New Methods for Determining Bearing Capacity and Settlement of Piles

Nouvelles méthodes pour la détermination des charges admissibles et des tassements de pieux

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Summary

On account of the development of large diameter cast-in-place piles, the costs of load tests have increased considerably. It is therefore very valuable to be able to predict and calculate the behaviour of such piles based on elementary tests in situ.

For this purpose the first mentioned author has developed a general theory for calculating the settlement of individual piles, based on the knowledge of the following two values: modulus of compressibility E (M_E) of the soil underneath the toe of the pile, and the proportion of load sustained by the toe related to skin friction as a percentage of the total load borne by the pile.

For the measurement of E at any desired depth, the authors have employed a specially developed model pile which is inserted into a borehole, the toe being driven about 1 m. into the soil below the bottom of the borehole while the driving resistance is recorded. After carrying out loading tests and measurements of skin friction, the E-modulus can be calculated, based on the above theory.

The practical application of a model pile test is shown by the example of a bored pile. The boring log is carried out in such a way that its area provides a scale for the total boring-energy, and serves as a further important basis for the critical examination of the bearing capacity of bored piles. The results of a test pile of the "Benoto" type, loaded to 450 tons, are discussed by the authors.

Sommaire

Par suite du développement de pieux de gros calibre moulés dans le sol, les frais des essais de charge ont fortement augmenté ; l'intérêt de méthodes permettant de prédire et de calculer le comportement de tels pieux au moyen d'essais élémentaires in situ s'est donc fortement accru.

Dans ce but, le premier des deux auteurs a développé une théorie générale pour le calcul du tassement de pieux isolés, laquelle est basée d'une part sur le coefficient de compressibilité ($E = M_E$) du sol situé sous la pointe du pieu, et, d'autre part, sur le taux de répartition de la charge totale entre la part prise par la résistance à la pointe et celle due au frottement latéral.

Afin de déterminer le coefficient de compressibilité à une profondeur quelconque, un pieu modèle est placé dans le trou de sondage. Puis on le fonce environ 1 m au-dessous de la base du trou de sondage, en mesurant la résistance au battage. Après l'essai de charge du pieu modèle et la mesure du frottement latéral, on peut calculer au moyen de la théorie sus-mentionnée, le coefficient de compressibilité. Une application sur un pieu foré sert d'exemple, illustrant comment les essais avec le pieu modèle sont exécutés et exploités.

Une autre base importante pour la détermination de la portance de pieux forés est le diagramme de forage. L'aire de ce diagramme est une mesure du travail total de forage. Les résultats d'un essai de charge d'un pieu d'essai (système « Benoto ») sous 450 tonnes sont donnés et discutés.

1. Introduction

In order to determine the bearing capacity of a pile, two conditions must be satisfied : on one hand there must be an adequate factor of safety for the bearing capacity, and on the other hand the permissible settlement must not be exceeded. In each individual case it must be decided which of the two criteria is decisive.

In the case of the settlement criterion, this does not usually depend on the settlement of the individual pile, but on the difference of settlement between two piles, or the considerably larger settlement of a group of piles. In order to compute the latter, the modulus of compressibility of the subsoil underneath the toe of the pile must be known. The principal task of the designer is therefore to find an adequate and relatively simple method which permits an estimate to be made of the modulus of compressibility of the subsoil on which the toe of the pile rests, either from the load-settlement curve of a test pile, or with the aid of a model pile. In order to solve this problem, it is essential to know the share and distribution of skin friction. The authors have developed a method of forecasting the settlement of cylindrical piles by using simplified assumptions.

2. Theory for the calculation of settlements of cylindrical piles

1. *Assumptions for the calculations and designations.*

(a) The total pile load P is composed of a point load $P_1 = \lambda P$ and the skin friction $P_2 = (1 - \lambda)$. P, which is distributed uniformly over the entire length of the pile shaft.

If this latter condition is not fullfined, it is sound design to subdivide the pile into different sections, with different but constant skin frictions.

(b) Constant modulus of compressibility $E(M_E)$ over the entire depth of the compressible soil layer underneath the toe of the pile S.

(c) Distribution of the axial stress σ_z in accordance with BOUSSINESQ [1]. The effect of the only partly restrained transverse strain upon the settlement of the pile point S is neglected.

(d) The deformation of the half space underneath the horizontal plane passing through S is not affected by the overlying soil layers (Fig. 1).

Designations used (Fig. 1) :

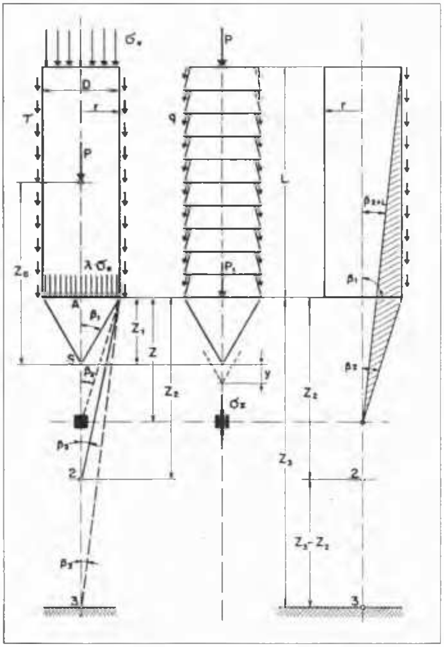

Fig. 1 Method of action of a pile.
Mode d'action d'un pieu (schéma).

r = radius,
D = diameter,
L = active length,
β_1 = half aperture angle of the point,
— of the cylindrical pile
F = $r^2 \cdot \pi$ = cross section,

$N = \frac{L}{D}$,

$\lambda = \frac{P_1}{P}$ = part of the tip load P_1 from the total load P

z = depth underneath the base plane of the point (tip),

z_0 = equivalent effective height of an ideal concentrated load P which is theoretically creating the same settlement as the pile under P,

$n = \frac{z}{r}$ = ratio (relative depth),

B = equivalent diameter of an ideal circular plate at A which, at the same load, is subject to the same settlement as the pile,

E = modulus of compressibility = M_E,

σ_z = vertical axial stress at a depth z,

$\sigma_0 = \frac{P}{F}$ = uniformly distributed load over the cross section of the top of the pile,

σ_{zm} = maximum soil pressure at the toe of the piles (reference stress),

τ = specific skin friction $= \frac{P_2}{M} = (1 - \lambda) \cdot \frac{P}{M}$,

M = active mantle surface,

y = settlement of the tip of the pile S,

C_1, C_2 & C_3 = integration constants.

2. *Method of calculation.*

The settlement y of the tip of the pile is composed of two parts :

$y = y_A + y_B$,

wherein

y_A = settlement due to point load (circular load $\lambda\sigma_0$) in A,

y_B = settlement due to pile shaft load (skin friction τ).

The calculation of y_A is known (Fig. 1) :

For a thickness of the layers fixed by the angles β_1 and β_3 we obtain [2, 3] (1) :

$$y_A = \frac{\lambda \cdot \sigma_0}{E} \left[\sin\beta(2 + \cos^2\beta) - \text{cotg}\,\beta(1 - \cos^3\beta) \right]_{\beta_3}^{\beta_1} \quad \ldots\ldots (2)$$

$$\text{for } \beta_1 = \frac{\pi}{2};\ \beta_3 = 0: y_A = \frac{\lambda \cdot \sigma_0}{E} \cdot D,$$

$$\text{for } \beta_1 = \frac{\pi}{6};\ \beta_3 = 0: y_A = C_1 \cdot \frac{\lambda \cdot \sigma_0}{E} \cdot D;\ C_1 = 0{\cdot}384$$

For the calculation of y_B, the skin frictions, effective upon a circular strip of the pile shaft surface, has been replaced by a respective circular vertical load q, therefore :

$$q = \frac{T}{2L} \cdot (1 - \lambda) \cdot \frac{\Delta z}{\Delta r} \cdot \sigma_0 \quad \ldots\ldots (3)$$

compare Fig. 2.

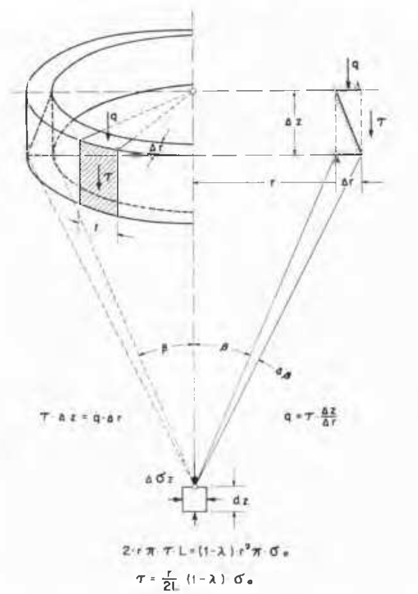

Fig. 2 Lateral friction is replaced by normal stresses.
Le frottement latéral est remplacé par des contraintes normales.

The integration gives [3] :

$$y = \frac{\sigma_0}{E} \cdot D \cdot \Phi_c = \frac{\sigma_0}{E} \cdot B; \; B = D \cdot \Phi_c \tag{4}$$

$$\Phi_c = \lambda \cdot C_1 + (1 - \lambda)(C_2 + C_3) = C_2 + C_3 + \lambda(C_1 - C_2 - C_3)$$

$$C_1 = \sin\beta_1(2 + \cos^2\beta_1) - \operatorname{cotg}\beta_1(1 - \cos^3\beta_1)$$

$$C_2 = \frac{3}{4} \cdot \frac{r}{L} \int_{z_1}^{z_2} \left[\sin\beta - \frac{\sin^3\beta}{3} \right]_{\beta_{z+L}}^{\beta_z} \cdot dn; \; n = \frac{z}{r}$$

$$C_3 = \frac{3}{4} \cdot \frac{r}{L} \cdot \ln \frac{1 + \frac{L}{z_2}}{1 + \frac{L}{z_3}}; \; \text{for } z_3 = \infty : C_3 = \frac{3}{4} \cdot \frac{r}{L} \cdot \ln\left(1 + \frac{L}{z_2}\right)$$

The integration for C_2 takes place graphically and comprises the range from z_1 to $z_2 = 25n$ ($n = 25$).

3. *Application to the model pile* 1959.

A schematic representation of the testing procedure is given in Fig. 3.

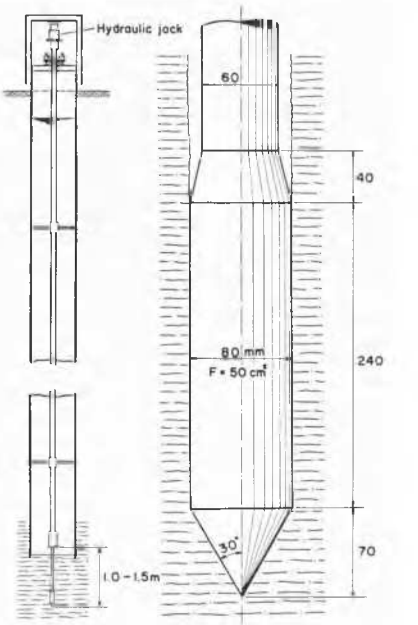

Fig. 3 Sketch showing the working principle of a model pile. Esquisse de principe pour le pieu-modèle.

(a) *Dimensions* :

$D = 8$ cm; $L = 24$ cm; $N = \frac{L}{D} = 3$; $\beta_1 = 30°$

$F = 50$ sq. cm; $M = D \cdot \pi \cdot L = 8 \cdot 24 \simeq 600$ sq. cm.

(b) *Calculation of the equivalent diameter* B :

After equ. (3) we obtain :

$$B = D \cdot \Phi_c = D[C_2 + C_3 + \lambda(C_1 - C_2 - C_3)]$$

$$B = 8{\cdot}0 \cdot \left[\frac{1{\cdot}1137}{8} + \frac{0{\cdot}2624}{8} + \frac{\lambda}{8}(0{\cdot}384{\cdot}8 - 1{\cdot}1137 - 0{\cdot}2624) \right]$$

$$B = 1{\cdot}3761 + 1{\cdot}696 \cdot \lambda(\text{cm}) \text{ for } z_3 = \infty$$
$$\Phi_c = 0{\cdot}172 + 0{\cdot}212 \cdot \lambda \text{ for } D = 8 \text{ cm} \quad \ldots\ldots \tag{5}$$

(c) *Calculation of the settlement* y *of the pile point* :

$$y = \frac{\sigma_0}{E} \cdot B = \frac{\sigma_0}{E} \cdot (1{\cdot}376 + 1{\cdot}696 \cdot \lambda) \tag{6}$$

(d) *Determination of the modulus of compressibility* $E(M_E)$:

Based upon the measured values of y and λ, the modulus E can be calculated as follows (equ. (6)) :

$$E = \frac{\sigma_0}{y} \cdot B = \frac{\sigma_0}{y} \cdot (1{\cdot}376 + 1{\cdot}696 \cdot \lambda) \tag{6a}$$

For a change of load $\Delta\sigma_0$ follows analogously :

$$E = \frac{\Delta\sigma_0}{\Delta y} \cdot B = \operatorname{cotg}\alpha \cdot B,$$

wherein :

α = inclination of the secant of the load settlement curve.

(e) *Calculation of the reference stress* σ_{zm} :

As E changes with the stress (consolidation pressure), the reference stress is defined as the vertical stress at the toe of the model pile and is calculated as follows :

$$\sigma_{zA} = \lambda \cdot \sigma_0(1 - \cos^3\beta) = 0{\cdot}35 \cdot \lambda \cdot \sigma_0 \text{ for } \beta = \frac{\pi}{6} = 30°$$

$$\sigma_{zB} = \frac{3}{2}(1 - \lambda)\frac{r}{L} \cdot \sigma_0 \left[\sin\beta - \frac{\sin^3\beta}{3} \right]_{\beta_{z_1+L}}^{\beta_{z_1}}$$

$$= \frac{3}{2}(1 - \lambda)\frac{4}{24} \cdot \sigma_0 \cdot 0{\cdot}33 = \frac{1}{12}(1 - \lambda) \cdot \sigma_0$$

$$= \frac{\sigma_0}{10}(0{\cdot}83 - 0{\cdot}83 \cdot \lambda)$$

$$\sigma_{zm} = \sigma_{zA} + \sigma_{zB} = \frac{\sigma_0}{10}(0{\cdot}83 + 2{\cdot}67 \cdot \lambda) \tag{7}$$

for $\lambda = 0 : \sigma_{zm} = 0{\cdot}083 \cdot \sigma_0$ (skin friction only),

for $\lambda = 1 : \sigma_{zm} = 0{\cdot}350 \cdot \sigma_0$ (no skin friction).

(f) *Calculation of the equivalent effective height* z_0 :

The settlement of the point S, dut to an ideal concentrated load P in a distance z_0 above S, is calculated as follows [4] :

$$y = \frac{3 \cdot P}{2 \cdot \pi \cdot E \cdot z_0} \text{ for } z_3 = \infty \text{ (Fig. 1)}$$

On the other hand, according to equ. 5 :

$$y = \frac{\sigma_0}{E} \cdot B; \quad \sigma_0 = \frac{P}{r^2 \cdot \pi}$$

By equalizing the two expressions we obtain :

$$z_0 = \frac{3}{2} \cdot \frac{r^2}{B} = \frac{3}{8} \cdot \frac{D}{\Phi_c} = \frac{3}{0{\cdot}172 + 0{\cdot}212 \cdot \lambda} \text{ in cm}$$

The representative diagrams are shown in Fig. 4.

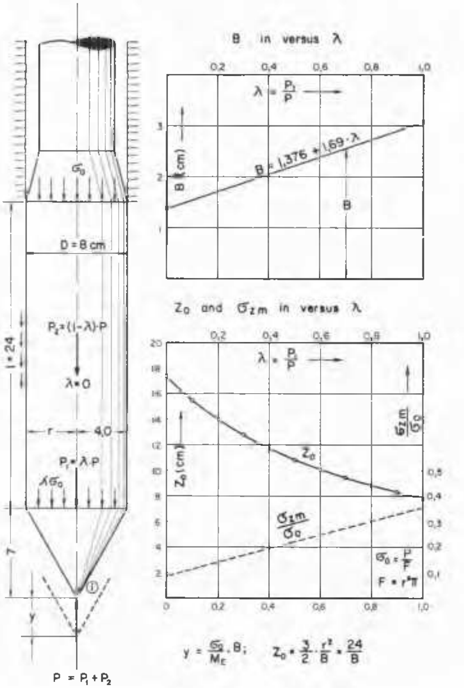

Fig. 4 Diagram showing how to determine E (M_E) with the help of a model pile.
Diagramme permettant de déterminer le module E (M_E) à l'aide du pieu-modèle.

N	Φ_c (see Fig. 5)	Φ_c for $= 1{\cdot}0$
20	$0{\cdot}0747 + 0{\cdot}9253 \cdot \lambda$	1·0
30	$0{\cdot}0549 + 0{\cdot}9451 \cdot \lambda$	1·0
40	$0{\cdot}0437 + 0{\cdot}9563 \cdot \lambda$	1·0
50	$0{\cdot}0366 + 0{\cdot}9634 \cdot \lambda$	1·0
70	$0{\cdot}0262 + 0{\cdot}9738 \cdot \lambda$	1·0
100	$0{\cdot}0210 + 0{\cdot}9790 \cdot \lambda$	1·0

Settlement of the tip of the pile

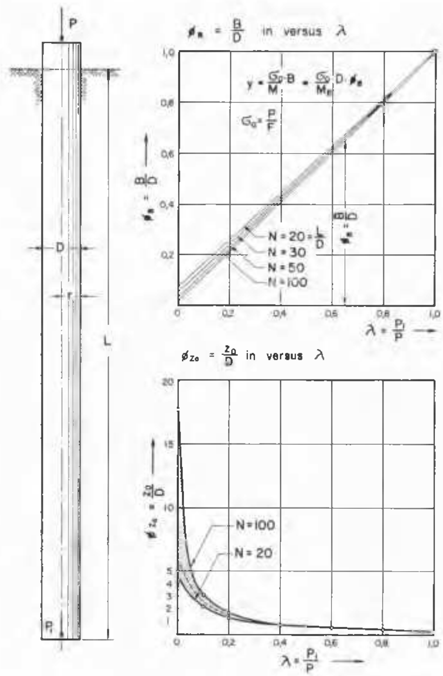

Fig. 5 Diagram for determining the settlement of cylindrical piles.
Diagramme servant à déterminer le tassement de pieux cylindriques.

4. *Application to the cylindrical pile.*

For the cast-in-place pile it is assumed that the pile terminates into a plane (Fig. 5).

(a) *Dimensions :*

L and D optional ; $N = \frac{L}{D}$; $\beta_1 = \frac{\pi}{2}$.

(b) *Calculation of the equivalent diameter* B (see Fig. 5) :

$$B = D \cdot \Phi c = D[C_2 + C_3 + \lambda \cdot (C_1 - C_2 - C_3)]$$

$C_1 = 1{\cdot}0$,

C_2 = is established in tabulated form or by planimetering,

$$C_3 = \frac{3}{4} \cdot \frac{r}{L} \cdot \log_e \left(1 + \frac{L}{z_2}\right) = \frac{3}{8} \cdot \frac{1}{N} \cdot \log_e \left(1 + \frac{L}{z_2}\right)$$

(c) *Settlement :*

$$y = \frac{\sigma_0}{E} \cdot B = \frac{\sigma_0}{E} \cdot D \cdot \Phi_c ; \; E = M_E$$

(d) *Modulus of compressibility :*

$$E = \frac{\sigma_0}{y} \cdot D \cdot \Phi_c = M_E$$

(e) *Reference stress :*

$$\sigma_{zm} = \left(\lambda + \frac{1 - \lambda}{2N}\right) \cdot \sigma_0 \text{ (see Fig. 5)} \qquad (9)$$

(f) *Equivalent effective height :*

$$z_0 = \frac{3}{8} \cdot \frac{D}{\Phi_e}.$$

5. *Fundamental remarks.*

The solutions given above are valid only for the special case of an infinite thickness of the layer $z_3 (z_3 = \infty)$*. According to Figs. 4 and 5, showing the required values of both the model pile and the cylindrical pile as a function of λ, follows :

The equivalent diameter (B), the settlement of the toe of the pile (y), and the reference stress (σ_{z_m}) increase in proportion with λ. The effect of λ upon these three values is the more pronounced the slimmer the pile; e.g. the higher the value N. It is evident from this that in the case of positive skin friction, it is always important to transfer as large a proportion of the load as possible by means of skin friction into the soil, in order to create only slight settlements and small concentrations of stresses.
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The assumption made under 2/1. (d) is not always entirely fulfilled, particularly in the case of model piles. In highly cohesive material, or with heavy overburden loads, part of the load P may radiate upwards, thereby relieving the lower half-space. In this case somewhat excessive E-values have been obtained, because the load on the lower half space is smaller than assumed in the normal calculation of σ_0. This disturbing effect increases with the skin friction.

3. Model pile tests for the determination of the modulus of compressibility E (M_E)

The model pile used for testing was provided above the toe with a cylindrical enlargement (friction cylinder) of length $3D$, which transfers the entire skin friction into the soil (Fig. 3). There are several methods available for eliminating the disturbance caused by the use of a driving rod, e.g. this can be achieved by the use of a casing which in this case is provided by the casing of the bored pile [4].

1. *Preparation of the tests :*

After the sinking of the casing (System Benoto, 88 cm diameter) to 23·3 m below foundation level (27 m below ground level), the model pile was driven 95 cm below the bottom of the bore hole, and (similar to the standard penetration test) the specific driving resistance w in kg per sq. cm measured (Fig. 3). The driving record (Fig. 7, left) shows that the subsoil has to be considered as disturbed to a depth of at least 50 cm below the bottom of the borehole. A soil sample (moraine) extracted in the proximity and examined by the VAWE had the following properties : No. 11 048 : grey, light clayey silt with sand and a small proportion of gravel (CL-ML).

Moisture content : 13·0 per cent. Density (saturated) = 2·23 tons per cub. m.

Consistency : L_L = 15·5 per cent; P_L = 11·1 per cent; P_I = 4·2 per cent.

Grain-size distribution : < 0·002 mm = 8·9 per cent; < 0·02 mm = 35·7 per cent; < 0·2 mm = 88·4 per cent.

Modulus of compressibility : Δ_c = 1·22 per cent; Δ_e = 0·25 per cent.

Permeability : k (tor σ = 4 kg. per sq. cm) $\sim 10^{-7}$ cm/sec.

* If this assumption is not fullfilled, the above theory can cesaly be modified, introducing a reduction factor (< 1) in equ. 2 for the partial settlement y_1.

2. *Execution of the tests.*

Immediately after the model pile had been driven the specific skin friction (boundary value) was established by torsion at 0·50 kg per sq. cm. After an interval of 1 ½ hours, this boundary value was increased to 0·74 kg per sq. cm. Under the assumption of a friction coefficient of 0·4 between steel and soil, a relief of the pore pressure in the contact zone of 0·6 kg per sq. cm would correspond to retention by suction of the model pile.

The model pile was subsequently loaded in stages by means of a hydraulic jacks (Figs. 6 & 7). Each increase of 250 kg

Fig. 6 Testing device of the model pile.
Disposition de mise en pression du pieu-modèle.

(5 kg per sq. cm) lasted 8 minutes. The exterior load was increased as high as P_{max} = 5·0 t, respectively $\sigma_{0\ max}$ = 100 kg per sq. cm without reaching the ultimate bearing capacity. Thereafter the extraction test was carried out, and the pile lifted about 20 mm, by increasing the pull in stages. The model pile was subsequently loaded once more for compression. At the beginning only skin friction did operate, because a hollow space had been created below the pile toe by lifting the penetrometer. This made it possible to measure the skin friction as a function of the displacement distance separately from the resistance of the toe of the pile. At the end of a series of tests lasting about 6 hours, the torsion test gave a final boundary value for skin friction of 0·77 kg per sq. cm.

3. *Qualitative results.*

The *load-settlement curve A-G*, represented in Fig. 7, shows a steady increase up to the final specified load of σ_0 = 100 kg per sq. cm. The remarkably steep position of the two hysteresis loops (shaded) which have been obtained by the two unloadings and reloadings, is caused by the relatively high part of the elastic deformation of the pile shaft.

The pull-lift diagram A-J shows a sharp angle at the true zero point (0) of the load (pull Z = dead weight of penetrometer). The load-settlement curve O-A-K-L was

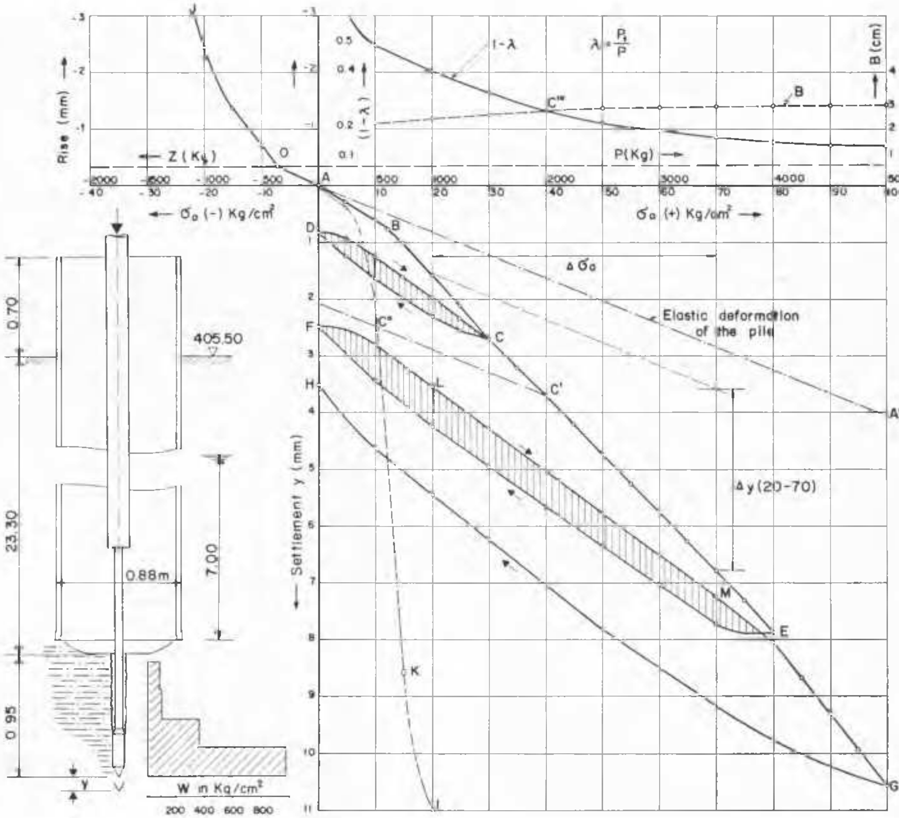

Fig. 7 Load-settlement and load-lifting diagrams of a model pile test.
Diagrammes des contraintes-tassements et des contraintes-soulèvements.

obtained by reloading. From the point K onwards the toe resistance is again becoming gradually evident in addition to skin friction.

The steep drop of the settlement curve A-K (without toe resistance) indicates that skin friction increases only slightly after exceeding a certain movement (about 2 mm). The percentage of the proportion $(1-\lambda)$ of the skin friction of the total load P therefore decreases with increased load [5].

4. *Quantitative evaluation.*

The proportion of the skin friction to total load is shown in Fig. 7 by the curve $(1-\lambda)$ as a function of P; the dead weight of the penetrometer has been neglected. The design of the curve is based upon the assumption, that with identical settlements of the pile tip (with and without point resistance), identical skin friction will be occuring. e.g. if the share of the skin friction has to be established for $P = 2\,000$ kg ($\sigma_0 = 40$ kg per sq. cm), a parallel line to A-A' (elastic deformation of the pile rod) is drawn through the point C', thus obtaining the point C'' with $P_2 = 520$ kg. Fromwhich follows : $(1-\lambda) = 520 : 2\,000 = 0{\cdot}26$ (point C'''). Fig. 7 shows that for an increase of the load from 500 kg to 5 000 kg (without dead weight of the pile) respectively of σ_0 from 10 to 100 kg per sq. cm, the share of skin friction $(1-\lambda)$ is decreasing from about 50 per cent about 14 per cent.

Knowing the value of λ as a function of P, the curve of the equivalent diameters B (cm) can be plotted (Fig. 7) by using Fig. 4. Thanks to the relatively small pile shaft area of the model pile, the variation of B is only small. An approximate determination of λ for the given load interval is sufficient.

The required E-values and the respective reference stresses are then obtained, based upon theory and Fig. 7 as follows :

Stage of load σ_0 *in* kg/cm²	$\Delta\sigma_0$ kg/cm²	Δy cm	λ —	B cm (equ.5)	M_E E (equ. 6a) kg/cm²	σ_{zm} (equ.7) kg/cm²
						$\sigma_{om} = 50$
Primary (B—E) 20-70	50	0·315	0·76	2·67	425	14
Secondary (L—M) 20-70	50	0·160	—	—	840	14

For the range A-B of the settlement curve the soil seems to be preconsolidated.

4. The boring diagram as a criterion for the required length of bored piles

1. *General principles.*

A boring log is a diagram of the ideal boring resistance $Bw(t)$ in the direction of the borehole as a function of the boring depth (m). The area of the diagram represents the total boring energy $A(mt)$ (Fig. 8). Inside the limited

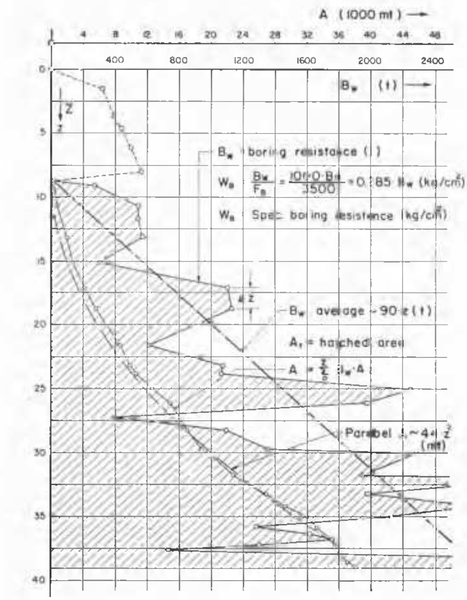

Fig. 8 Boring diagram of the test pile ∅ 67 cm.
Diagramme de perforation d'un pieu moulé dans le sol de ∅ 67 cm.

construction site of a given sequence of geological layers, but with variable thickness of these layers, the adjustment of pile lengths to the local soil conditions can take place as follows :

(a) For *point bearing piles*, where the point resistance is the most important part, while the skin friction may become negative in relatively soft layers above a solid stratum the position of the latter and the necessary pile length, can be established from the sudden increase of boring resistance. The area of the boring diagram which has been obtained during penetration into the solid layer gives a criterion for the required depth of penetration.

(b) In the case of *friction piles*, the skin friction, acting between the soil layer and the pile, is larger in proportion to the boring resistance. The area of the boring diagram (total boring energy A_t) therefore gives a relative value for total skin friction (sum of the skin friction of all layers penetrated). If now, with a test pile of an active length L_0 (depth of penetration), of which the test has given a certain bearing capacity P_0, and for which a total boring energy A_0 has been measured, we obtain a certain proportion $X_0(m)$ between A_0 and P_0, which is regarded as criterion for the required pile length (depth of penetration) :

$$X_0 = \frac{A_0}{P_0} \text{ in } mt/m = m \qquad (10)$$

For an adjacent bored pile the safe load of which should be P_{zul}, the total boring energy A_{erf}, as measured from the boring diagram, must fulfill the following condition :

$$A_{erf} : A_0 = P_{zul} : P_0$$
$$A_{erf} = \frac{P_{zul}}{P_0} \cdot A_0 = P_{zul} \cdot X_0 \qquad (11)$$

The bored pile must then be bored to such a depth that the condition (11) has been fulfilled, independent of its diameter. Uneven settlements which depend on the heterogeneity of the subsoil, can thus be reduced. Because X_0 represents the necessary boring energy in *mt* for a permissible load of one ton, the subsoil conditions of a certain construction site are thus the more favourable, the smaller X_0, the validity of which is locally limited.

2. *Determination of the boring diagram with the System "Benoto".*

With the Benoto pile, the partial boring energy ΔA for a sinking of the casing over a distance Δz can be computed as follows (Fig. 8) :

$$A = n \cdot P \cdot s + G \cdot \Delta z \qquad \ldots\ldots (12)$$

n = number of stokes during the turning of the pipe,
P = measured mean value of the piston pressure,
s = path of the piston per stroke,
G = dead weight of the equipment which is acting upon the boring bit.

For the specific boring energy B_w we obtain :

$$B_w = \frac{A}{\Delta z} = \frac{n \cdot P \cdot s}{\Delta z} + [G] \qquad \ldots\ldots (13)$$

As G is subject only to small variation, for comparison purposes, it can be neglected. For the total boring energy we obtain :

$$A = \sum_0^z \Delta A = \sum_0^z Bw \cdot \Delta z \qquad (14)$$

(area of the boring diagram).

Fig. 8 shows the boring diagram belonging to test pile (∅ 67 cm) as described below. The value X_0 amounted hereby to about 100 metre tons per ton of permissible load.

5. Results of a test pile

Fig. 9 shows the load-settlement diagram of a test pile of Benoto type having a diameter of 67 cm and driven to a depth of 30·2 m, compared with the calculated deformation of the pile (boring diagram, see Fig. 8). For calculating the deformation of the pile, the following deformation moduli, measured by *EMPA* on concrete samples of the same curing time, have been employed :

σ = 0-50 kg per sq. cm,
V_B = 412 000 kg per sq. cm,
σ = 50-100 kg per sq. cm,
V_B = 312 000 kg per sq. cm.

The contractions of the pile, as calculated for the various values of λ ($\lambda = P_1 : P$) are shown in Fig. 9 as a function of the pile load P. The comparison between the calculated deformations of the pile and the measured settlements leads to the following conclusions :

1. For $P \leqq 100$ tons the measured settlements of the pile head are smaller than the calculated contractions of the pile for $\lambda = 0$ (friction pile only). From this it follows that small

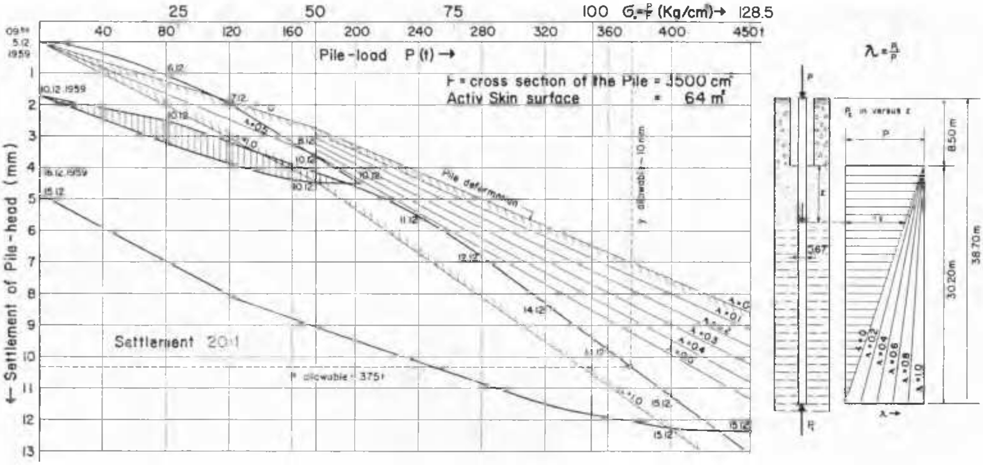

Fig. 9 Load-settlement diagram of the test pile ∅ 67 cm.
Diagramme des contraintes-tassements du pieu d'essai ∅ 67 cm.

loads P are already compensated by the skin friction in the upper parts of the pile, so that the lower part remains practically unstressed.

2. For higher values of P the determining E-values of the subsoil of the pile can be calculated from the difference between the measured settlement and the calculated contraction of the pile. The skin friction is supposed to be evenly distributed along the total active length of the pile (Chap. II) For $P = 360$ tons ($\sigma_0 = 103$ kg per sq. cm) we obtain the following values for the described test pile :

$$\lambda \quad = 0 \quad 0{\cdot}1 \quad 0{\cdot}2 \quad 0{\cdot}3 \qquad \lambda = \frac{P_1}{P},$$

$$M_E = E = 840 \quad 3\,150 \quad 6\,730 \quad 10\,300 \text{ kg per sq. cm.}$$

From other tests it may be concluded that the value E of the subsoil (preconsolidated, lean, morainic clay) will not be higher than 3 000 kg per sq. cm and $\lambda \leqq 0{\cdot}1$ (friction pile). The bearing capacity of the tip of the pile has to be regarded as supplementary reserve. It will become effective only in case of bigger settlements and higher loads.

3. In case of smaller removals of the pile load, the pile head will show very small or even hardly any rises because considerable negative skin friction is created. At the final removal of load from the test pile which had been loaded to $P_{max} = 450$ tons this negative skin friction became apparent during almost the entire unloading procedure.

6. Conclusions

1. The approximate determination of the moduli E or M_E can be done by means of a model pile even if the soil has been preconsolidated and the measuring point is submerged in groundwater.

2. The larger the share of the friction of the total load, the smaller will be the settlement. The well known downward curvature of the load-settlement diagram of the test pile can at least partly be explained by the fact that with the increase of the pile load, the proportion of skin friction becomes smaller.

3. Even in the case of relatively good bearing capacity of the pile toe, due to the settlement, those piling systems are the best which have the highest possible proportion of skin friction. In the case of bored piles the skin friction can be increased by increasing the horizontal pile shaft pressure during concreting.

On the other hand, in the case of negative skin friction, piling systems with small skin friction are to be preferred.

4. The use of the total boring energy as a criterion for the bearing capacity of friction piles makes it possible to adjust the pile lengths according to locally changing soil conditions. The method is conclusive only in those cases where the relationship between the boring energy and the permissible load of the pile has been previously established by means of a test pile.

5. The model pile test can be used to estimate ultimate bearing capacity, similar to the Dutch cone penetration test [6 and 7]. On the other hand, tests of long duration enable the secondary time effect to be investigated, which should be considered in all settlement calculations [8].

References

[1] Fröhlich, O. K. (1944). Druckverteilung im Baugrund.

[2] Haefeli, R. (1944). Theorie zur Zetzungsanalyse bei konstantem M_E-Wert. *Interner Bericht*, No. 89 der Versuchsanstalt für Wasserbau an der E.T.H.

[3] — (1960). Theoretische Grundlagen zur Setzungsberechnung von zylindrischen Pfählen. *Interner Bericht.*

[4] — and Fehlmann, H. B. (1958). Measurement of Soil Compressibility in situ by means of model pile test. *Proceedings of the 4th International Conference on Soil Mechanics*, London, Vol. I, pp. 225-229.

[5] Kezdi, A. (1957). Bearing Capacity of Piles and Pilegroups. *Proceedings of the Fourth Int. Conf. on Soil Mechanics*, London, Vol. II, pp. 46-51.

[6] Geuze, E. C. W. A. (1952). Résultats d'essais de pénétration en profondeur et de mise en charge de pieux-modèle. *Ann. Inst. Batim. Serie.* Sols et Fondations, pp. 313-319.

[7] Van Der Veen, C. (1957). The bearing capacity of a pile pre-determined by a cone penetration test. *Proceedings of the Fourth Int. Conf. on Soil Mecanics*, London, Vol. II, pp. 72-75.

[8] Bierrum, L., Jonson, W. and Ostenfeld, C. (1957). The settlement of a bridge abutment on friction Piles. *Proceedings of the Fourth Int. Conf. on Soil Mechanics*, London, Vol. II, pp. 14-18.