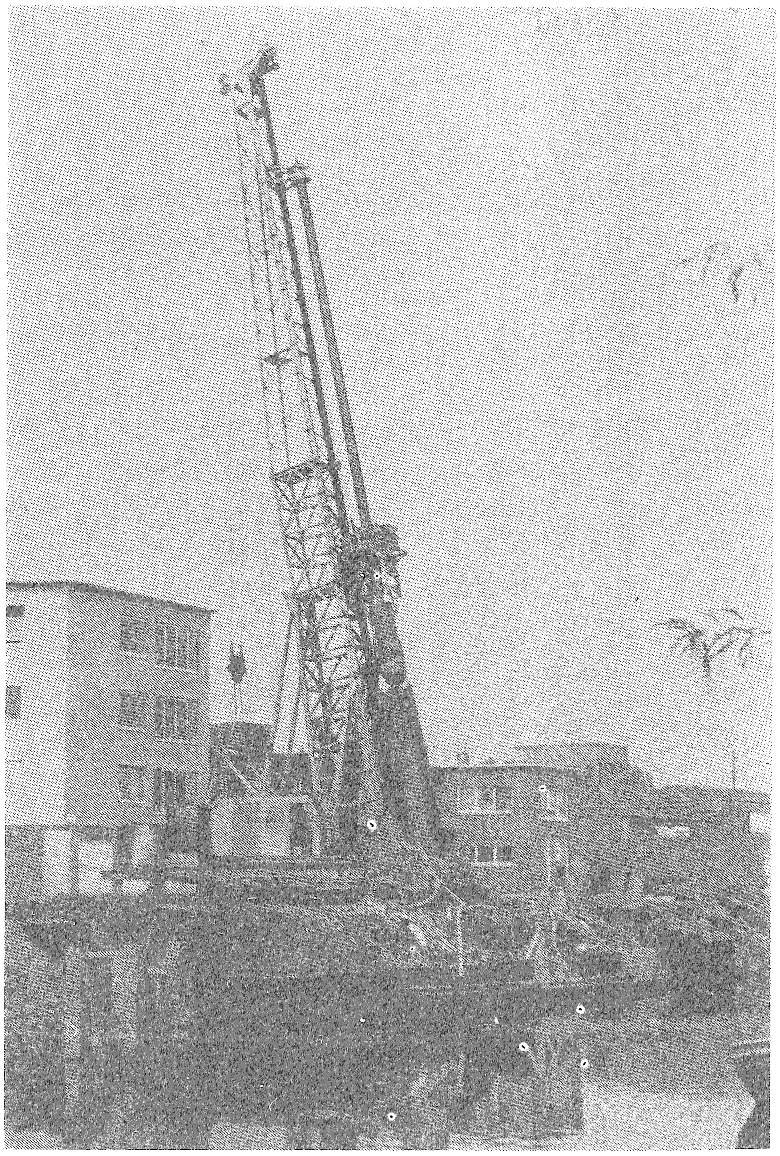


Fig. 4 Example of execution of large diameter bored piles

4b

Fig. 2.5 Execution of small diameter flight-auger piles

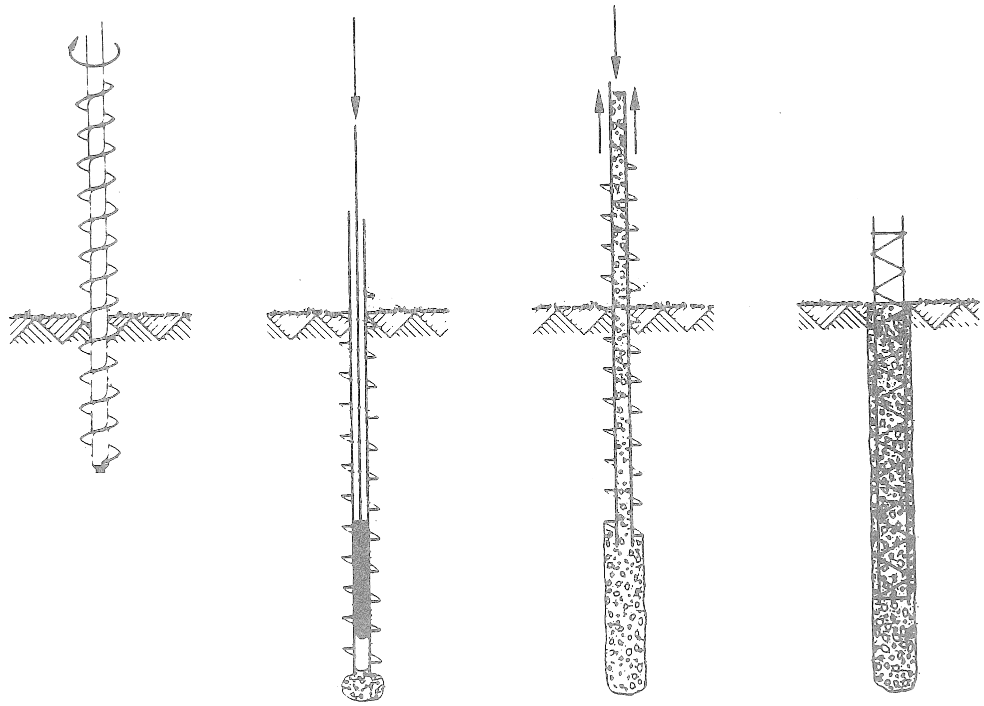


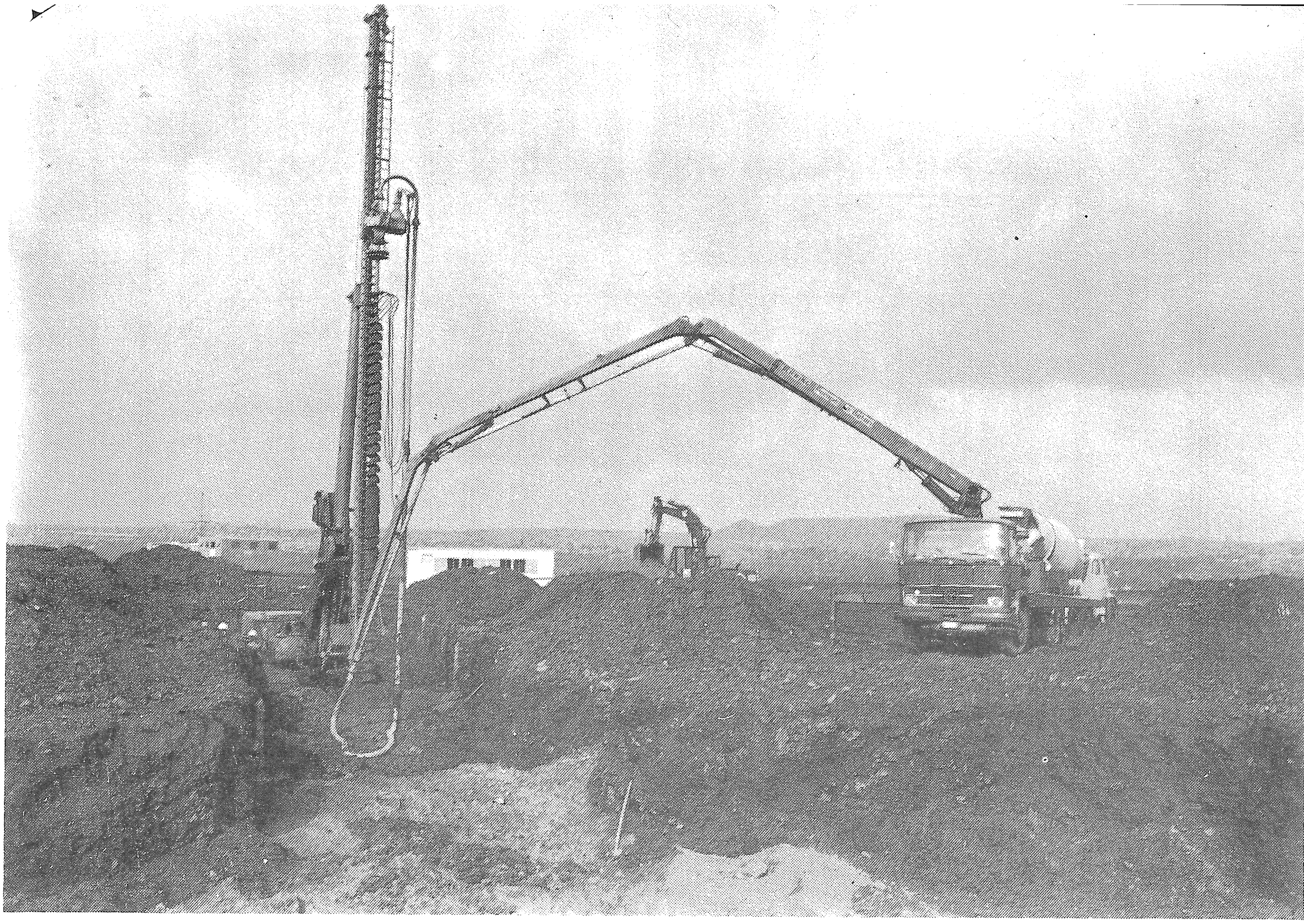
Fig. 2.5a Fig. 2.5b Fig. 2.5c Fig. 2.5d

Fig. 2.5a Drilling with the continuous hollow flight auger

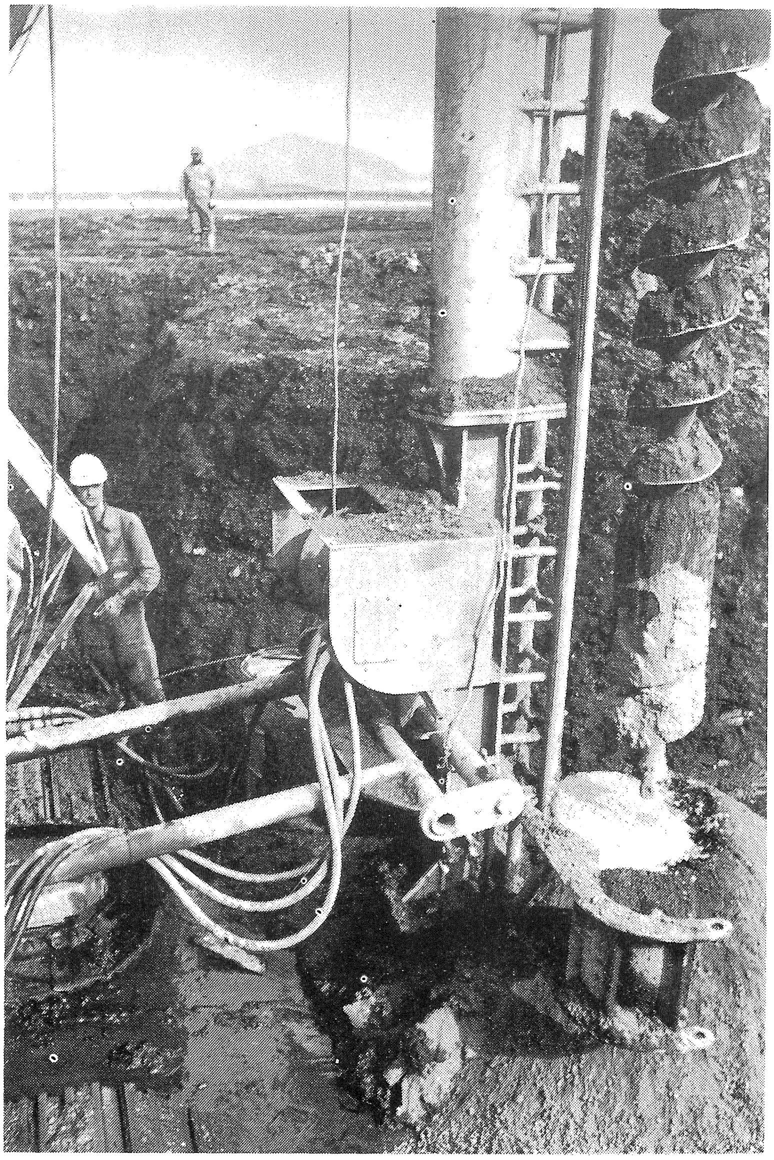
Fig. 2.5b Optional ramming of an enlarged base

Fig. 2.5c Pumping concrete down the hollow shaft of the auger during extraction of the auger

Fig. 2.5d Placement of the steel reinforcing cage



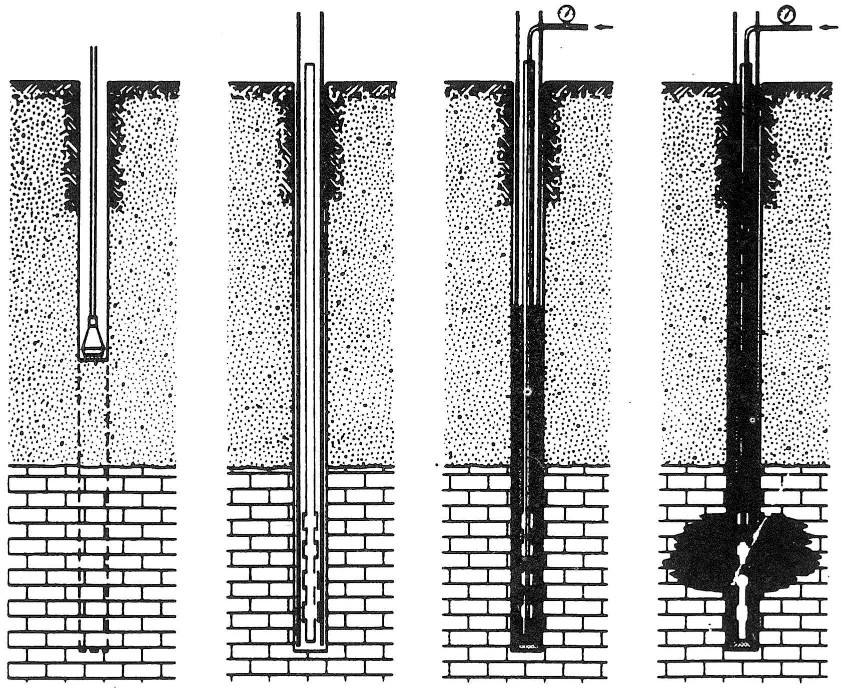
a



b

Fig 2.6 Example of execution of small diameter drilled piles

Fig. 2.7 Execution of micro-piles



a b c d

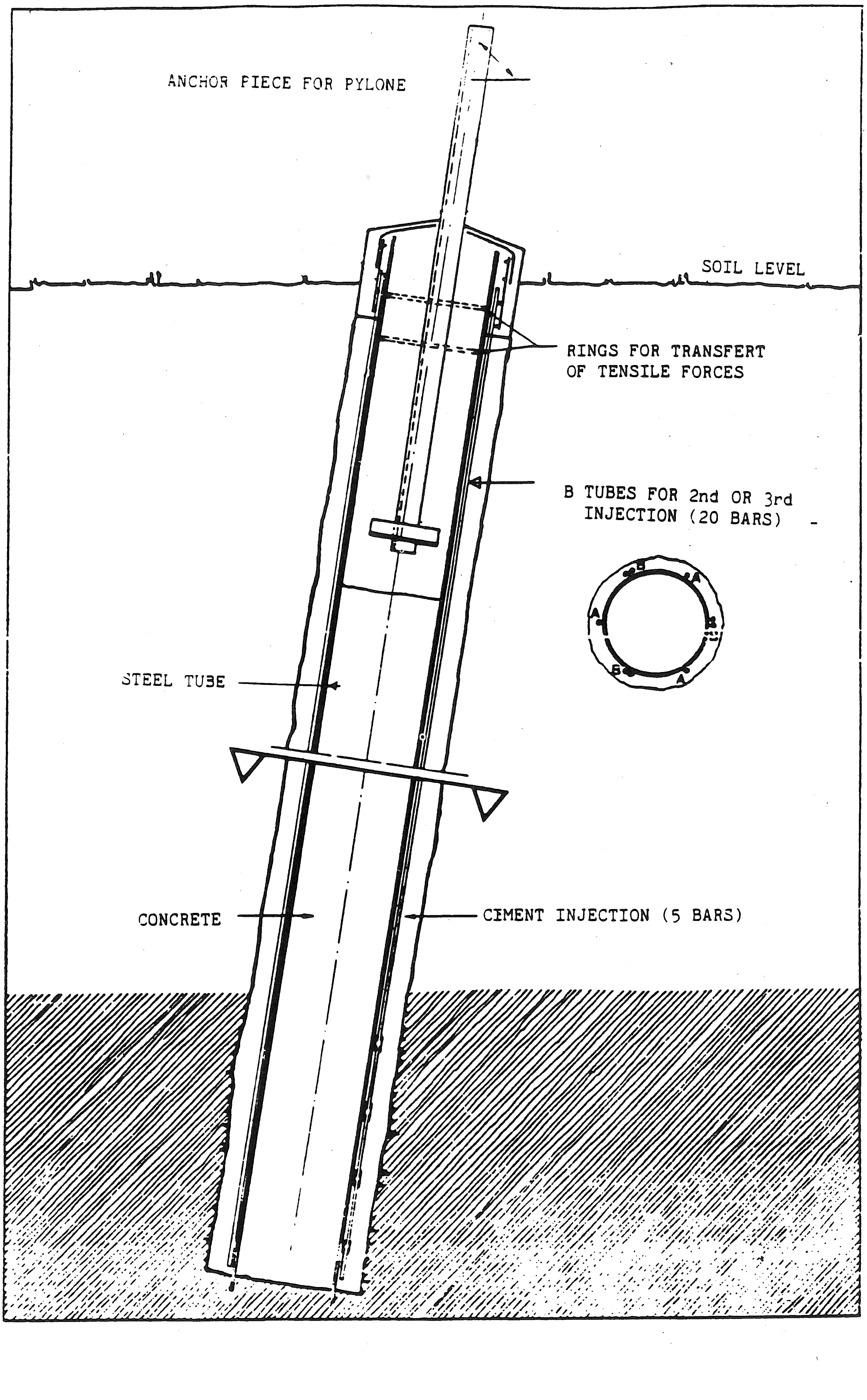
Fig. 2.7a Boring

Fig. 2.7b Placement of the support of the injection tube ("tube à manchette")

Fig. 2.7c Filling with grout

Fig. 2.7d Bulb injection by the injection tube

Fig. 2.8 Execution of large diameter injected piles for the foundation of pylons for high voltage lines



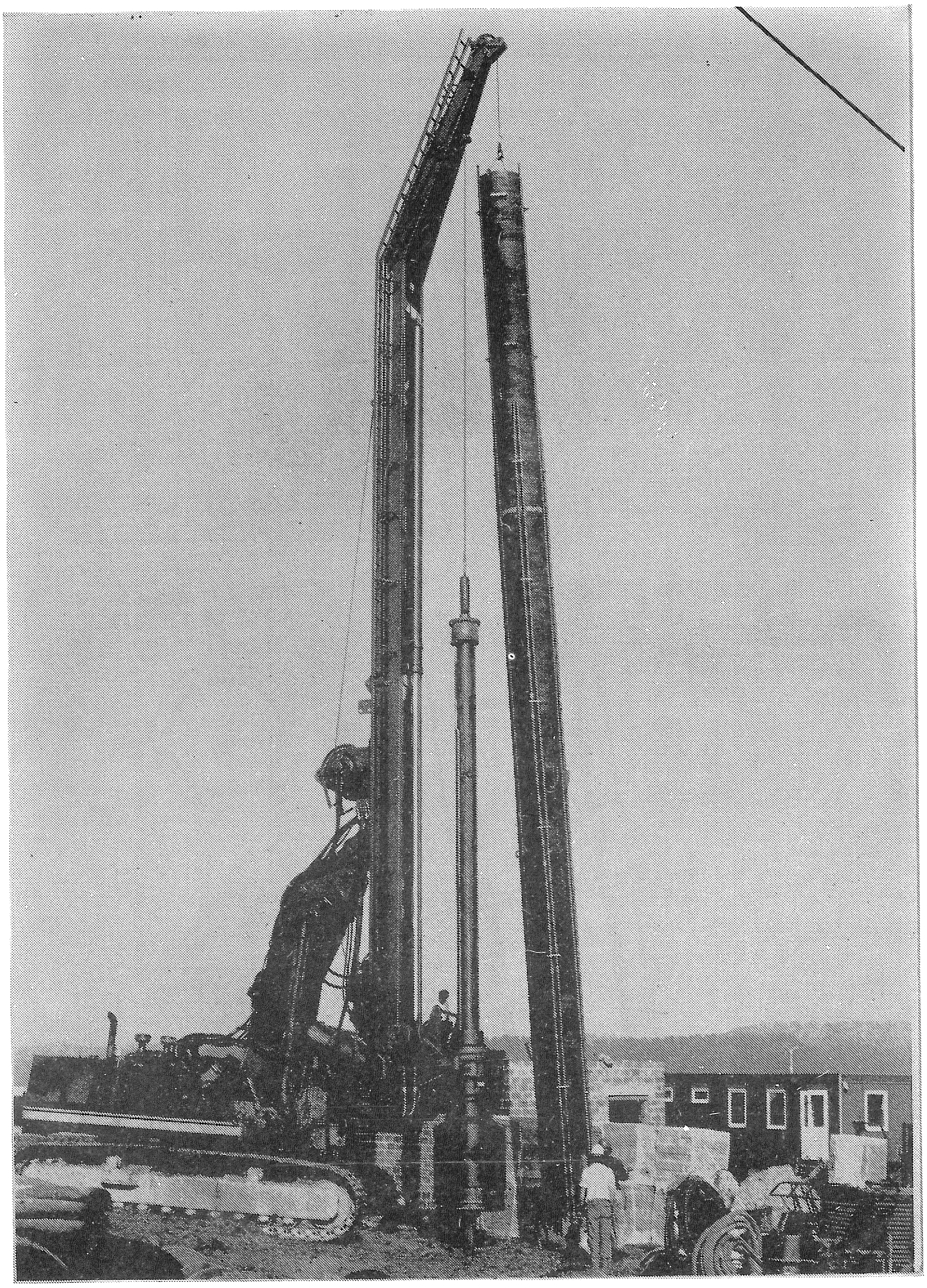


Fig. 2.9 Example of execution of large diameter bored and grouted piles for foundations of pylons for high-voltage lines

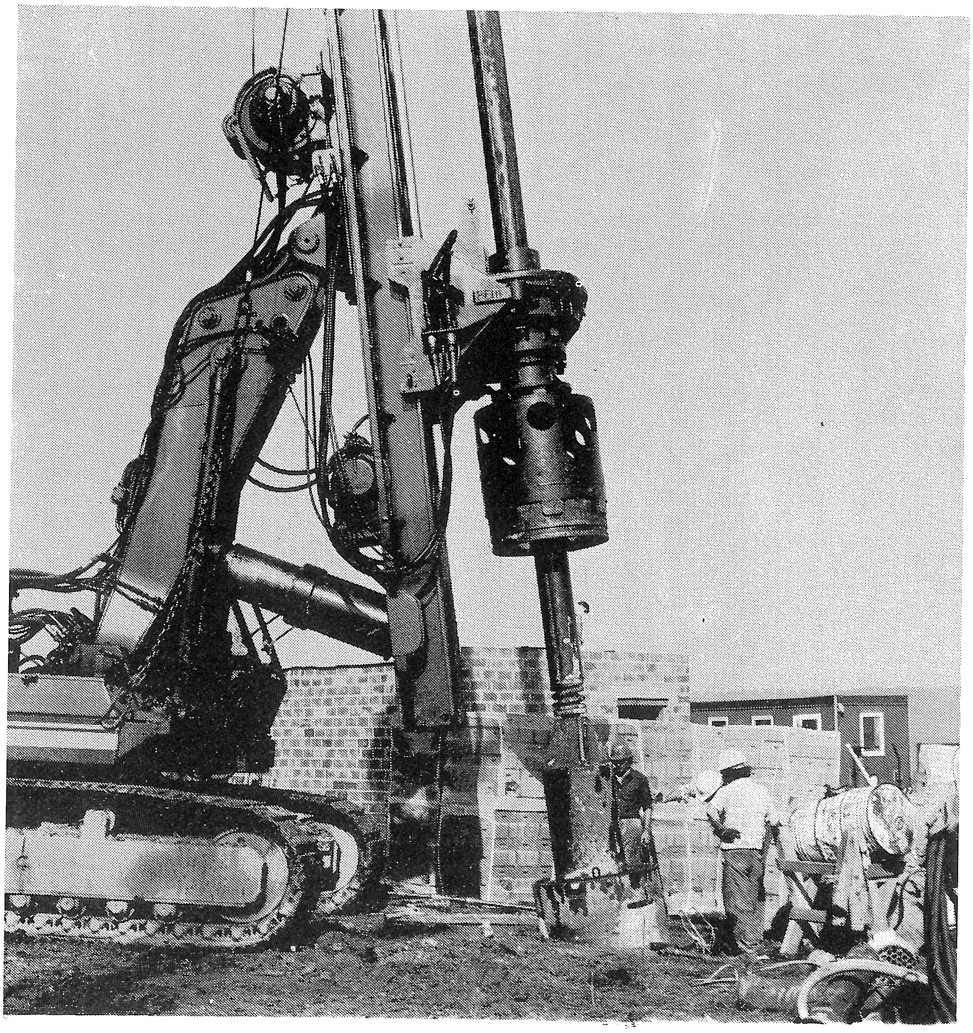


Fig. 2.10

Examples of execution of large diameter bored and grouted piles for foundations of pylons for high-voltage lines

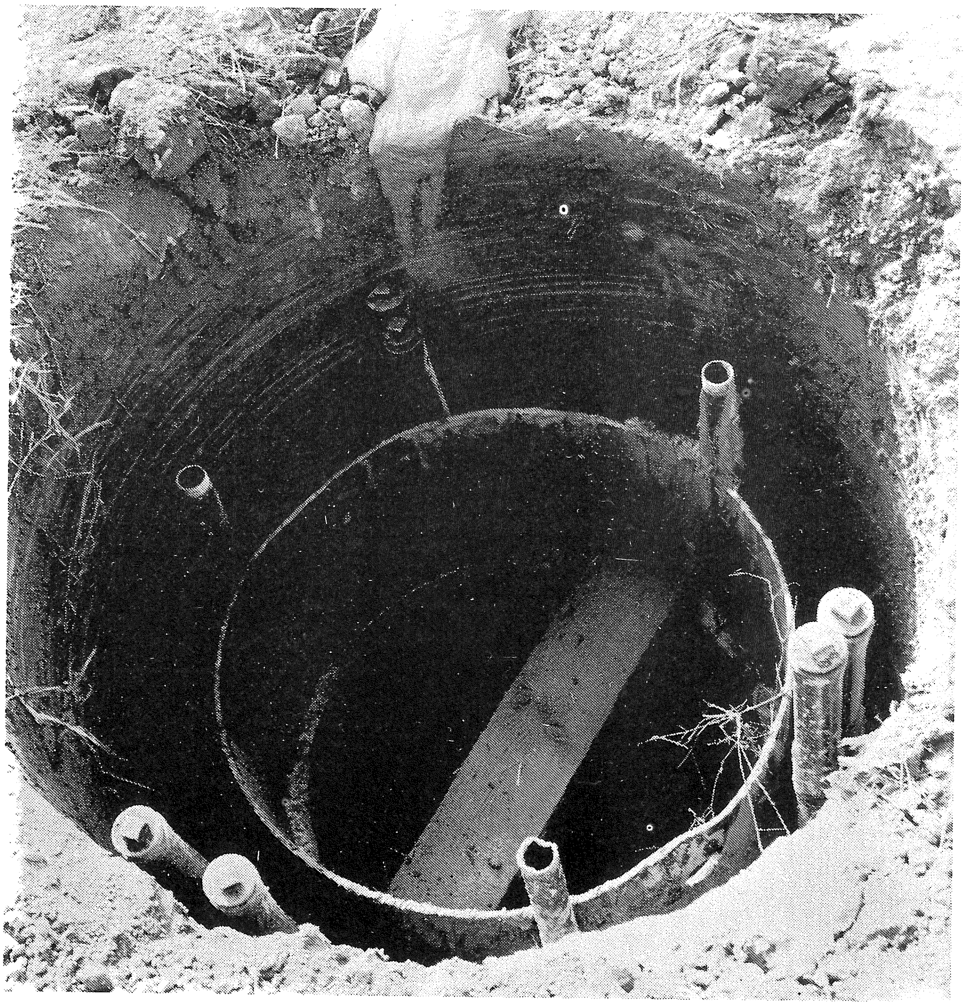


Fig.2.11

Fig. 4.1 Unit limit friction resistance for group I, II, III. A following LPC - cone method

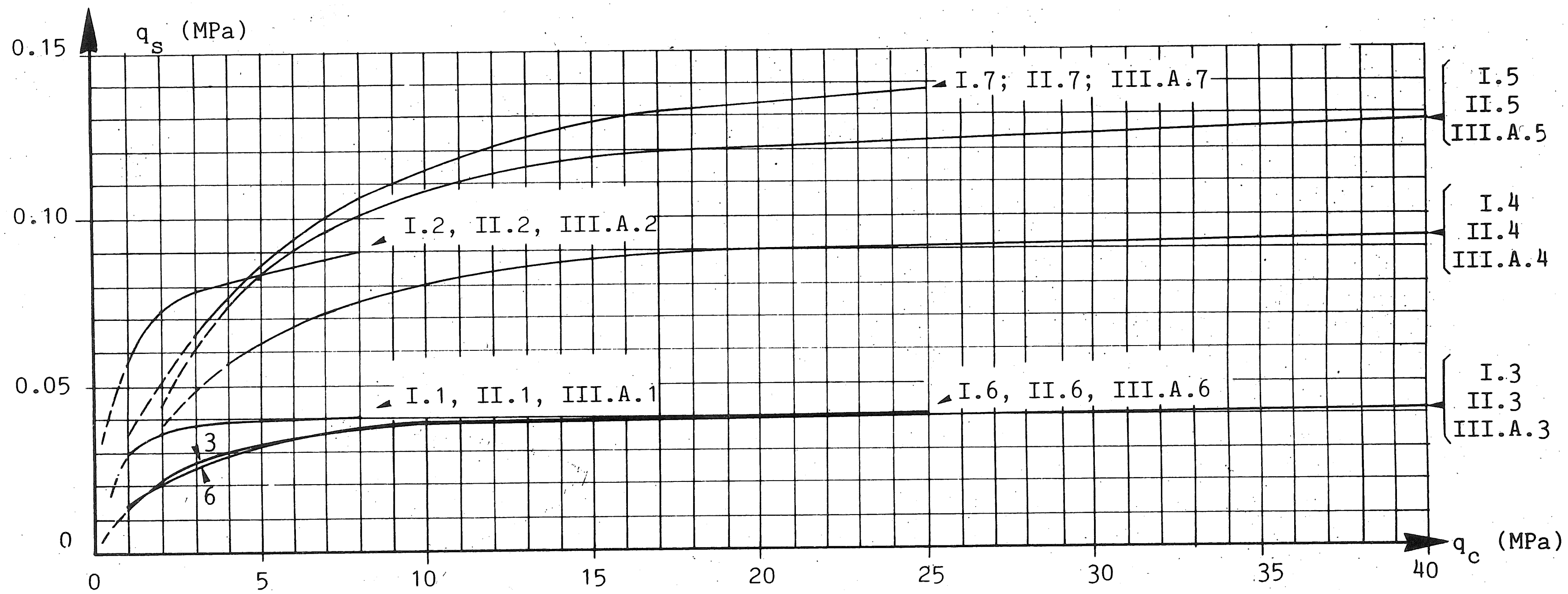


Fig 4.2 Unit limit friction resistance for group I following LPC-pressuremeter method

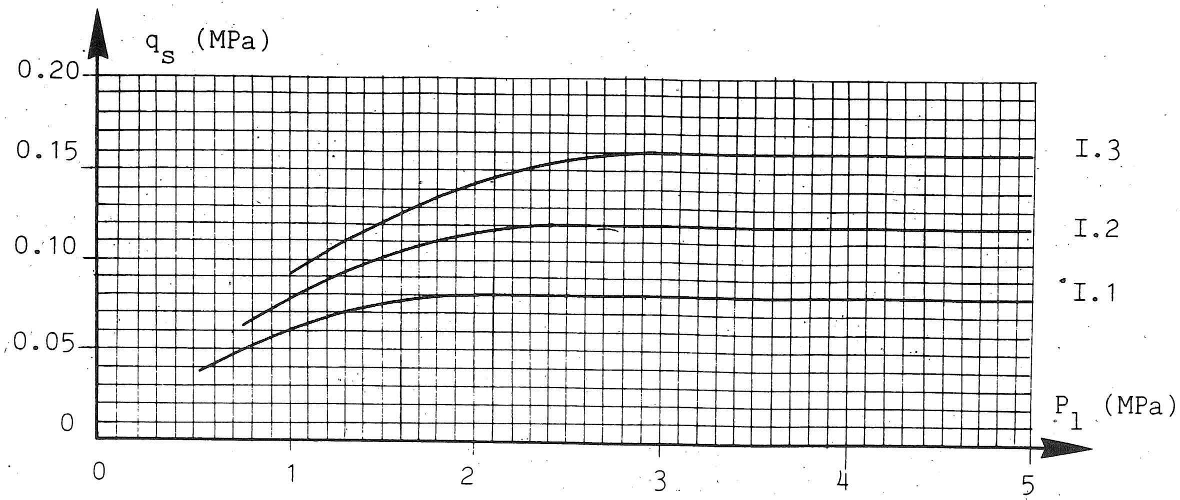


Fig. 4.3 Unit limit friction resistance for group I following French proposition for Eurocode 7

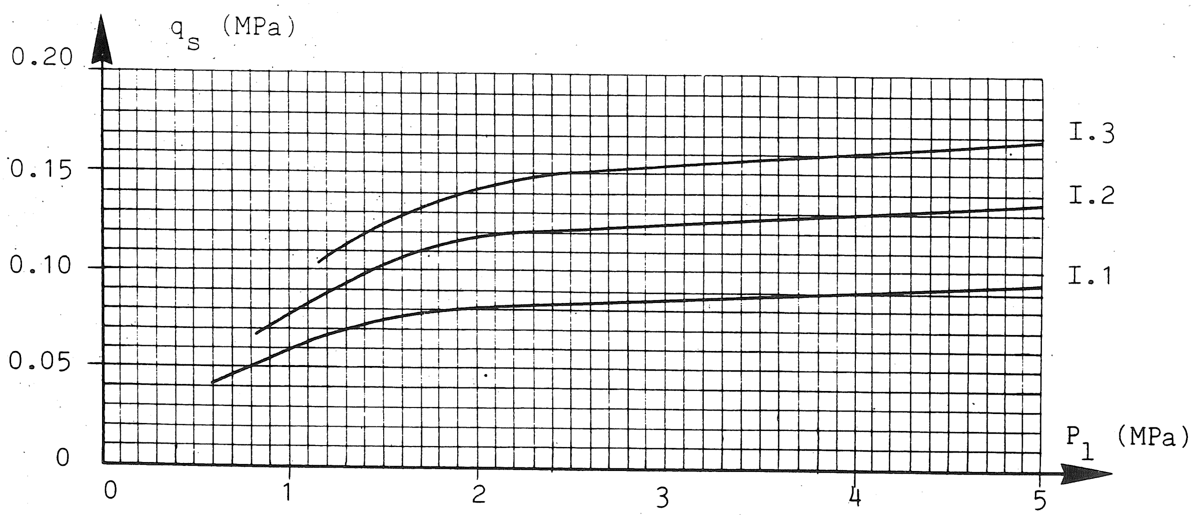


Fig. 5.1 Unit limit friction resistance for group II following LPC-pressuremeter method

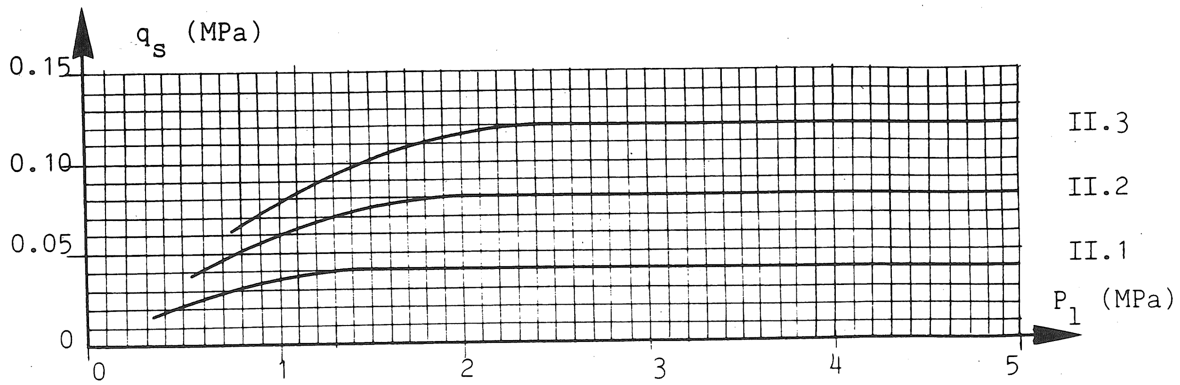


Fig. 5.2 Unit limit friction resistance for group II following French proposition for Eurocode 7

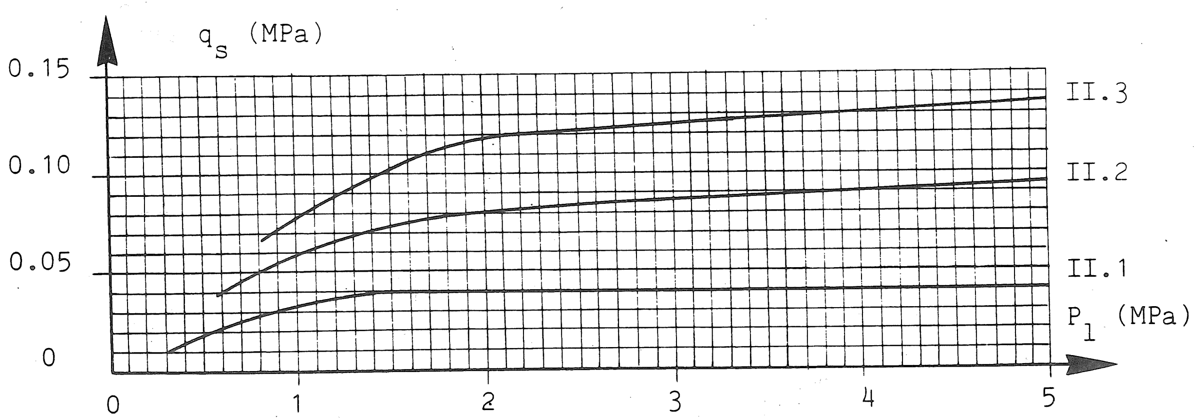
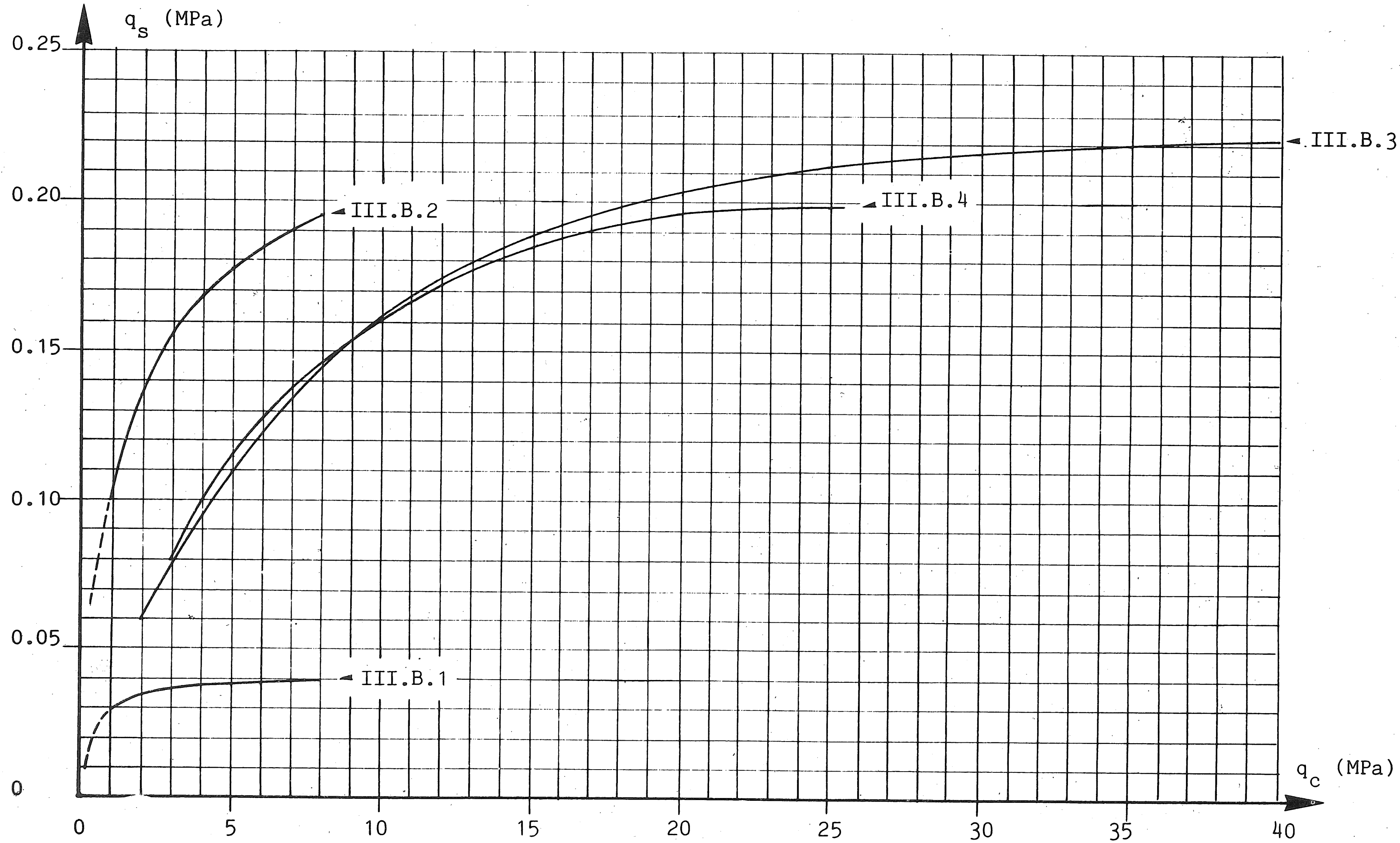


Fig. 61. Unit limit friction resistance for group III. B following LPC-cone method



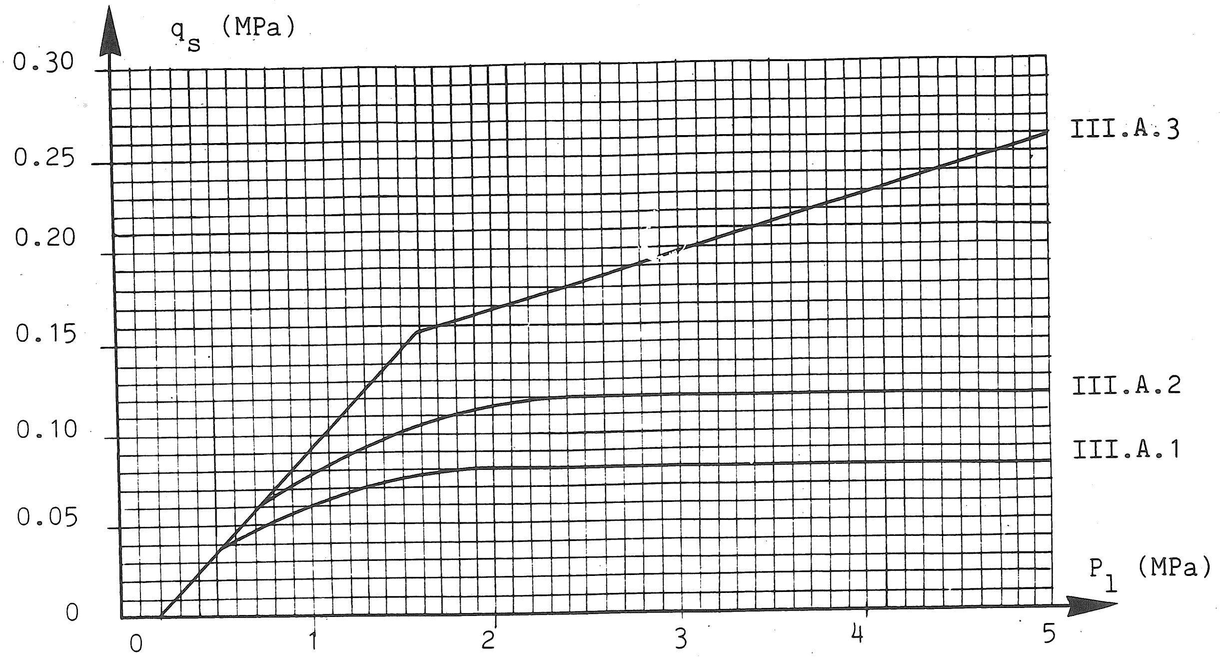
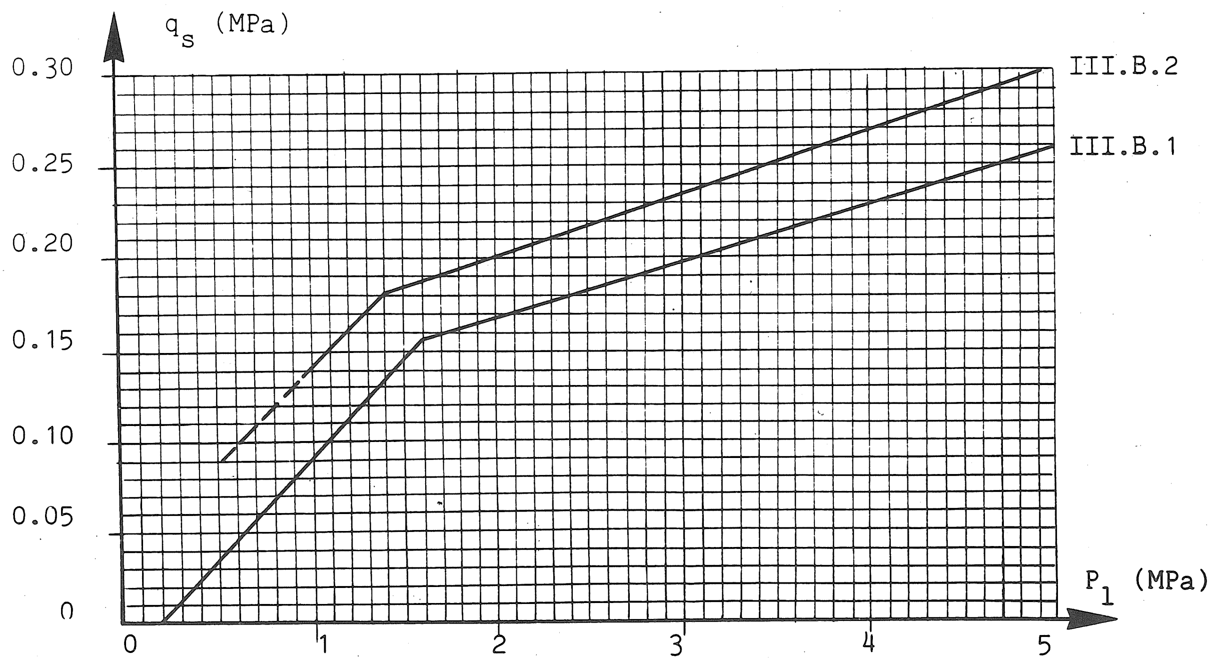


Fig. 6.2 Unit limit friction resistance for group III following LPC-pressuremeter method



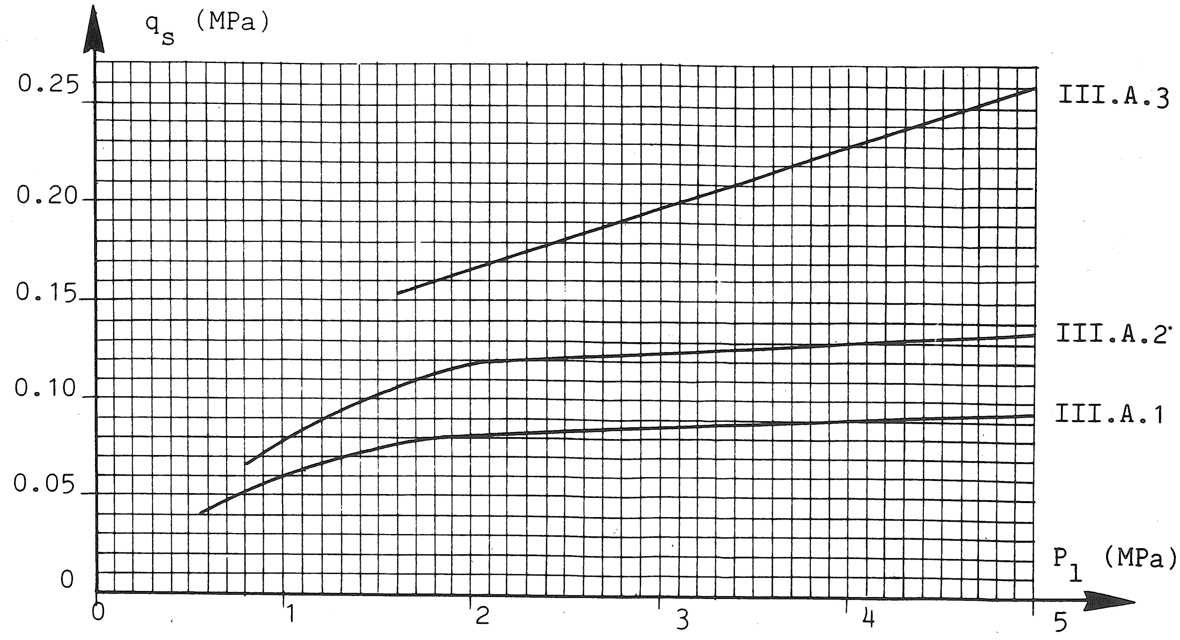


Fig. 6.3 Unit limit friction resistance for group III following French proposition for Eurocode 7

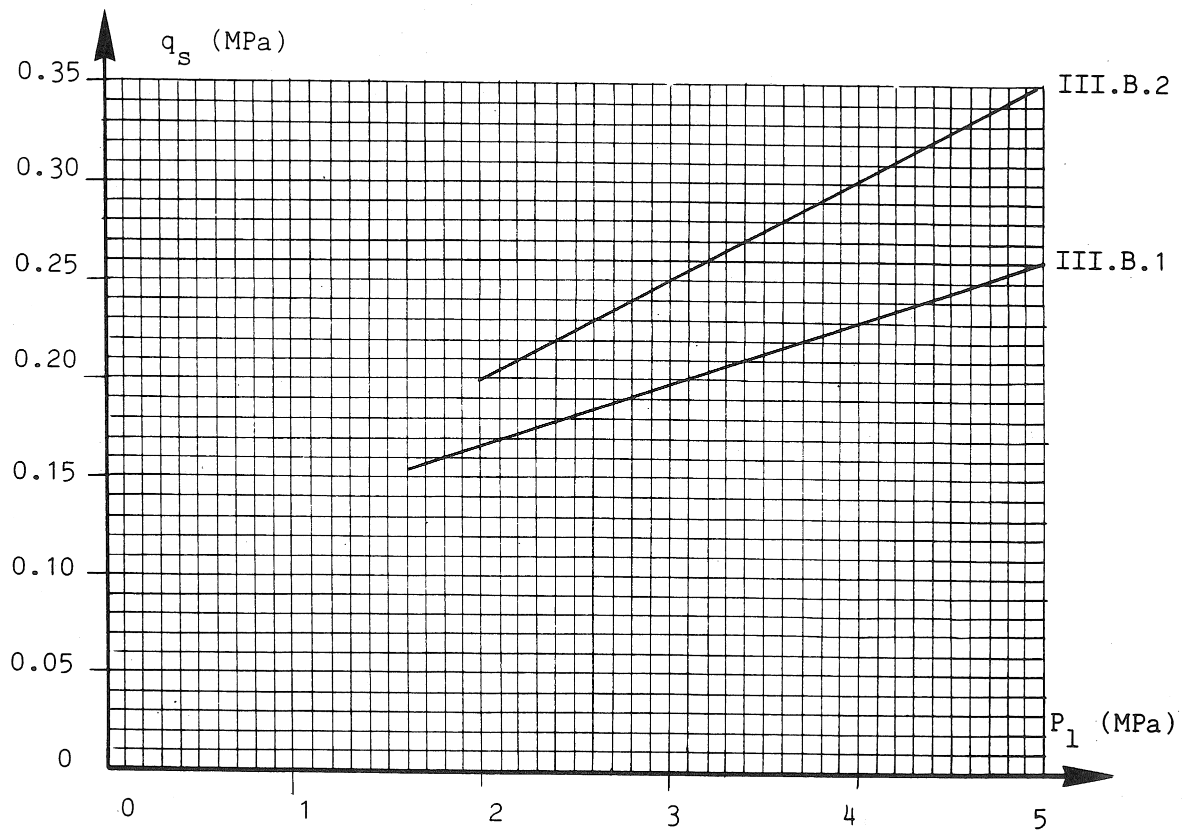


Fig. 6.4 Unit limit friction resistance for group III with the Bustamante's method

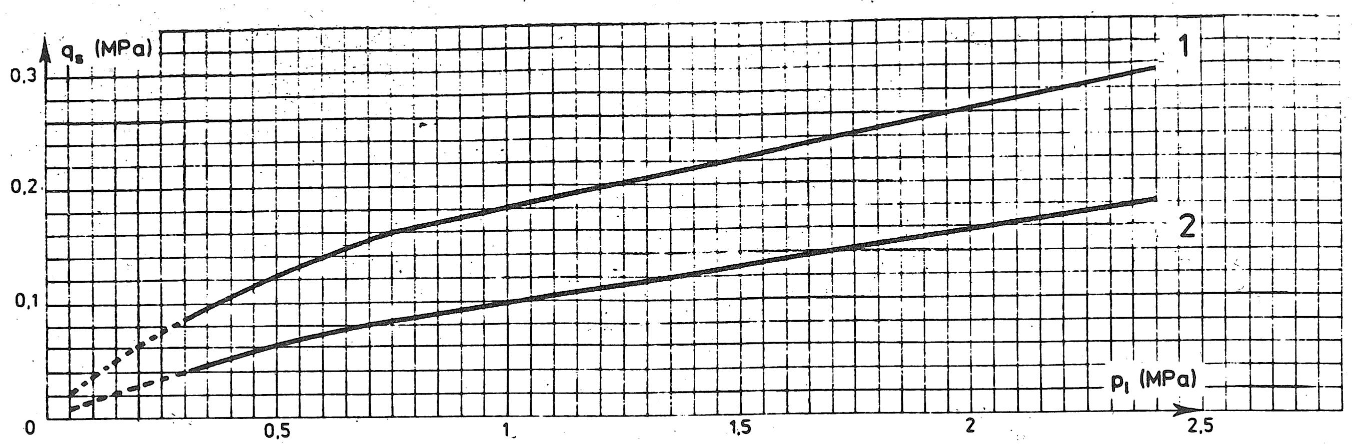


Fig. 6.4a Clay and silt

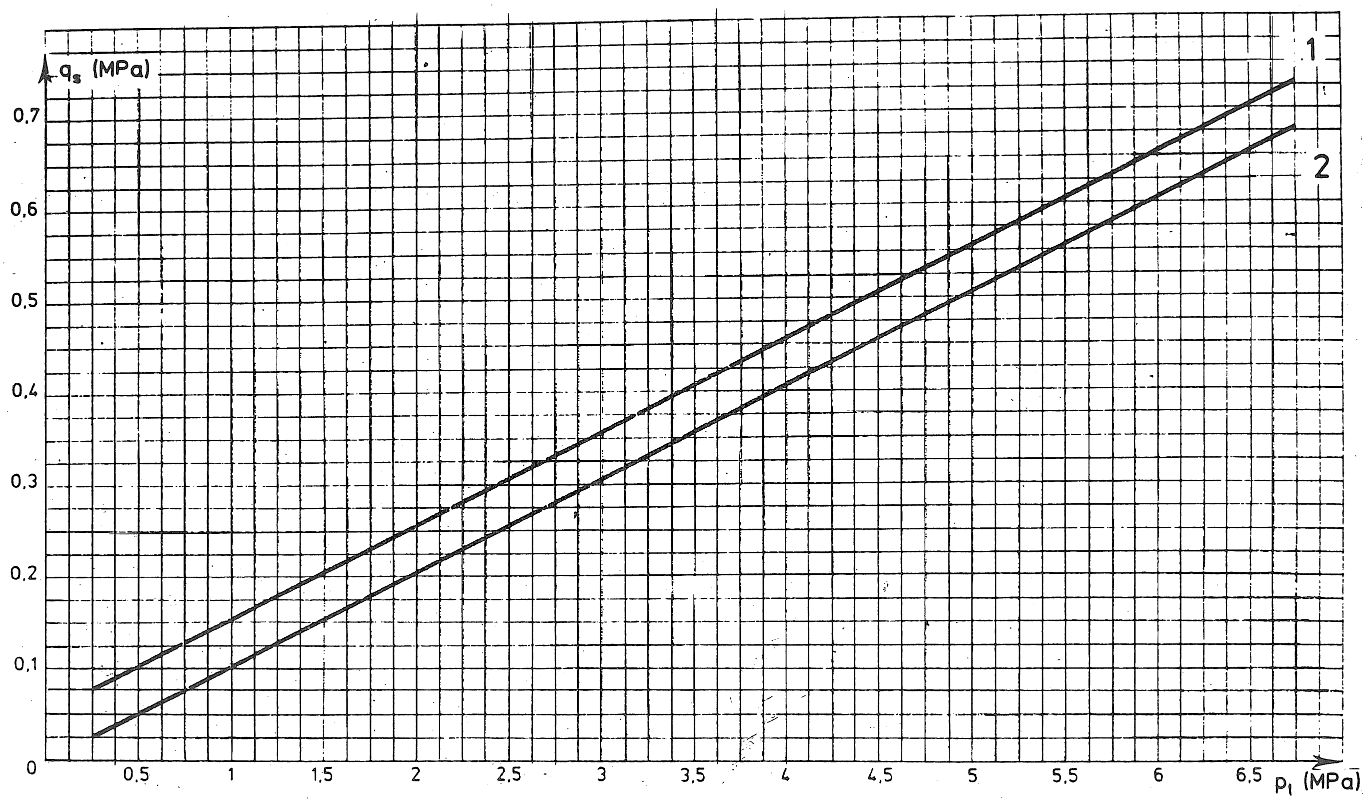


Fig. 6.4b Sand and gravel

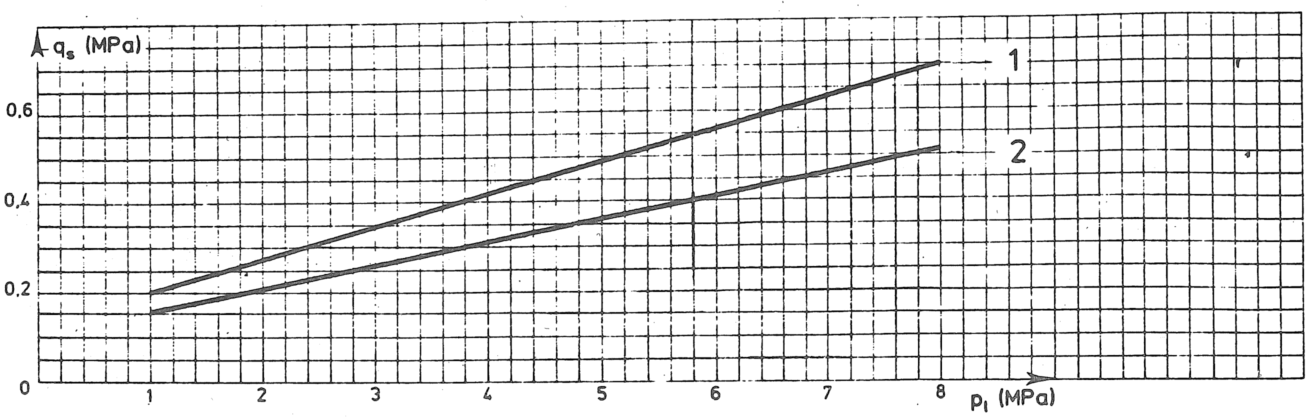


Fig. 6.4c Chalk