STATENS GEOTEKNISKA INSTITUT

SWEDISH GEOTECHNICAL INSTITUTE

No. 35

SÄRTRYCK OCH PRELIMINÄRA RAPPORTER

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Supplement to the "Proceedings" and "Meddelanden" of the Institute

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1. New Pile Force Gauge for Accurate Measurements of Pile Behavior during and Following Driving

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2. Methods of Calculating the Ultimate Bearing Capacity of Piles. A Summary

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NEW PILE FORCE GAUGE FOR ACCURATE MEASUREMENTS OF PILE BEHAVIOR DURING AND FOLLOWING DRIVING

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NEW PILE FORCE GAUGE FOR ACCURATE MEASUREMENTS OF PILE BEHAVIOR DURING AND FOLLOWING DRIVING

BENGT H. FELLENIUS *Swedish Geotechnical Institute, Banergaten 16, 115-26 Stockholm, Sweden*
THOMAS HAAGEN *The Axel Johnson Institute for Industrial Research, Sweden*

The paper reports a new pile-force gauge based upon the principle of the vibrating wire. The gauge is intended to be driven down with a precast concrete pile and can be placed at an arbitrarily chosen depth in the pile. The impacts from the pile driving will not impair the gauge. The gauge registers the static loads and bending moments in a pile with an error not exceeding 2% of the linear measnring range. This maximum error includes the drifting of zero point and change of sensitivity with time.

The design of the gauge and laboratory and full-scale tests are reported, and suitable use of the gauge is suggested.

Cet article présente une nouvelle jauge de mesure des efforts appliqués aux pieux; cette jauge est basée sur le principe des mouvements vibratoires de fils métalliques. La jauge est conçue pour être battue avec un pieu en béton préfabriqué et peut être placée à une profondeur arbitraire choisie dans le pieu. Les impacts du battage n'endommagent pas la jauge. La jauge mesure les charges statiques et les moments fléchissants dans un pieu avec une erreur ne dépassant pas 2% de l'étendue linéaire de l'échelle. Cette errenr maximum inclut la déviation du point zéro et les variations de sensibilité avec le temps.

Une description de la jauge est présentée; un compte rendu d'essais en laboratoire et en chantier est également donné; les adaptations possibles de la jauge sont suggérées.

INTRODUCTION

In 1965, A. Johnson & Co. (Canada) Ltd., Montreal, consulted with the Axel Johnson Institute for Industrial Research (AJFO) in Sweden and asked for guidance on problems concerning negative skin friction on piles. AJFO in turn, asked the Swedish Geotechnical Institute for assistance. The three organizations set up a program to carry out full-scale pile tests in the field. It was soon realized that a condition for successfully resolving the problem was to have an accurate force-measuring device. Since this type of measuring equipment did not exist, it had to be developed. This work was undertaken by AJFO, who, after two years of extensive designing and testing, developed a pile-force gauge that satisfied all of the requirements. The actual field test started in June, 1968. The work is being carried out in close cooperation with the Pile Commission of the Royal Swedish Academy of Engineering Sciences. The cost is partly covered by a grant from the Swedish Council for Building Research. This article deals with the design, testing and application of the gauge.

DESIGN AND PRINCIPLE OF THE "PILE-FORCE GAUGE"

In Sweden the measuring of forces in piles was previously based upon the use of the electric strain gauge (IVA Pile Commission, 1964). Outside of Sweden, a system of rods has been used (Bjerrum and Johannessen 1965; Bozozuk and Jarrett 1968). These methods have several disadvantages. The electric strain gauge has an unsatisfactory accuracy due mainly to zero drift, whereas the rods must be installed after the pile driving, and therefore any influence during the pile installation is lost. The accuracy of both systems is limited, because one is not measuring forces, but deformations, which then are transferred into forces by using the modulus of elasticity of the pile material. As mentioned above, it was therefore necessary to develop a special pile-force gauge that would satisfy the following conditions:

(1) The gauge shall, during a long period of time (5–10 years), measure loads up to 150 tons,[1] with a maximum error of 2%.

(2) The gauge shall be able to withstand loads up to 400 tons, without damage.

(3) The gauge shall measure tension loads up to 50 tons.

(4) The gauge shall be able to withstand all stresses during the driving of the pile, i.e. withstand 10 000 blows with impact forces of the order of 150 tons.

(5) The gauge shall measure bending moments in the pile.

(6) It shall be possible to place the gauge at any depth in a pile, and have it function at a surrounding pressure, equivalent to a height of water of 300 ft (91 m).

(7) The gauge shall be adaptable to different types of piles, and adjustable to variable measuring ranges.

Different principles of measuring systems were studied. Two different prototypes were manufactured, one of which was based on the use of load cells with vibrating wires. This design was judged to be the more favorable one.

Measurements with vibrating wire are based upon the principle that a wire under tension will change vibrating frequency with changes in tension. A shortening or lengthening of a steel cylinder, for example, can then be recorded as a change in frequency of a vibrating wire inside the cylinder. The reading of the frequency is transmitted through a magnet, which first activates the wire, when the magnet is subjected to an electrical impulse. The resulting vibrations of the wire induce an alternating current in the magnet. Tbe frequency of the induced current is recorded by an electronic counter.

[1]All "tons" mentioned in this note are metric tons.

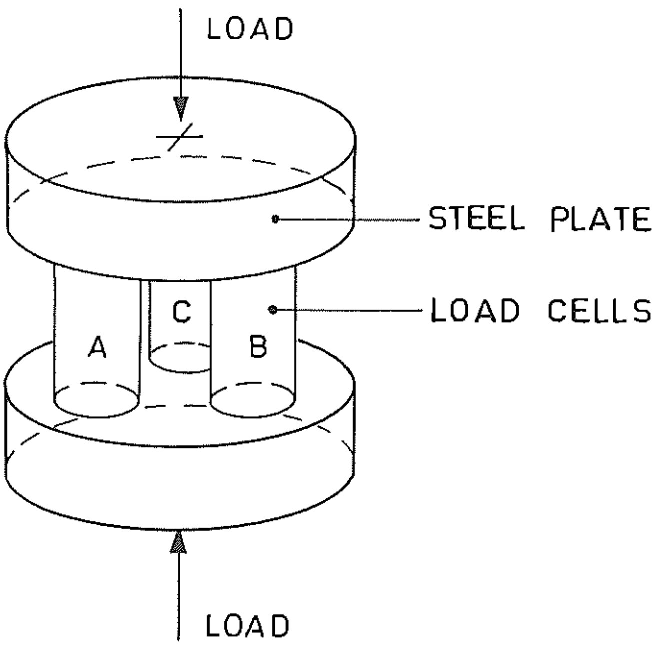

FIG. 1. Basic design of the pile-force gauge.

The mathematical expression for the lowest frequency, as a function of the tension of the wire, is

$$f = \frac{1}{2L}\sqrt{\frac{\sigma \cdot g}{\nu}} \qquad [1]$$

where f = the frequency (Hz)
L = the length of the wire (m)
g = 9.806
ν = the density of the wire (kg/m^3)
σ = the tension of the wire (kg/m^2)

The wire is placed in a load cell (steel cylinder) and pretensioned. When the load cell is under zero load the frequency is f_0.

If a load P is applied, the frequency is changed to f_1. The mathematical expression for P, as a function of frequency, is

$$P = \text{const.}\ (f_0^2 - f_1^2) \qquad [2]$$

The value of the constant is established through calculations and calibrations.

This principle is used extensively, and has been known for a long time. However, the special feature of the present design is a 'clamping in' of the wire ends, which is not impaired by impact and dynamic loadings, as encountered in a pile-driving operation.

DESIGN OF THE PILE-FORCE GAUGE

The basic design is shown in Fig. 1. Three load cells A, B, and C are placed symmetrically around the center of the gauge, and in between two steel plates.

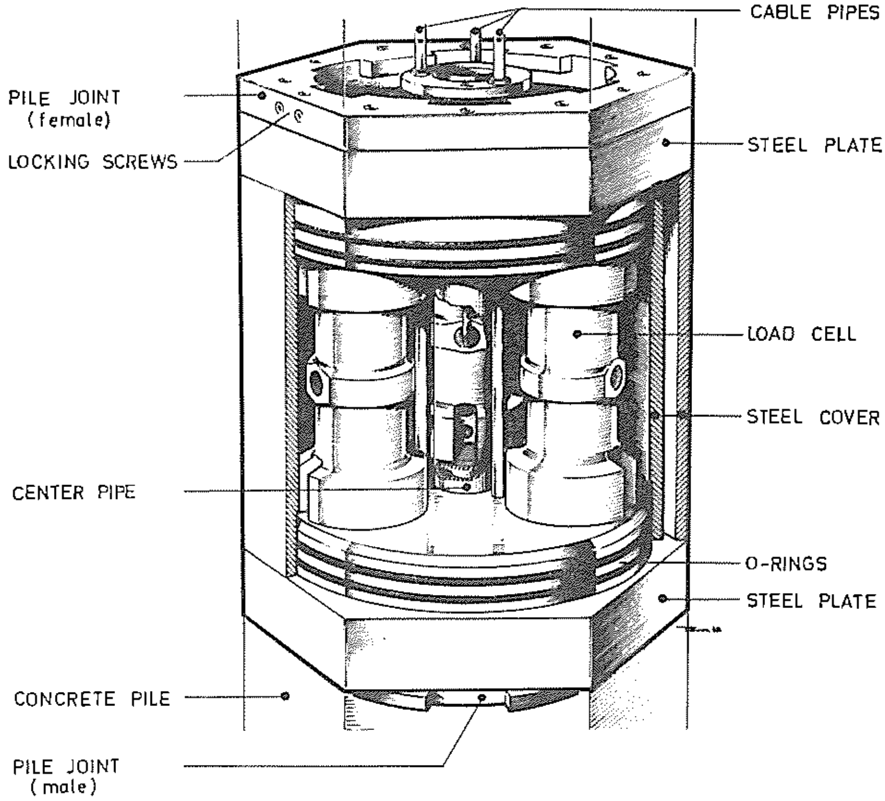

FIG. 2. Cross section of the pile-force gauge.

Pile joints are then mounted onto the plates, and a cover is placed around the gauge. The cross-sectional area and form of the gauge is adjusted to the size and shape of the pile in which the gauge is to be used. The gauge will be placed between two pile joints, and acts in fact as a short pile section.

The load cells in the pile-force gauge are shown in the 'cross section' (Fig. 2). The electric cables from the load cells are assembled through cable pipes, which are cast into the pile. Three cable pipes can be seen in Fig. 2, two of which are coming from other gauges in the pile. The gauge is placed in a pile equipped with a center pipe, which is used for measuring deformations by means of special rods. This center pipe, along with the thick steel plates above and below the load cells, are also shown in Fig. 2. In this case, the pile is a Herkules precast concrete pile of hexagonal cross section.

Figure 3 shows a gauge mounted on a pile equipped with a rock point. This pile, consisting of five sections, was driven down to a total depth of 180 ft (54.9 m) with a 4-ton (metric) drop hammer falling 20 inches (50.8 cm). The total number of blows required for the installation was 6000. After the driving was completed, the gauge was operating perfectly.

EVALUATION OF RECORDED VALUES

The pile-force gauge gives three frequencies, fa, fb, and fc, from which the load P, the bending moment M, and the direction β of the bending moment are

FIG. 3. Pile-force gauge mounted on a pile tip and equipped with a rock point.

evaluated. First, the load in each load cell is calculated from the recorded frequences [eq. 2]. The formula for the load P, as a function of the load in the separate load cells, is then

[3] $$P = P_A + P_B + P_C$$

Then, the moment vectors M_x and M_y at the center are

[4] $$M_y = (P_A + P_C)R \cos 60° - P_B R$$

[5] $$M_x = (P_A - P_C)R \overset{\sin}{\cancel{\cos}}\, 60°$$

and the resulting moment M is

[6] $$M = \sqrt{M_x^2 + M_y^2}$$

Finally, the direction β of the moment is

[7] $$\tan \beta = M_y/M_x$$

The evaluation can be done by hand, but time and money are saved by using a data computer.

CALIBRATION AND ERRORS

Every load cell is first calibrated in a 100-ton hydraulic press. Then the cells are submitted to 20 000 pulsations between 0 and 100 tons, to eliminate tendencies to changes in the system. Following this step a new calibration is carried out. Finally, when the three load cells have been assembled in place, the gauge is calibrated for axial load and bending moment, in a 200-ton hydraulic press.

The accuracy of the readings have been carefully studied, and special interest given to the long-term stability. The annual zero drift has been established at less than 0.8% of the upper limit of the linear range, with the total error being less than 2%.

During the design procedure, a full-scale pile-driving test was performed. A prototype of the gauge was placed on a pile. The pile was driven down through 60 ft (18.3 m) of loose soil to rock, with a 3.5-ton drop hammer falling

20 inches (50.8 cm). After the pile tip was driven into the rock, 10 000 additional blows were given to the pile and the gauge. As a result of this hammering, the investigation that followed showed a zero drift of 0.5% of the upper limit of the linear range, which was 150 tons. The linearity and sensitivity were unchanged.

FIELD TESTS

Seven pile-force gauges are being used in the full-scale test at Gothenburg, Sweden. The gauges have been placed at different depths in two piles driven to a depth of 180 ft (54.9 m). One gauge is right at the tip of the pile (**Fig.** 3). During the driving, the piles received 6000 blows with a 4.2-ton drop hammer falling 20 inches (50.8 cm). All gauges are operating as planned. The results from this test are being reported by Fellenius and Broms (1969).

The pile-force gauge has clearly demonstrated that it has satisfied beyond expectations all of the conditions originally specified for an accurate force-measuring device.

SUGGESTED USE OF THE GAUGE

A problem encountered during every load test, and worthy of further study, is the actual distribution of skin and tip resistance. For instance, all of the load on an end-bearing pile driven through clay will finally reach the tip of the pile. During a load test, however, a substantial part of the load is taken by skin resistance, and therefore the load–deformation relationship for the pile tip is not known. A gauge placed at the tip of the pile would solve this problem.

A study of the group action behavior of piles could be carried out and results obtained by placing gauges at suitably chosen depths in a few piles in the group. Then the effect on a pile when driving an adjacent pile in the group, the resulting forces in piles upon completion of the driving, the time effects of backfill, negative skin friction, horizontal movements in the soil around the piles, and other related problems could be studied.

Further applications of the pile-force gauge in full-scale tests worthy of special studies include (1) the buckling of long piles in soft clay under long-term testing, (2) the evaluations of bending moments due to lateral forces against piles, etc.

There is an unlimited opportunity to improve pile-load test results and establish pile capacities, with the pile-force gauge, due to its exceptional accuracy, time stability, and ability to separate axial load from bending moment.

ACKNOWLEDGMENT

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Methods of Calculating the Ultimate Bearing Capacity of Piles a Summary

by Bengt B. Broms*

Résumé Français page 32. Deutscher Abriss Seite 32. Resumen Español pàgina 32.

Summary

Methods for the evaluation of the bearing capacity of piles from strength properties of the soil or from dynamic and static penetration test are presented. It can be stated that it does not exist today a reliable general method to determine satisfactorily under all conditions the bearing capacity of piles. With the methods included herein the pile bearing capacity can be determined for some ideal conditions. In most cases considerable uncertainty exists about the actual bearing capacity of a pile when load tests have not been carried out.

Introduction

Investigations are carried out at present (1965) in Sweden by the Swedish Committee on Pile Research in cooperation with the Swedish Geotechnical Institute to improve presently available methods of calculating the bearing capacity of piles. This article represents an attempt to summarise presently available methods.

The ultimate bearing capacity of a pile is limited by either the compressive strength of the pile material or the bearing capacity of the surrounding soil (i.e. failure occurs when the load bearing capacity of the pile itself or of the surrounding soil is exceeded).

The bearing capacity of piles can be calculated from:

a) measured or estimated shear strength of the soil surrounding the pile;

b) static penetration test methods where the penetration resistance of a probe, which is slowly pushed into the soil, is measured;

c) dynamic penetration tests where, for instance, the number of blows required to drive a standard sampler or a conical point a certain distance into a soil is measured;

d) pile driving formulas, which are based on the number of blows required to drive a pile a certain distance, or

e) load tests

The calculation of the ultimate bearing capacity according to the methods *a*), *b*) and *c*) is discussed in this article as well as the validity and the accuracy of these methods.

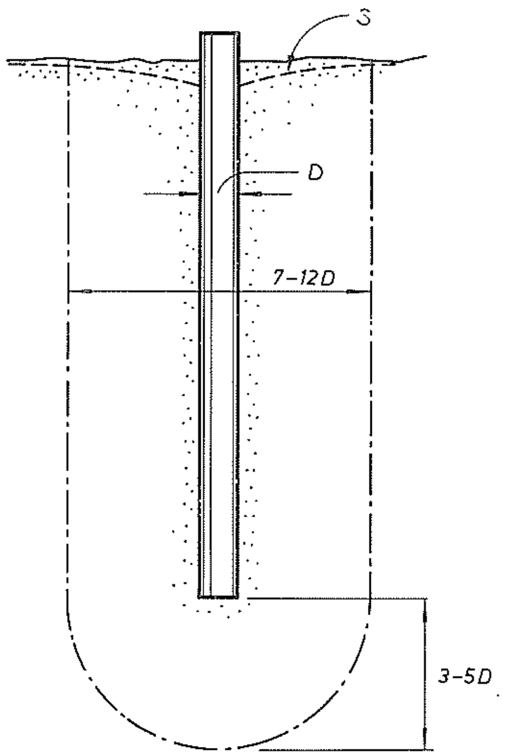

Fig. 1

Compaction of Cohesionless Soils During Driving of Piles
Compactage des sols pulvérulents sous l'effet du battage de pieux
Compactación de suelos sin cohesión bajo el efecto del hincamiento de pilotes
Verfestigung von nichtbindigen Bodenarten als Folge des Einrammens von Pfählen

D pile diameter
diamètre du pieu
diámetro del pilote
Pfahldurchmesser

S settlement
tassement
asentamiento
Setzung

*Swedish Geotechnical Institute, Stockholm, Sweden.

Change of soil properties during pile driving

The bearing capacity of piles driven into cohesionless soils depends primarily on the relative density of the soil. During driving the relative density is increased close to the pile due to vibrations. The relative density is increased within an area with a diameter of 7 to 12 pile diameters and to a depth of 3 to 5 pile diameters below the pile point as shown in fig. 1.

The increase in relative density which is caused by driving has been investigated by Plantema & Nolet (1957) using the Dutch cone penetrometer. This penetrometer consists in principle of a cone-shaped probe with a cross-sectional area of 10 cm² which is slowly pushed into the soil. Plantema and Nolet measured the changes in penetration resistance which occurred during and after the driving of a concrete pile through sand by inserting the penetrometer through a tube which was cast into the pile. The measured penetration resistance was, close to the pile tip, four times the penetration resistance measured before driving of the pile. At a distance of about three pile diameters below the pile point the measured penetration resistance was about 1.5 times the penetration resistance of the undisturbed material. At a distance of about five pile diameters below the pile point no change in penetration resistance was observed. Similar observations have been made by Meyerhof (1959), Széchy (1960), Kézdi (1960), Weele (1961), Nishida (1961), Kerisel (1961), Robinsky & Morrison (1964) and Weele (1964).

Investigations have also shown that the increase in relative density caused by pile driving is larger for loose than for dense sand, and that the zone of influence where an increase of relative density occurs is larger for loose than for dense sand. This increase in relative density affects the bearing capacity of single piles and of pile groups. It can therefore be expected that the bearing capacity of piles driven into cohesionless soils will be higher than the bearing capacity which corresponds to the relative density of the undisturbed soil. In addition it can be expected that the bearing capacity of piles placed in prebored holes or driven with the aid of jetting will be less than the bearing capacity of driven piles. This fact has been pointed out, among others, by de Beer (1964).

Cohesive soils are also disturbed by pile driving. Measurements have shown that the shear strength of the soil is affected by pile driving to a distance from the pile surface corresponding to one pile diameter and to a depth of one pile diameter below the pile point as shown in fig. 2. Measurements have furthermore shown that the shear strength close to the pile surface is decreased for a driven pile to a value which corresponds to the shear strength of the remolded material. However, the reduction in shear strength is small at a distance of one to two pile diameters from the pile surface (Cummings, Kerkhoff & Peck, 1948).

The skin friction resistance immediately after driving can be calculated as the product of the shear strength of the remolded clay and the surface area of the pile. However, the shear strength of the soil and hence the bearing capacity of the pile increase with time. Measurements have shown that one to six months after driving the bearing capacity of the pile corresponds to the shear strength of the clay before driving (i.e. the shear strength of the undisturbed material). Consequently it can be expected that the increase in bearing capacity will be dependent of the sensitivity of the clay and that the increase will be large for quick clays. This increase in bearing capacity occurs in general faster for wooden piles than for concrete piles due to differences in permeability of the two materials.

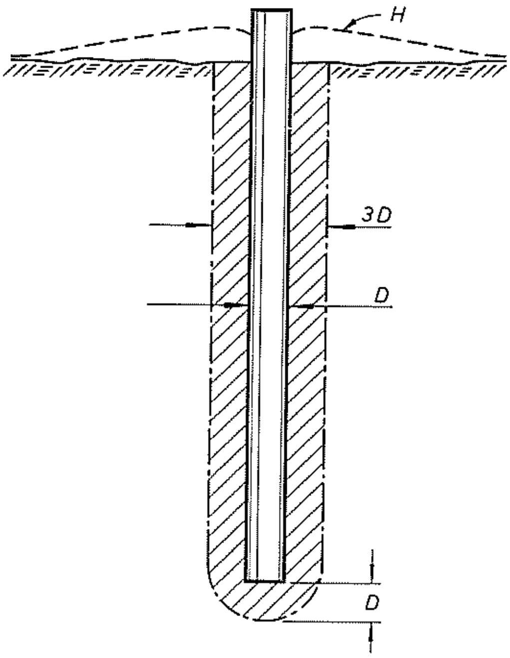

Fig. 2

Disturbance of Cohesive Soils During Driving of Piles
Remaniement des sols cohérents sous l'effet du battage de pieux
Suelos cohesivos alterados bajo el efecto del hincamiento de pilotes
Störung von bindigen Bodenarten unter der Einwirkung des Rammens von Pfählen

H heave / soulèvement / levantamiento / Heraushebung

(hatched) high pore water pressure zone / zone à forte pression intersticielle / zona con alta presión intersticial / zone mit hohem Poren wasserdruck

The roughness of the pile surface and the straightness of the pile also affect the skin friction resistance. Fellenius (1955) has shown that the skin friction resistance will be lower for a pile with a rough surface than for a pile with a smooth surface due to differences in remolding of the surrounding soil during driving.

Tests have also shown that the pore pressures increase considerably during driving and that in cohesive soil the pore pressure may in many cases approach the total overburden pressure (Bjerrum & Johannessen, 1960). These high pore pressures indicate that slope failures can occur when piles are driven through e.g. embankments.

Pile driving through cohesive soils causes heave around the piles. The heave decreases with increasing distance from the pile or pile group and is insignificant at a distance of 10 to 15 pile diameters from an individual pile. However, heave affects frequently the bearing capacity of surrounding point bearing piles since these piles can be lifted and thus may loose part of their point support. Heave may also cause separation of spliced piles if the tensile strength of the splices is low. Redriving of such piles may therefore be necessary.

Calculation of Bearing Capacity from Soil Data

Methods have been developed to calculate the bearing capacity of piles from the bearing capacity of the surrounding soil as illustrated in fig. 3. The ultimate capacity Q_{ult} of the pile shown in fig. 3 consists of skin friction Q_{skin} and point bearing Q_{point}.

Consequently:

$$Q_{ult} = Q_{skin} + Q_{point} \qquad (1)$$

For calculation purposes it is generally assumed that the skin friction resistance and the point resistance can be determined separately and that these two factors do not affect each other. Test results reported by Cambefort (1953), Kézdi (1957) and Stuart, Hanna and Naylor (1960) show however that the skin friction resistance affects the point resistance for piles which have been driven through cohesionless soils. However this influence is in most cases small and can be neglected. The point resistance is for a cohesive material independent of the intensity of the skin friction resistance.

Very small axial deformations are generally necessary to mobilize completely the skin friction resistance along a pile as observed, among others, by Müller (1939), Schenck (1951), Zweck (1953), D'Appolonia & Romualdi (1963), D'Appolobia & Hribar (1963) and Weele (1964). In contrast relatively large deformations are required to mobilize the maximum point resistance of piles which are driven into cohesionless soils. Therefore the largest part of the applied load is carried by skin friction at low applied loads while at high load levels the largest part is carried by point resistance (Mansur & Kaufman, 1958, Mohan, Jain & Kumar, 1963).

The following methods to calculate the bearing capacity of piles from soil data are limited to clays and sands and cannot as a rule be used to calculate the bearing capacity of piles driven through silts.

Cohesionless soils

Calculation of skin friction resistance Q_{skin}

The skin friction resistance of a pile driven through a cohesionless soil is first mobilized at loading close to the ground surface (Mogami & Kishida, 1961, D'Appolonia & Romualdi, 1963). The mobilization of skin friction spreads along the pile with increasing applied load and at failure the skin friction resistance is mobilized along the full length of the pile.

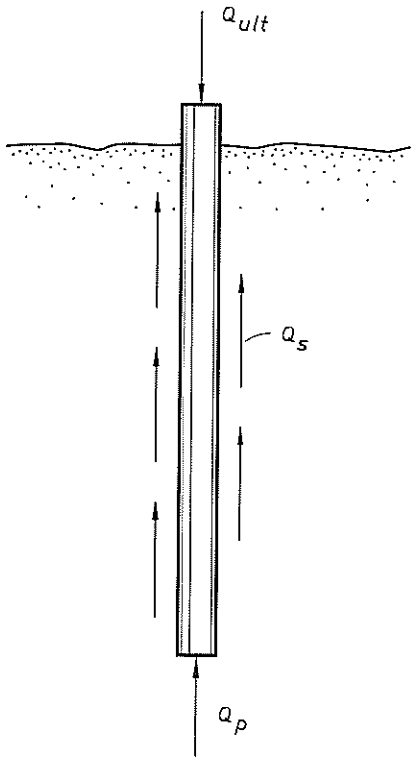

Fig. 3

Skin Friction Resistance Q_s and End Bearing Q_p
Frottement latéral Q_s et effort en pointe Q_p
Rozamiento lateral Q_s y esfuerzo en punta Q_p
Seitenreibung Q_s und Spitzenwiderstand Q_p

The skin friction resistance decreases approximately linearly with the depth below the ground surface for a pile driven in a cohesionless soil except for an area located close to the pile point. At this point the skin friction resistance is frequently lower than that which acts at three to four pile diameters above the pile point (Mohan, Jain & Kumar, 1963, Mansur & Kaufman, 1958). This deviation will be neglected in the following calculations.

The skin friction resistance of piles which are driven into a cohesionless soil can be calculated from the assumed distribution of lateral earth pressure along the pile. At the distance z below the ground surface (fig. 4) the vertical effective pressure $\bar{\sigma}_v$ is calculated from the following equation,

$$\bar{\sigma}_v = \bar{\gamma} z \qquad (2)$$

where $\bar{\gamma}$ is the submerged unit weight of the soil when the ground water table is located at the ground surface and is equal to the unit weight of the soil when the ground water surface is located below the depth z. The vertical effective pressure is thus assumed to increase linearly with depth.

The corresponding effective lateral pressure $\bar{\sigma}_h$ is:

$$\bar{\sigma}_h = K_o \, \bar{\gamma} z \qquad (3)$$

where the coefficient K_o is an earth pressure coefficient which is dependent of the volume per unit lenght of the driven piles and of the relative density of the surrounding soil. In the case when the volume per unit length of the

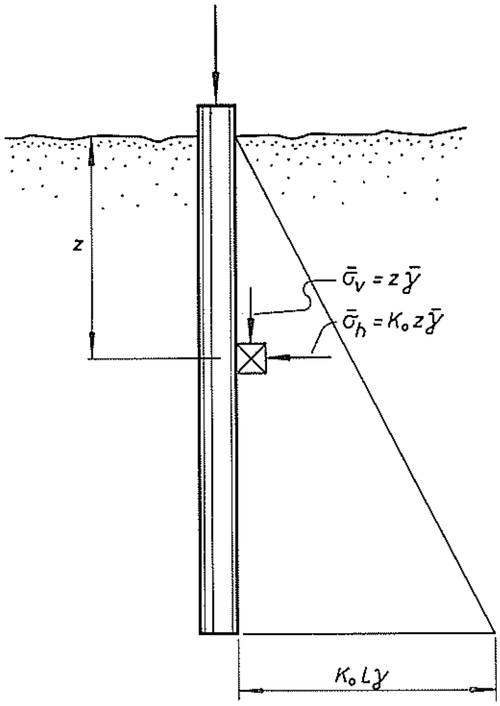

Fig. 4
Lateral Pressure Distribution for a Pile Driven in Cohesionless soil
Distribution des contraintes latérales pour un pieu battu dans un sol pulvérulent
Distribución de tensiones laterales en un pilote hincado en un suelo sin cohesión
Verteilung der seitlichen Spannungen bei einem Rammpfahl in nichtbindigem Boden

pile is small (e.g. a steel pile) the coefficient K_o approaches the lateral earth pressure at rest (Mansur & Kaufman, 1948, D'Appolonia & Romualdi, 1963). For a displacement pile (e.g. wood or concrete piles) the coefficient K_o can assume very high values as shown among others by Müller (1939), Ireland (1957), Feda (1963), Nordlund (1963) and Broms & Silberman (1964) especially for the case when the relative density of the surrounding soil and the roughness of the pile surface are high and the piles are tapered.

The skin friction resistance of relatively short straight-sided piles is often less than 30-40 % of the total pile bearing capacity (Skempton, Yassin & Gibson, 1953, Meyerhof, 1960) and therefore it is generally sufficient to estimate only the value of the end bearing resistance of such piles. However, the skin friction resistance of tapered piles can be considerably larger than the end bearing resistance (Robinsky, Sagar & Morrison, 1964 and D'Appolonia & Hribar, 1963) even for the case when the piles are driven through a cohesionless soil with low relative density. For such piles it is important to estimate accurately the skin friction resistance as well as for relatively long straight-sided piles. In a design method proposed by Meyerhof (1950) it has been assumed that the coefficient K_o can be taken as 0.5 for loose sand and 1.0 for dense sand. This coefficient is thus assumed to be independent of pile type, roughness of the pile surface and pile taper.

Analysis of test results have, however, indicated that the values given by Meyerhof (1951) result in a calculated skin friction resistance which is somewhat too low for concrete and wood piles. The following values are therefore recommended for the calculation of earth pressure coefficient K_o. Considerations have been taken to the volume per unit length for the different pile types.

Table I. Calculation of earth pressure coefficient K_o for cohesionless soils.

Pile types	Low relative density	High relative density
Steel piles	0.5	1.0
Concrete piles	1.0	2.0
Wood piles	1.5	4.0

The recommended values of the coefficient K_o correspond for steel piles to the lateral earth pressure at rest as the volume of, for example, H-piles is small. The roughness of a concrete surface and the relative large volume per unit length of concrete piles have been taken into account for concrete piles. In addition pile taper has been considered for wood piles (Peck, 1958, Nordlund, 1963, Robinsky, Sagar & Morrison, 1964).

The skin friction resistance can then be determined from the following equation:

$$Q_{skin} = \tfrac{1}{2}\ K_o\, L\ \gamma\ \tan \varnothing_a\ A_{skin} \tag{4}$$

In this equation A_{skin} is the skin area for the pile (πDL for a circular pile and 4 DL for a pile with a square cross section) and $\tan \varnothing_a$ is the coefficient of friction for the pile surface. This equation was first suggested by Dörr (1922).

The friction coefficient $\tan \varnothing_a$ has been determined experimentally, among others, by Potyondy (1961) and by Broms & Silberman (1964). The value of $\varnothing_a$ was 23°-25° for a polished steel surface and a fine to medium coarse sand. These values were not influenced by the relative density of the surrounding soil. Potyondy (1961) measured for a smooth concrete surface a value of the angle $\varnothing_a$ which was 4°-5° lower than the angle of internal friction of the soil. The friction angle $\varnothing_a$ for wood surfaces varied with the direction of the shear force with respect to the fiber direction. The friction angle $\varnothing_a$ was 4-10° lower than the angle of internal friction of the soil when the direction of the shear force coincided with the fiber direction. On the basis of these test results the following values are recommended for the evaluation of the angle $\varnothing_a$.

Table II. Computation of the friction angle $\varnothing_a$ on the basis of the angle of internal friction $\varnothing'$ of the surrounding cohesionless soil.

Pile types	$\varnothing_a$
Steel piles	20°
Concrete piles	3/4 $\varnothing'$
Wood piles	2/3 $\varnothing'$

If these recommended values are used the calculated skin friction resistance will probably be somewhat lower than the actual skin friction resistance. Thus these recommended values will yield results which are probably on the safe side. However, great caution should be excercised when this method is used in design.

Calculation of point resistance Q_{point}

The end bearing resistance can be calculated from the following general equation (Terzaghi, 1943):

$$q_{point} = \frac{Q_{point}}{A_{point}} = K_c\, c\, N_c + K_\gamma\, \bar{\gamma}\, D N_\gamma + K_q\, \bar{\gamma}\, L\, N_q \qquad (5)$$

In this equation the factors K_c, K_γ and K_q are shape factors which depend on the shape of the foundation. The factors N_c, N_γ and N_q are bearing capacity factors which are dependent of the angle of internal friction of the surrounding soil, D is the side or diameter of the support and L is the distance from the ground surface. The shape factors K_c, K_γ and K_q have been determined from laboratory investigations by Meyerhof (1951), Feda (1961), Hansen (1961) and by L'Herminier et al (1961). The test results show that the coefficients K_c, K_γ and K_q are equal to 1.3, 0.6 and 1.0 respectively for a circular foundation. As the cohesion *c* is equal to zero for a cohesionless soil, Eq. (5) can be simplified to :

$$q_{point} = 0.6\, \bar{\gamma}\, DN_\gamma + \bar{\gamma} L N_q \qquad (6)$$

The bearing capacity factors N_γ and N_q in this equation are of the same order of magnitude. The length L, the distance below the ground surface, is large for a pile in comparison with its diameter or side. Thus the first term on the right hand side of this equation is small and can generally be neglected. Eq. (6) can therefore be rewritten as:

$$q_{point} = \bar{\gamma}\, L\, N_q \qquad (7)$$

The shear strength is for a cohesionless material proportional to the effective confining pressure. The term $\bar{\gamma}L$ is thus the effective overburden pressure which acts at the level of the pile point, and the unit weight $\bar{\gamma}$ is equal to the submerged weight when the ground water surface is located at the ground surface and is equal to the unit weight of the soil when the ground water table is located below the pile point. The soil supporting the pile will not only carry the applied load but also the weight of the pile itself. The resulting net point bearing capacity q_{point} (the useful load carried by the pile) will thus be:

$$q_{point} = \gamma L N_q - \gamma_{pile} L \qquad (8)$$

where γ_{pile} is the unit weight of the pile material. The term $\bar{\gamma} L N_q$ is large in comparison to the term γ_{pile} L. If one assumes as an approximation that the unit weight of the soil is equal to the unit weight of the pile material Eq. (8) can be rewritten as:

$$q_{point} = \bar{\gamma} L \ (N_q - 1) \qquad (9)$$

The bearing capacity factor N_q can be computed with the aid of the theory of plasticity. The value of this factor varies with assumed failure surface. The failure surface used e.g. by Meyerhof (1951) for calculation

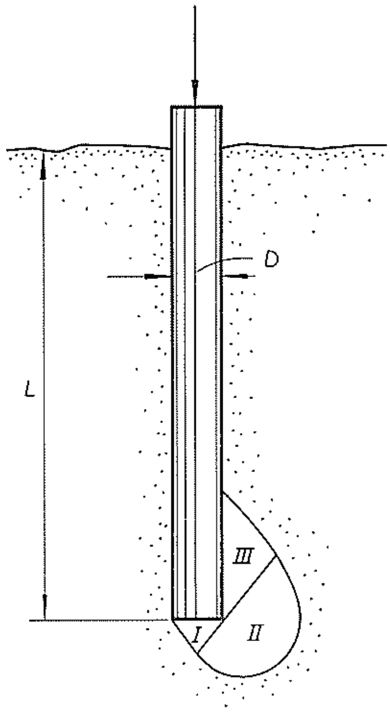

Fig. 5

Rupture Zones for a Pile Driven in Cohesionless Soil
Zones en équilibre limite pour un pieu battu dans un sol pulvérulent
Zonas de rotura en un pilote hincado en un suelo sin cohesión
Bruchzonen bei einem Rammpfahl in nichtbindigem Boden

I Active Rankine zone
Zone en poussée de Rankine
Zona activa de Rankine
Zone mit aktivem Erddruck (nach Rankine)

II Prandtl Zone
Zone plastifiée de Prandtl
Zona de Prandtl
Plastische Zone (nach Prandtl)

III Passive Rankine zone
Zone en butée de Rankine
Zona pasiva de Rankine
Zone passiven Erddruckes (nach Rankine)

purposes is shown in fig. 5. Meyerhof thus assumes that just below the pile point a wedge-shaped zone (marked "I" in fig. 5) is formed. This is the active Rankine zone, which at failure moves together with the piles. This triangular shaped zone displaces a spiral-shaped zone, the Prandtl zone, (marked "II" in fig. 5). This zone in its turn displaces an additional wedge-shaped zone, the passive Rankine zone (marked "III" in fig. 5).

In fig. 6 is shown the calculated values of the bearing capacity factor N_q as a function of the angle of internal friction. It can be seen from this figure that the bearing capacity factor N_q increases rapidly with increasing value of the angle of internal friction $\varnothing$. The bearing capacity factor N_q is equal to 50 when this angle is equal to 30° and equal to 450 when the bearing capacity $\varnothing$ is 40°. However field and laboratory test have shown that the values of the bearing capacity factor N_q calculated by Meyerhof overestimate the bearing capacity.

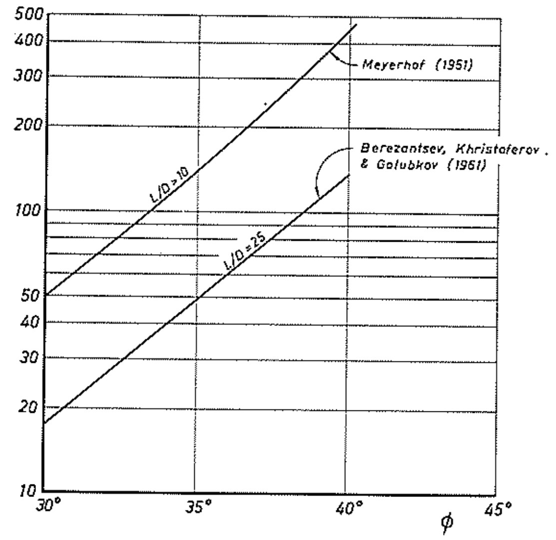

Fig. 6

Relationship Between Bearing Capacity Factor N_q and the Angle of Internal Friction ∅
Relation entre le facteur de force portante N_q et l'angle de frottement interne ∅
Relación entre el factor de capacidad portante N_q y el ángulo de rozamiento interno ∅
Beziehungen zwischen Faktor der Tragkraft N_q und Winkel der inneren Reibung ∅

Comparisons with test data (Nordlund, 1963) have shown that the values of the coefficient N_q suggested by Berezanth sev, Khristoforov & Golubkov (1961) agree better with measured values. Therefore the relationship suggested by Berezantsev, Khristoforov & Golubkov is recommended for the calculation of the point bearing capacity of piles.

In many cases it is difficult to use Eq. (9) to calculate the bearing capacity of piles because this equation requires an accurate estimate of the angle of internal friction ∅ and it is difficult to evaluate this angle for field conditions.

The friction angle ∅ can in general be determined from drained triaxial or direct shear tests. By this method the unit weight or the porosity of the undisturbed material is first determined from samples obtained with a thin walled piston sampler. Thereafter several series of drained triaxial or direct shear test are carried out at different void ratios. Thus the friction angle ∅' can be determined as a function of the void ratio of the soil. It is then possible to calculate the angle ∅' from the void ratio of the undisturbed material.

The angle ∅' can also be estimated trom the effective particle size, the grain size distribution, the relative density and the angularity of the soil particles as has been suggested by Lundgren & Brinch Hansen (1958).

Frequently, piles are driven through a layer of clay down to a cohesionless material with high bearing capacity as is shown in fig. 7. The point bearing capacity will in this case correspond to the value of N_q which is applicable to foundations located close to the ground surface. The point bearing capacity of a pile will be overestimated if the N_q-values suggested by Berezantsev Khristoforov & Golubkov (1961) are used.

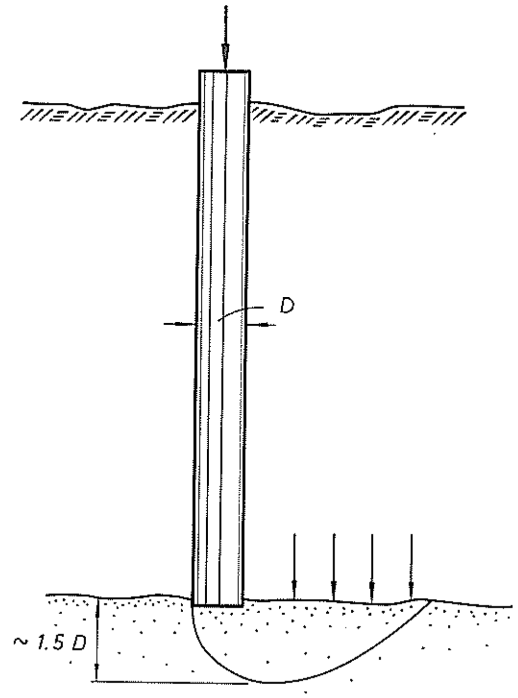

Fig. 7

Determination of End Bearing Capacity for a Pile Which Has Been Driven Through a Cohesive Material to a Cohesionless Material with High Bearing Capacity
Détermination de la force portante en pointe pour un pieu qui a été battu à travers un matériau cohérent jusqu'à un terrain pulvérulent de grande résistance
Determinación de la capacidad portante en la punta de un pilote hincado en un material cohesivo hasta un terreno sin cohesión de gran resistancia
Bestimmung der Spitzentragkraft für einen Pfahl, der durch bindiges Material hindurch in nicht bindige Schichten hoher Belastungsfähigkeit gerammt wurde

Another common case is shown in fig. 8. In this case the pile is supported by a relatively thin layer with a high bearing capacity. Toe failure will occur either along A or B. If on one hand failure occurs along failure surface A the bearing capacity of the pile can be calculated with the N_q-values mentioned above. If failure on the other hand occurs along surface B the bearing capacity can be estimated by assuming that the load is distributed over an area with a diameter which, at the bottom of the dense layer itself, is equal to the sum of the pile diameter and the thickness of this layer. The ultimate capacity corresponds in this case to that of a pile with a diameter $(D + t)$ and a length $(L \div t)$, where t is the thickness of the dense layer. This calculated ultimate load will govern if it is lower than that which corresponds to failure surface A.

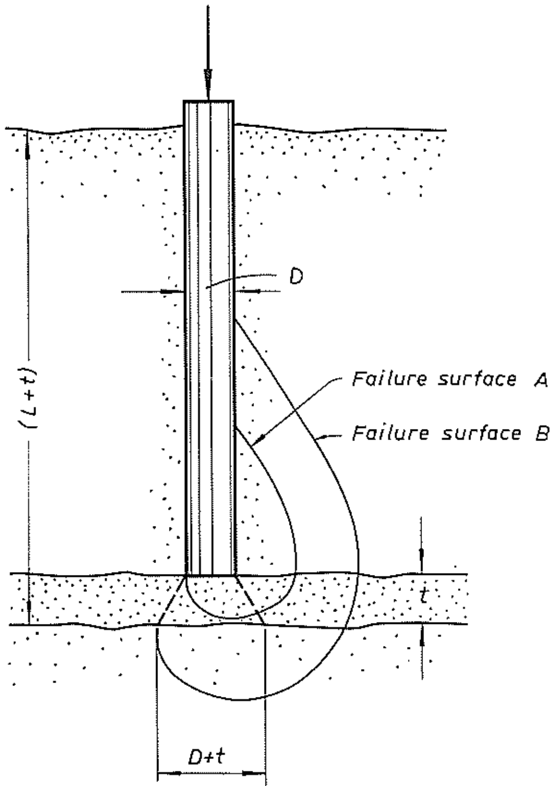

Fig. 8

Determination of End Bearing Capacity for a Pile Driven to a Thin Layer with High Bearing Capacity
Détermination de la force portante en pointe pour un pieu battu jusqu'à une mince couche de grande résistance
Determinación de la capacidad portante en la punta de un pilote hincado hasta una capa delgada muy resistente
Bestimmung der Spitzen Tragkraft eines Pfahles, der in eine dünne Schicht hoher Belastungsfähigkeit hinein gerammt wurde

L + t equivalent pile length longueur équivalente du pieu longitud equivalente del pilote äquivalente Länge	D + t equivalent diameter diamètre équivalent diámetro equivalente äquivalent Durchmesser

Numerical example

Calculate the bearing capacity of a wood pile which has been driven through 30 ft of loose or medium sand with an average angle of internal friction $\varnothing'$ of 32°. The diameter of the pile is 10.0 in. and 6.0 in. at top and bottom, respectively. The pile point has been driven to a coarse sand with an estimated angle of internal friction $\varnothing'$ of 35°. The ground water table is located 15 ft below the ground surface. The unit weight of the soil is 110 lb/ft^3 above the ground water table and 120 lb/ft^3 (the saturated unit weight) below the ground water surface.

The skin friction resistance Q^1_{skin} can be calculated from Eq. (4) for the part of the pile which is located above the ground water surface. The length L is in this equation equal to 15 ft, and coefficient K_0 and $\varnothing_a$ can be estimated as 1.5 (Table I) and 21.3° (Table II) respectively. The corresponding unit weight of the soil and the average diameter of the pile is 110 lb/ft^3 and 9 in respectively. The resulting skin friction resistance Q^1_{mantel} is 18.9 kips (Eq. 4).

The skin friction resistance Q^2_{skin} is proportional to the submerged unit weight of the soil for the part of the pile which is located below the ground water surface. The corresponding submerged unit weight is 57.5 lb/ft^3 (120-62.5) and the average pile diameter for this part of the pile is 7 in. The resulting skin friction resistance Q^2_{skin} is 33.5 kips.

The point bearing capacity Q_{point} can be calculated from Eq. (9) and is proportional to the vertical effective overburden pressure $\bar{\gamma}L$ which exists at the level of the pile point and the bearing capacity factor N_q. The corresponding vertical effective pressure $\bar{\gamma}L$ is 25.1 kips/ft^2. The bearing capacity factor N_q corresponding to the angle of internal friction of 35° is 45. The end bearing capacity Q_{point} can then be calculated from Eq. (9) as 22.1 kips. The total bearing capacity of the pile, the sum of the skin friction resistance and the point resistance can be calculated as 74.5 kips. If a factor of safety of 2.5 is chosen the allowed load on the pile will be 15 tons (14.9).

Cohesive soils

Calculation of skin friction resistance Q_{skin}

The skin friction resistance for piles which are driven in cohesive soils is frequently larger than 80-90 % of the total bearing capacity. For such piles it is of importance that the skin friction resistance can be estimated accurately. The total skin friction resistance is directly proportional to the total surface of the pile, the average adhesion of the soil c_a being the ratio of proportion. Thus

$$Q_{skin} = c_a \, A_{skin} \tag{10}$$

Comparisons with test data have shown that the adhesion c_a depends on the undrained shear strength c_u of the cohesive material. Test results which have been reported by Seed & Reese (1955), Bjerrum (1953), Peck (1955), Fellenius (1955), Tomlinson (1957), Bergfelt (1957), Vey (1957), Peck (1958), Mohan & Jain (1961) and Woodward, Lundgren & Boitano (1961) have been used in this comparison. When the shear strength c_u is less than approximately 1,000 lb/ft^2 the adhesion c_a is approximately equal to the undrained shear strength. When c_u is larger than 1,000 lb/ft^2 the adhesion will be dependent of the pile material. The adhesion will as a rule be larger for wood or concrete piles than for steel piles (Lo & Stermac, 1964).

One reason for the observed variations in adhesion is vibrations which develop in the pile during driving. These vibrations cause a hole in the soil with a diameter which is somewhat larger than the diameter of the piles. When the shear strength of the soil is larger than 1,000 lb/ft^2 the shear strength of soil is generally sufficiently large to keep the enlarged hole open without lateral support and the soil will not flow back around the pile. An additional factor which is of importance is that wood and concrete piles serve as vertical drains because of the relatively high permeability of the pile material. Consolidation of the clay located close to such piles will therefore occur relatively rapidly. Around steel piles consolidation will take place slowly since they cannot serve as drains.

Wood piles are generally somewhat conical and due to this reason good contact is obtained between such piles and the surrounding soil. This is one reason why a higher adhesion is generally observed for wood piles than for concrete or steel piles. Other explanations have also been suggested for this phenomenon (Fellenius, 1938 and 1955).

The amplitude of the lateral vibration during driving is probably smaller for concrete piles than for steel piles because of differences in stiffness between the two pile types. One can therefore expect that the adhesion along the pile surface will for steel piles be lower than that for concrete piles at least close to the ground surface. This has been substantiated by field measurements (Tomlinson, 1957).

High adhesion has also been measured for cast-in-place piles (Lo & Stermac, 1964).

The following values of the adhesive strength are recommended to be used for the calculation of the bearing capacity of piles which have been driven into cohesive soils. (Field investigations have shown that often six months are required to develop this adhesion.)

Table III. Evaluation of the adhesion c_a (lb/ft^2) from the measured undrained shear strength c_u of the surrounding cohesive soil*.

(a) $c_u < 1{,}000$ lb/ft^2	Adhesion c_a
Steel piles	0.5 c_u
Concrete piles	0.8 c_u
Wood piles	1.0 c_u
(b) $c_u > 1{,}000$ lb/ft^2 :	
Steel piles	200 lb/ft^2
Concrete piles	600 lb/ft^2
Wood piles	1,000 lb/ft^2

(*) The undrained shear strength of the clay can be determined from unconfined compression tests, undrained triaxial tests, undrained direct shear tests, vane tests or Swedish fall-cone tests. The adhesion values determined from pile load tests have in general been compared with the shear strength obtained from unconfined compression tests, undrained direct shear tests or Swedish fall-cone tests. The shear strengths determined by vane tests are often higher than those determined by other methods. Due to this reason the shear strengths determined by vane tests are frequently reduced by 20-30 % before they are used to calculate the skin friction resistance.

The skin friction resistance can be very low for the upper part of a spliced wood pile when the pile has been driven through a dry crust (Fellenius, 1955). The reason for this low adhesion is that the lower part of the pile forms a hole within the dry crust with a diameter which is larger than the diameter of the upper section of the pile. The clay will not be able to flow back around the pile due to the high shear strength of the stiff clay in the dry crust and the low overburden pressure (the distance to the ground surface is small). Consequently the adhesion will therefore be low for such piles. It is recommended to neglect the skin friction resistance of the upper part of the pile which is located in the stiff layer.

An other case to consider is the skin friction resistance of a pile which has been driven with its larger part first (Fellenius, 1955). This resistance is in this case considerably lower than that which corresponds to the undisturbed shear strength of the clay. To determine the bearing capacity of such piles one is forced to carry out pile tests.

In this connection it should also be mentioned that the adhesion for piles which have been placed in drilled holes or been placed with the aid of jetting is considerably lower than that of driven piles (Mohan & Chandra, 1961).

Calculation of point bearing capacity Q_{point}

The point bearing capacity of cohesive materials can also be calculated from Eq. (5). The bearing capacity factors N_γ and N_q are equal to zero and 1.0 respectively for a cohesive soil. The cohesion for such piles is equal to the undrained shear strength c_u. This cohesive strength c_u can be determined from vane tests, Swedish fall-cone tests, undrained shear tests or unconfined compression tests. Eq. (5) can then be rewritten as:

$$q_{point} = .3\ c_u N_c + L\bar{\gamma} \qquad (11)$$

The useful load carried by a pile (the net load) is the difference between the gross bearing capacity and the weight of the pile. If the unit weight of the pile material is assumed equal to the unit weight of the surrounding soil the net point resistance will be:

$$q_{point} = 1.3\ c_u\ N_c \qquad (12)$$

The bearing capacity factor N_c can be calculated from the theory of plasticity. Theoretical calculations, laboratory and field investigations have shown that the combined bearing capacity factor 1.3 N_c is approximately 9.0 when the pile point is located at the depth exceeding four pile diameters below the ground surface (Meyerhof, 1951, Skempton, 1951). Thus:

$$q_{point} = 9.0\ c_u \qquad (13)$$

The bearing capacity of the pile can then be calculated as the sum of the point bearing resistance (Eq. 13) and the skin friction resistance. In general the point bearing capacity is 10-20 % of the total bearing capacity of the pile. Due to this reason it is not necessary to calculate accurately the end bearing resistance of piles driven in cohesive soils. Variations in the end bearing resistance will not have a large influence on the total bearing capacity of the pile.

Numerical example

Calculate the ultimate bearing capacity of a 45 ft long concrete pile which has been driven through 12 ft of clay with an average shear strength of 2,000 lb/ft^2 into a thick clay layer with an average shearing strength of 500 lb/ft^2. The cross section of the pile is 10 × 10 in.

The skin friction resistance can be calculated from equation (10). The adhesion of the upper portion of the pile is limited to 600 lb/ft^2 (Table III) and is for the lower part of the pile equal to 400 lb/ft^2 (0.8 × 500 lb/ft^2), the corrected undrained shear strength of the soil. The resulting skin friction resistance is 68.0 kips.

The point bearing capacity can be calculated from Eq. (13) and depends on the undrained shearing strength of the clay (500 lb/ft^2). The total end bearing capacity is thus 3.1 kips. The resulting total ultimate load is 71.1 kips. If a factor of safety of 2.5 is chosen, then the allowable pile load of the pile is 14 tons (14.2 tons).

Bearing Capacity of Piles from Static Penetration Tests.

The bearing capacity of piles driven into cohesionless soils can also be calculated from static penetration tests. Static penetration tests have been described, among others, by Schultze (1957), Gamski (1961), Haefeli & Bucher (1961), Kallstenius (1961) and Shockley, Cunny & Strohm (1961)).

Calculation of point bearing capacity Q_{point}

With the Dutch cone penetrometer one measures the penetration resistance of a conical probe which is pushed slowly into a soil. This type of penetrometer has been described by Plantema (1948 a), Vermeiden (1948), Kantey (1951), Allaart, Mierlo & Nanninga (1960).

The point bearing capacity of a pile is for cohesionless soils dependent of the effective overburden pressure which exists at the level of the pile point and of the bearing capacity factor N_q (Eq. 9). One can see from Eq. (9) that the point bearing capacity is independent of the point diameter. Thus the results which are obtained with Dutch cone penetrometer can be used directly to determine the point bearing capacity of piles.

Menzenbach (1961) has made an extensive investigation of the relationship between the penetration resistance of the Dutch cone penetrometer and the bearing capacity of piles. These comparisons have shown that the measured penetration resistance is approximately equal to the point bearing capacity. Similar observations have been made by Plantema (1948), van der Veen (1953), and Mohan, Jain & Kumar (1963). However one can observe that the measured point bearing capacity of a pile is smaller than the penetration resistance when the penetration resistance measured by the Dutch cone penetrometer is larger than 100 t/ft^2. These investigations show that the results obtained with the Dutch cone penetrometer can be used directly without corrections when the penetration resistance is less than 100 t/ft^2.

On basis of these observations it is recommended that the ultimate point bearing capacity is taken as the penetration resistance when this resistance is lower than 100 t/ft^2 and that the ultimate point bearing capacity is taken as 100 t/ft^2 when the penetration resistance exceeds 100 t/ft^2. It should be noted that the point bearing capacity should be taken as the average penetration resistance which is measured within an area which extends from 3.75 pile diameters above the pile point down to one pile diameter below the pile point (van der Veen & Boersma, 1957).

Meyerhof (1960) has suggested that the point bearing capacity of « rammed » piles (e.g. "Franki" piles) should be evaluated as twice the penetration resistance measured by the Dutch cone penetrometer.

Kerisel (1961) has shown that the ultimate bearing capacity of a pile can be lower than the point resistance measured by the Dutch cone penetrometer when the diameter of the pile is large. Due to this reason it is recommended that the results from the Dutch penetrometer should not be used without reduction of point resistance when the pile diameter is larger than 20 in.

Calculation of skin friction resistance Q_{skin}

The skin friction resistance can be estimated with the Dutch cone penetrometer when the piles are driven into cohesionless soils. In this case the skin friction resistance is frequently low in comparison with the total bearing capacity of the pile and can be calculated from Eq. (4). The relative density of a cohesionless soil can be considered to be low when the point resistance measured with the Dutch cone penetrometer is 0-50 t/ft^2, normal when it is between 50 and 100 t/ft^2 and high when exceeding 100 t/ft^2.

Meyerhof (1956) has recommended that skin friction resistance should for design purposes be taken as 0.5 % of the measured point bearing capacity. However field tests carried out by Mohan, Jain & Kumar (1963) have shown that the skin friction resistance can be considerably larger than the value recommended by Meyerhof.

Numerical example

Calculate the bearing capacity of a 45 ft long concrete pile with a cross section of 10 x 10 in. The pile has been driven through a layer of loose, fine sand with an estimated submerged unit weight of 65 lb/ft^3 down to a dense layer of coarse sand with an average point bearing resistance of 120 t/ft^2 within an area which extends from 37.5 in (3.75 x 10) above the pile point to 10 in (1.0 x 10) below the pile point. The ground water table is located at the ground surface.

The skin friction resistance of the pile can be calculated from Eq. (4). The coefficient $K_o = 1.0$ according to Table I for a concrete pile which has been driven through a cohesionless material with a low relative density. The friction angle $\varnothing_a$ for the pile surface with respect to the surrounding soil can be estimated from Table II as 22.5° (3/4 × 30°) when the angle of internal friction $\varnothing'$ of the soil is 30°. Using these values the calculated skin friction resistance $Q_{skin} = 91.2$ kips.

The point bearing capacity is limited to 100 t/ft^2 if the point resistance measured by the Dutch cone penetrometer exceeds 100 t/ft^2. The corresponding calculated ultimate point bearing capacity of the pile is 138.6 kips.

The total pile bearing capacity is equal to the sum of the skin friction resistance and point resistance. The total bearing capacity is thus 229.8 kips or 114.9 tons. If a safety factor of 3.0 is used the allowable pile load is 38 tons (38.3). It should be observed in this connection that a higher safety factor is frequently used for piles with small base areas to limit the settlements of such piles at working loads (Allaart, Mierle & Nanninga, 1960).

Bearing Capacity of Piles from Dynamic Penetration Tests

The bearing capacity of piles which are driven in cohesionless soils can be estimated from the standard penetration test. The N-value which is obtained from this test is the number of blows which is required to drive a standard sampler 12 in into the bottom of a bore hole by a 140 lb weight with a free fall of 30 in (Terzaghi & Peck, 1948).

Calculation of point bearing capacity Q_{point}

Meyerhof (1956) has suggested on the basis of pile tests that the point resistance of a pile expressed in t/ft^2 is equal to approximately 4N where N is the standard penetration resistance. The test results show however considerable scatter. A lower limit represents the relationship 2.5 N and this relationship may be used for the calculation of the point bearing capacity of piles which are driven into cohesionless materials.

Calculation of skin friction resistance Q_{skin}

The skin friction resistance of piles is governed by the relative density of the surrounding soil (Terzaghi & Peck, 1948). When the N-value from the standard penetration test is between 0 and 10 blows the relative density of the soil is low to very low, between 10 and 30, the material is medium dense and, larger than 30, it is dense to very dense (Terzaghi & Peck, 1948). Meyerhof has suggested that skin friction resistance can be evaluated directly from the N-value as 0.02 N (t/ft^2).

The bearing capacity of piles driven in cohesive materials can also be estimated from the standard penetration test. Comparisons with test data have shown that the N-value increases with increasing shear strength of the cohesive material. The undrained shear strength c_u in t/ft^2 is approximately equal to N/5. However according to Terzaghi and Peck (1948) the undrained shear strength for a clay in t/ft^2 is approximately N/8. From this relationship the point bearing and the skin friction resistance of a pile can be calculated as described previously. However this method of evaluating the bearing capacity is very uncertain.

Numerical example

Calculate the bearing capacity of a 45 ft long wood pile which has been driven through a sand layer with an average N-value of 5 blows/ft. The pile has been driven into a sand stratum with an average N-value of 60.

The pile diameter at top and bottom is 16 in and 8 in respectively. The ground water table is located at the ground surface. The submerged unit weight of the soil is 60 lb/ft^3. The angle of internal friction for the loosely packed sand is estimated to 30°.

The coefficient K_0 (Table I) and the skin friction resistance $\varnothing_a$ (Table II) are estimated to 1.5 and 20° respectively. The skin friction resistance calculated from Eq. (4) is then 104 kips.

The point bearing resistance can safely be calculated from the relationship:

$q_{point} = 2.5 \ N \ (t/ft^2)$

and is 150 t/ft^2. The corresponding calculated point resistance and total bearing capacity are 52.2 t and 104.2 t respectively. If a satefy factor of 3.0 is chosen the allowable pile load is 35 t (34.7 t). It should be noted that settlements have not been considered in these calculations.

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Revue des méthodes de calcul de la force portante limite des pieux

Bengt B. Broms

Résumé

On décrit ici les méthodes d'évaluation de la force portante des pieux à partir des propriétés mécaniques du sol ou à partir d'essais de pénétration dynamiques ou statiques. On peut dire qu'il n'existe pas actuellement de méthode générale valable pour déterminer d'une manière satisfaisante dans n'importe quel cas la force portante d'un pieu. Avec les méthodes proposées ici la force portante d'un pieu peut être déterminée dans quelques cas particuliers. Dans la plupart des cas de nombreuses incertitudes existent relativement à la force portante réelle d'un pieu quand des essais de chargements directs n'ont pas été réalisés.

Durchsicht von Berechnungsmethoden für die Grenztragfähigkeit von Pfählen

Bengt B. Broms

Zusammenfassung

Der Verfasser beschreibt die Methoden der Bestimmung der Tragkraft von Pfählen nach den mechanischen Eigenschafften des Bodens, die durch statische oder dynamische Penetrationsversuche ermittelt wurden. Man kann sagen, dass es zur Zeit keine allgemeingültige Methode gibt, in allen Fällen, zu friedenstellend die Tragkraft von Pfählen zu bestimmen. Mit der hier Vorgeschlagenen Rechnungsart kann die Tragkraft in einigen besonderen Fällen errechnet werden. In der Mehrzahl der Fälle bleiben zahlreiche Unsicherheiten über die wirkliche Tragkraft eines Pfahles übrig, besonders wenn direkte Belastungsversuche nicht ausgeführt worden sind.

Revista de los métodos de cálculo de la capacidad portante límite de los pilotes

Bengt B. Broms

Resumen

Se describe aquí los métodos de evaluación de la capacidad portante de los pilotes a partir de los ensayos de penetración dinámicos o estáticos. Se puede decir que no existe actualmente un método general aceptable para determinar de una manera satisfactoria en cualquier caso la capacidad portante de un pilote. Con los métodos aqui propuestos se puede determinar la capacidad portante de un pilote en algunos casos particulares. En la mayor parte de los casos quedan numerosas dudas relativas a la capacidad portante real de un pilote cuando los ensayos de carga directa no se han realizado.

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