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SOME CONSIDERATIONS WITH REGARD TO THE BEARING CAPACITY OF FOUNDATION PILES

A. F. VAN WEELE[1]

ABSTRACT

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A practical approach is made to analyse the stresses and deformations in the ground before, during and after installation of foundation piles of various types. This exercise leads to the following conclusions:
(1) Displacement type of piles improve the stress state in the surrounding soil during installation, while non-displacement type of piles will have an adverse effect. (2) A foundation on many small-diametre piles will show less deformation than an alternative foundation on a small number of large-diametre piles. When an equal deformation is aimed at, the factor of safety to be applied to the large-diametre piles, should be substantially larger. (3) Driven piles improve the stress-state in the soil around their lower ends most, but increasing pile penetration during installation deteriorate this good result progressively. (4) There seems to be room for increasing the stress level around the pile section in the bearing stratum after pile installation. Such an approach would enable pile-installation techniques, which are more friendly to the pile and its environment than the techniques applied nowadays. (5) Theories for the prediction of pile capacity do not take into consideration, in a sufficient manner, the way piles are installed. (6) Pile-installation techniques do not exploit the possibilities to improve pile capacities. (7) Researchers approach the problem of pile capacities as if this is a matter of arithmetics without studying in detail what really happens with the pile and the ground. (8) The predetermination of pile capacities will require a lot more attention in the future. Using our knowledge about the geology of the underground, about the installation techniques to be applied and about the pile-shape and the pile-material needs more attention than hitherto.

INTRODUCTION

The predetermination of the ultimate capacity of a foundation pile of a given size in a given soil profile, is an extremely complicated problem, which cannot be solved by means of a purely theoretical approach only. Factors, such as the geological history of the subsoil, the way and sequence of the pile installation and the establishment of relevant soil parameters, play a very important role. Based on a continuing experience with a certain type of pile in one and the same type of soil profile, experience may become a factor of importance in predetermining the pile's behaviour. In this paper a number of factors, determining a foundation pile's behaviour, are discussed in detail.

[1] Professor in foundation engineering, Technological University, Dept. of Civil Engineering, Stevinweg 1, 2628 CN DELFT, The Netherlands.

SOIL HISTORY

It is customary to use pile foundations in soil conditions, where soft and compressible layers are present, that show unacceptable deformations, when loaded by a structure to be founded. A deep foundation enables the designer to bring the structure's load to a level, deeper than that of the soft and compressible layers. Usually these soft layers are mainly present directly below the surface; in The Netherlands normally to max. 20 m depth (Fig. 1) (VELDHUYZEN, 1971). General practice has shown that with a pile length of 40-45 m practically 100% of all piling problems on land can be solved in our world.

Soft soils are usually those, which were deposited during the Holocene and which have a large water content, such as clays and especially peats. Between both soil types, the unit weight shows a distinct difference: saturated soft clays 15-17 kN/m^3 and peat 10-11 kN/m^3.

The effective stress level and the period this level has been present, are a measure for the density each soil layer shows

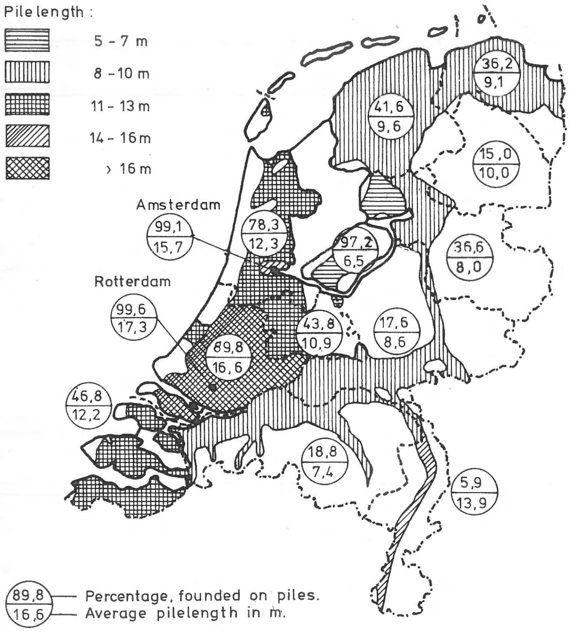

Fig. 1
Information on pile foundations for one-family houses in The Netherlands.

nowadays. For clays these effective stresses increase with depth and so the strength of the clay–and thus its bearing capacity–increases with depth (Fig. 2a).

For most of the young peat deposits in the western Netherlands, which were formed in the recent 5000 years, the submerged weight was practically zero, so that compression under its own weight would hardly take place (Fig. 2b). Such layers, when loaded, show a very large compression and make a deep foundation necessary for any kind of structure – no matter how light in weight.

The history of a given soil profile should be known before a foundation design is made. To illustrate this point an example is chosen from the northern part of The Netherlands. Here the glaciers from the Riss ice age covered the whole area approximately 200,000 years ago during more than 50,000 years. Soft clay deposits from just before that period were overlain by the glaciers and compressed to a considerable extent. The large effective stresses and the very long loading time have resulted in an increased density which is reflected in the cone resistance (Fig. 3).

After melting of the glacier, the surcharge load disappeared while at the same time the sea level and thus also the level of the groundwater moved upwards which resulted in increased groundwater pressures and thus to a decreased effective stress level. Loading such a clay layer nowadays will only result in fairly direct elastic compression while a time-dependent consolidation process of the clay will actually not take place. Under such circumstances the clay can in most of the cases be used as a foundation layer.

In The Netherlands such over-consolidated clays are rare while the soft young clay deposits are very generally present. Our appreciation of clay is therefore not a favourable one, while in Houston for instance, where over-consolidated clays are very common, the foundation engineer is of the opinion that clay is the best foundation material.

EXISTING STRESSES

Another aspect is that of the existing stress pattern. Well known are the simple rules:

total vertical stress = vertical effective stress + water pressure

$$\left.\begin{array}{l}\text{or: } \varrho_{\text{v}} = \varrho_{\text{v}'} + \text{u} \\ \text{and: } \varrho_{\text{v}} = \Sigma\,\gamma\,\text{h}\end{array}\right\} \rightarrow \Sigma\,\gamma\,\text{h} = \varrho_{\text{v}'} + \text{u}.$$

γ = unit weight of soil in kN/m^3.
h = soil layer thickness in m.

This formula is a simplification of the real situation. It assumes that in each vertical plane the shear stresses are zero. This might be true only if the ground surface is, and has always been, exactly level, while the soil profile should also be uniform, so that differential settlements did not occur. Old dunes or old rivers may have had their effects in the deeper subsoil which is able to remember its past and to translate it into the existing stress pattern, thus introducing shear stresses in the vertical planes. Also man could have created discontinuities in vertical stresses, as is for instance the case in areas where underground mines were exploited.

We do not know at all, however, the horizontal effective stress. Nevertheless, we have learned in soil mechanics how to live with this problem. We say $\varrho'_{\text{h}} = \text{K}.\varrho_{\text{v}}'$, in which formula we call K the coefficient of earth pressure. In a given soil profile at rest we call this coefficient then the 'at rest' coefficient and give it the 'neutral value' K_o. The neutral value is that relationship between horizontal and vertical effective stresses in the soil which does not give any horizontal deformation. We still do not know its value; we only know that it must be in between the two ultimate values called active and passive coefficients of earth pressure.

In each elementary soil element, there is only one ϱ_{v}', but of course there are very many ϱ_{h}''s. Our next assumption is that the ϱ'_{h} is equal in all directions. In situations of present or past discontinuities, this assumption is of course highly questionable. In the case of the subsoil overlain by glaciers for instance, the effective stresses in the past flow direction of the glacier will certainly be in excess of those in the opposite direction. This also applies to points under a sloping riverbank etc.

In practice it has proven to be very difficult to measure in the field the magnitude of the average K_o. In the recent past development work has been going on to develop self boring pressio-meters with which rough indications can be obtained about the average value of this most important coefficient. We usually assume that K_o = 0.5-0.6 for young sediments, but for the over-consolidated North Sea clays we know that K_o can be as high as 5-8!

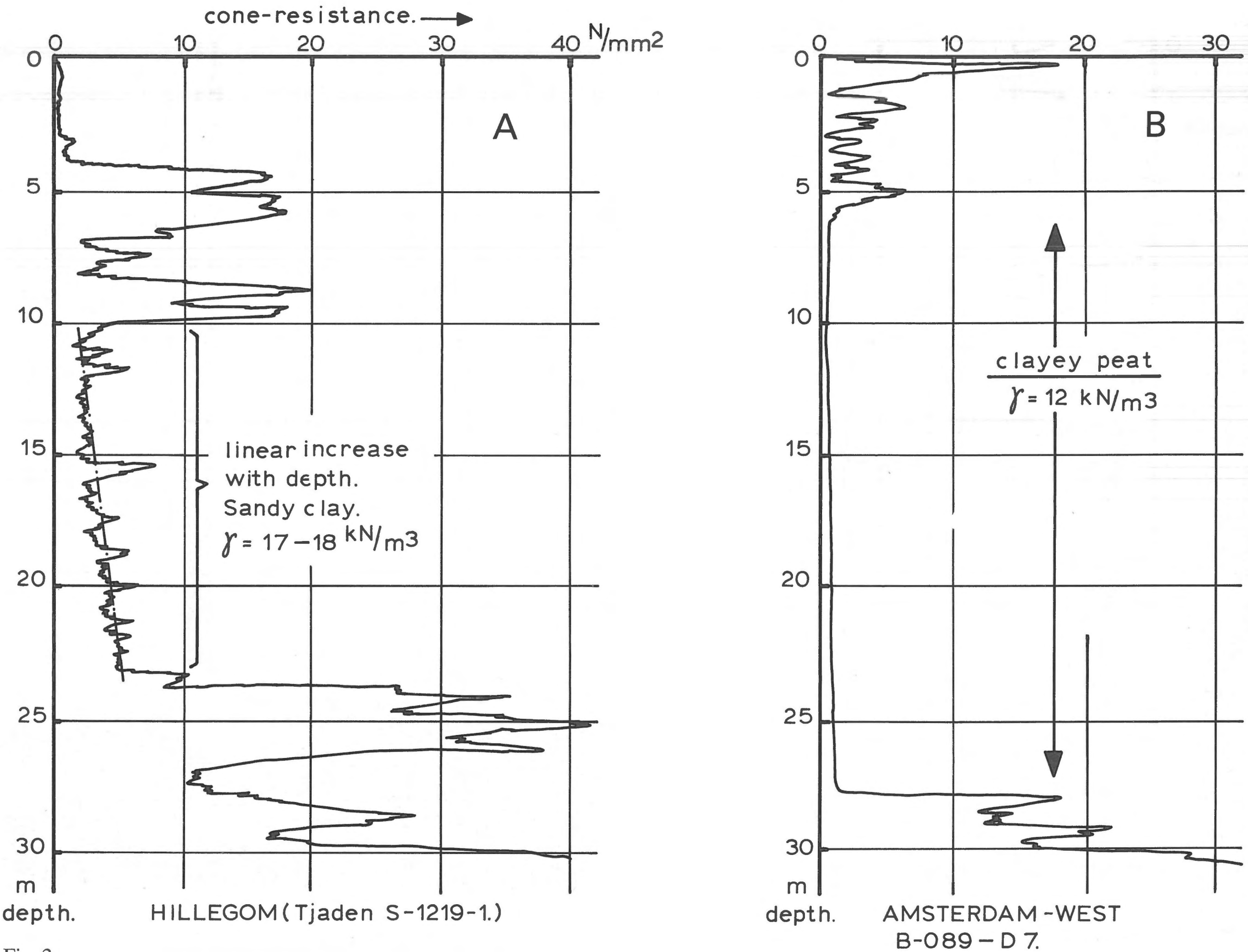

Fig. 2
Influence of consolidation pressure on the strength of cohesive soils.

PILE FOUNDATION DESIGN

The load exerted by the super-stucture is transferred by a foundation pile to the bearing layers by means of:
(1) skin friction and
(2) end bearing.

This load transfer takes only place in case that the pile moves downwards relative to the soil. Such movement is of course kept small, namely ranging from a few millimetres to a few centimetres.

There are many locations, especially in The Netherlands, where the very soft top layers show a continuing settlement ranging between 1 and 100 mm per year (see Fig. 4) (ANONYMOUS, 1965). In such situations the top layers of the soil profile settle more than the pile, so that the pile experiences a permanent downdrag which has to be added to the nett load on the pile head. This process results therein that the weight of the top layers is partly fransferred through the downdrag or negative skin friction to the piles, thus decreasing the effective vertical stresses in the underlying layers. This decrease in stress level leads to a deceleration of the rate of settlement until a new equilibrium is found between loads and deformations (ANONYMOUS, 1975).

It is clear that the pile loads which are transferred to the bearing stratum, will result in an increased level of the vertical and horizontal effective stresses in this stratum. The deformations resulting from the stress increase should remain well below what is acceptable to the super-structure (SKEMPTON & MACDONALD, 1956).

The soils engineer in his design is confronted with the following problem: *How much increase in horizontal and vertical effective stresses is allowable above those already existing (without knowing actually what the existing stresses are!)?* Furthermore he must be aware of the fact that the pile installation in itself may create considerable changes in stresses, in vertical as well as in horizontal direction. It is beyond saying that solving this problem requires not only thorough knowledge of soil mechanics, but also local experience. Experience from Amsterdam cannot be exported to Tokyo etc. This aspect creates a lot of hectic discussion between colleagues from different parts of the world with different experience gained in soils with different geological background.

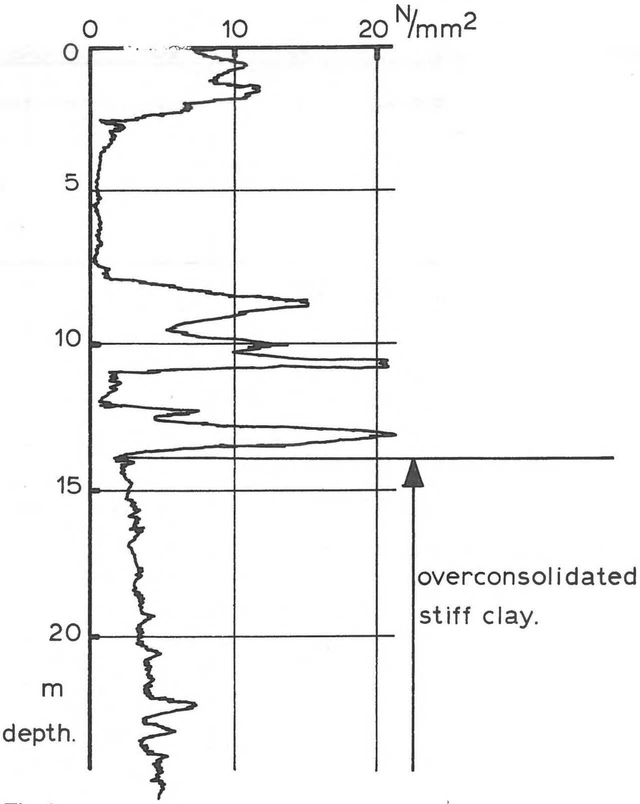

Fig. 3
Examples of over-consolidated clays.

Fig. 5
Driving of precast concrete piles.

PILE INSTALLATION

Pile installation always results in a change in the prevailing stress pattern in the ground. A local discontinuity around the pile is created which has an undefinable shape. Foundation piles can be classified in two main groups based on their way of installation:

(1) Displacement type of piles. These force the ground aside in order to make room for the pile volume. To this category belong piles which are driven, vibrated or jacked into the ground (Fig. 5).

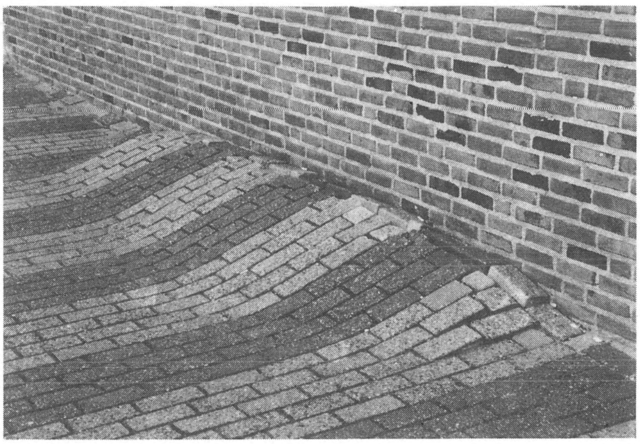

Fig. 4
Obstructed ground-surface settlement due to an extended footing, founded on piles. Indication for occurrence of negative friction.

(2) Non-displacement type of piles. Such piles are formed in prebored holes, where the soil is taken out of the ground beforehand. This group includes jetted piles and bored piles with or without temporary casing to line the hole (Fig. 6).

Actually there is even a third type of pile which is of the non-displacement type, but the borehole is not allowed to stand open during the installation process. This augered pile makes use of a continuous flight auger which is withdrawn from the ground without rotating while pumping cement grout through the hollow stem to the foot of the auger (Fig. 7). The extracted auger with soil is immediately followed by the pile material, thus leaving the surrounding soil little time to deform and to change its stress pattern.

The displacement-type of pile will compress the soil and thus mostly result in an increased stress level, but the non-displacement pile first forms a borehole which must result in a considerable stress release in the surrounding layers. The starting position of the stress pattern, when loading the completed pile, will therefore for all three pile types in the same

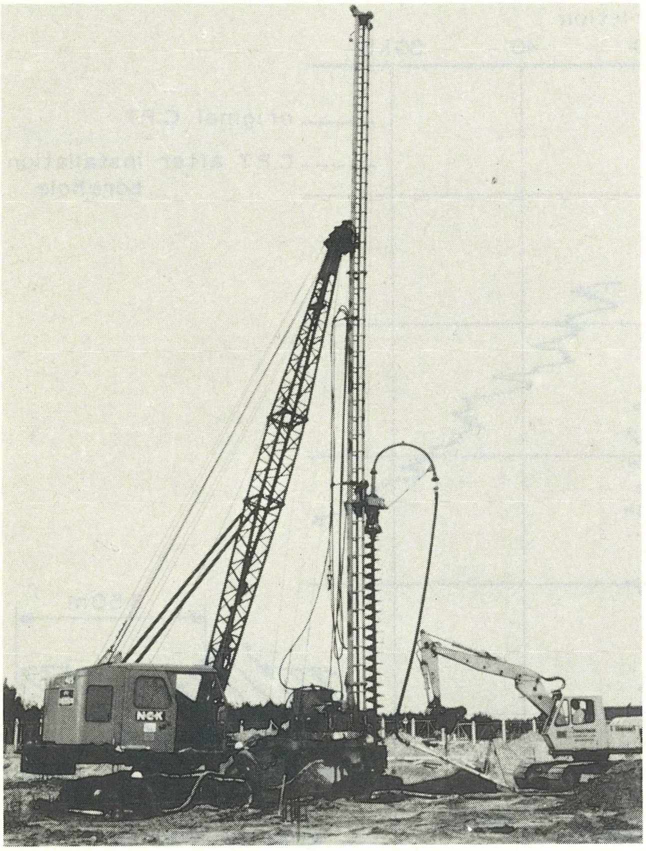

Fig. 6
The installation of a large-diametre bored pile.

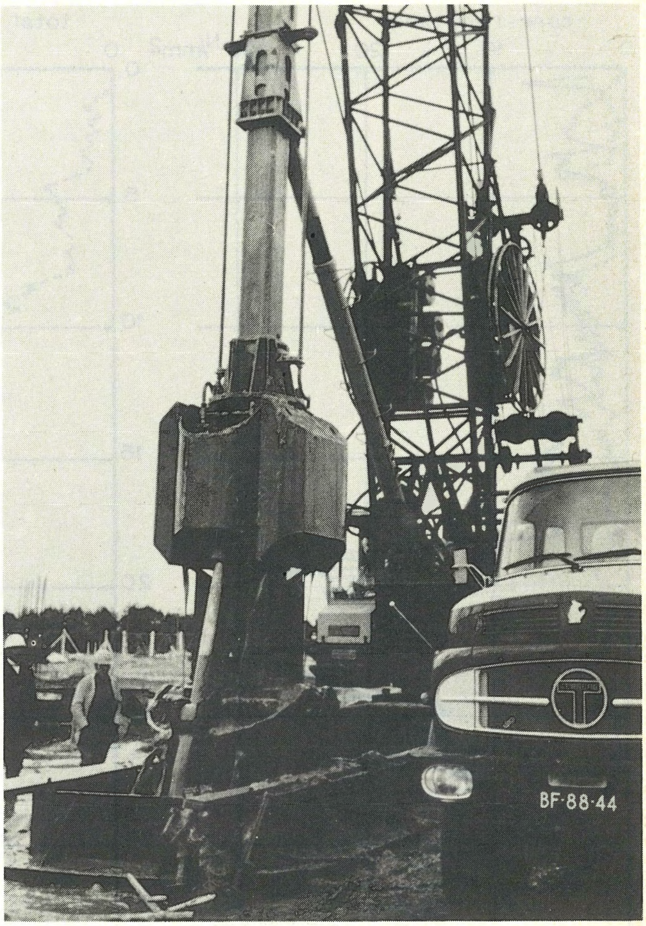

Fig. 7
The installation of a cast in place augered pile.

soil for the same pile dimensions be entirely different. Each sound design of a pile foundation must take such differences into consideration.

The changes themselves, mentioned above, will also be influenced by the type of soil. It is the permeability of the soil that is the most important parameter in this respect. In sandy soils the permeability is large and thus changes in density can easily take place. Sands act spontaneously. A decrease in stress will at once lead to some expansion of the grain skeleton and the other way around.

The behaviour of pure clays, however, is entirely different. Any change in stress level will primarily affect the water pressure and leave the intergranular stresses unaffected, unless time is long enough. As the pile installation is usually a matter of one or a few hours, the time available for the clay to react is relatively short and the effective stress level will usually not undergo important changes as a consequence of pile installation. This means that the effective stress level as a consequence of pile installation is hardly affected. It therefore can be concluded that the properties of a clay layer expressed in skin friction and end bearing will not change appreciably as a consequence of bored pile installation. It can even be concluded that skin friction in clays along a driven pile or a bored pile will show substantial differences.

In sands, however, such differences will certainly be significant. Driven piles generally have more skin friction and end bearing than is to be expected based on the undisturbed properties, while at the same time bored piles will show less skin friction and end bearing due to a loosening of the sand.

Examples of observations in the field are presented in figures 8, 9 and 10. Figure 8 shows the adverse influence of the sinking of a borehole to 30 m depth as registered by cone penetration tests (CPT's). By means of a number of CPT's the approximate shape of the area influenced as found at a certain depth is indicated. This figure also proves that the soil does not react in a uniform way. Certain directions have preference above others indicating that the horizontal effective stresses may not have been uniform in all directions.

The second example concerns a bored pile foundation for a bridge pier where the boring operation carried out under bentonite slurry led to a decreased stress level in the fine sand surrounding the piles, which stress level was not improved sufficiently by the concrete placement. This phenomenon was not found to be consistent around all bored piles, so that the boring procedure itself apparantly played an important role (Fig. 9).

A third example concerns the installation of driven piles. The increased soil strength as a consequence of the pile in-

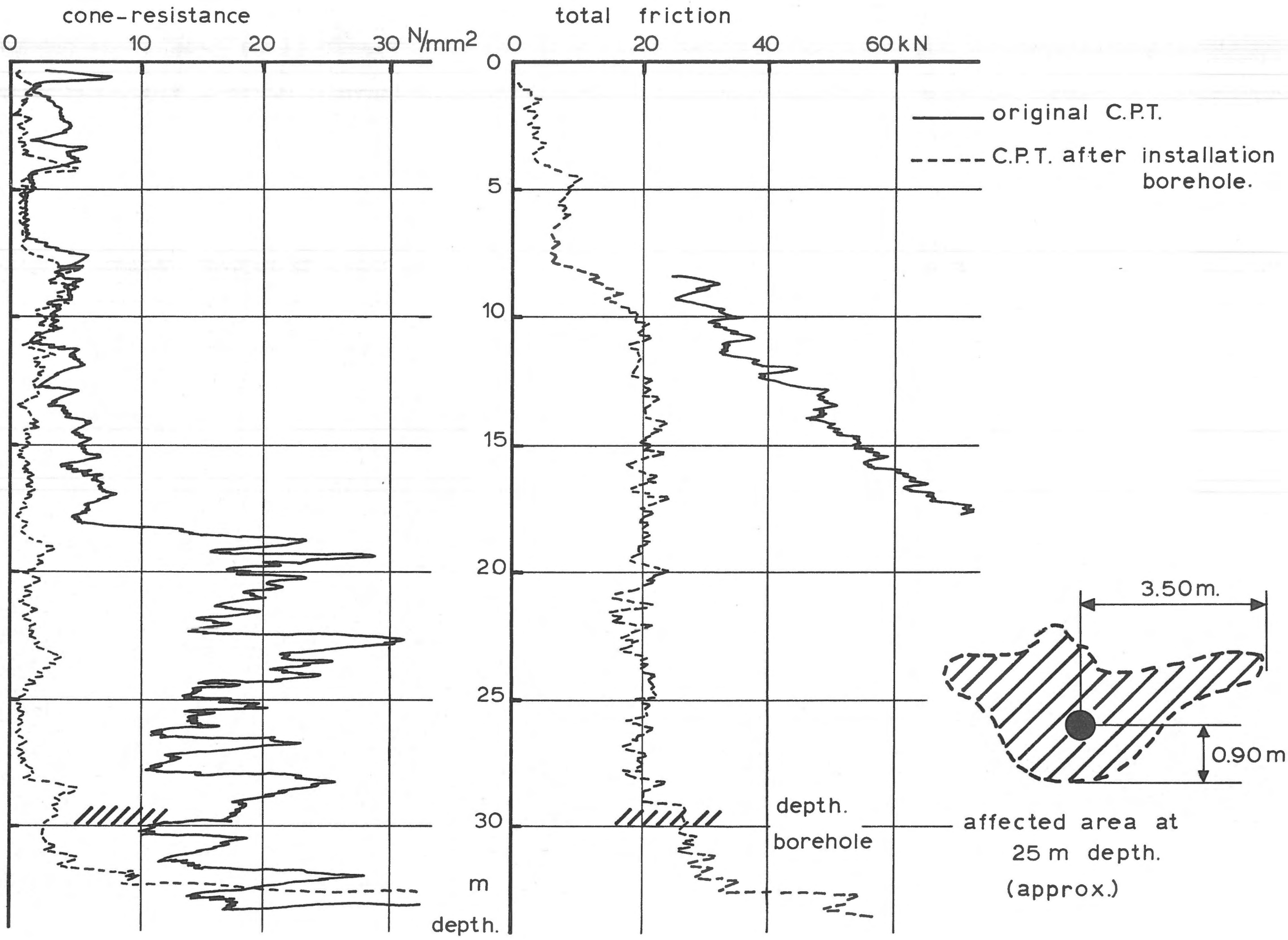

Fig. 8
Decrease in stress level due to a former bore hole as recorded by the cone penetration test.

stallation is clearly indicated by the CPT's made before and after (Fig. 10) (ANONYMOUS, 1963).

SOIL HEAVE AND NEGATIVE FRICTION

Another aspect of pile installation is that of soil heave which can easily be explained. Before pile installation we assume that in vertical planes the shear stresses are zero. During pile driving in a clay layer having a low permeability, the water pressure is increased while the effective stresses remain largely unaffected. This means that in the zone surrounding the pile, the vertical total stress is not any more balanced by the weight of the overlaying soil mass. Thus shear resistance in the vertical planes is mobilized in order to compensate the unbalance. For a single pile, such a compensation is easily feasible. In case of a large group of piles such compensation becomes more difficult, because the surface area in the horizontal plane of the group increases with the square of its diametre, while the shear stresses are mobilized around the circumference which is only linearly proportioned to that diametre. For the pile group the upward forces may therefore easily result in upward movements as a consequence of which the excess water pressures will decrease to restore the equilibrium of vertical stresses. Piles in the group are subject to upward frictional forces as a consequence of this upward soil movement. Several cases are known where pile shafts failed under tensile loads and where piles were pulled upwards from their seating in the bearing layer over several centimetres both as a result of this heave created by the installation of displacement piles. Such pile installation in overconsolidated clays will increase the risk of heave compared to that in normally consolidated clays as the lateral movements that can be taken by such clays will be less due to the much higher stress level in this direction.

In dands, the driving of piles will directly result in compaction of the sand-grain skeleton. The compaction of the sand will usually result in small vertical settlements in the ground around the pile, thus creating some temporary negative friction along nearby adjacent piles driven before. The same influence may be expected from boring operations in sandy soils. The loosening of the sands will create settlements

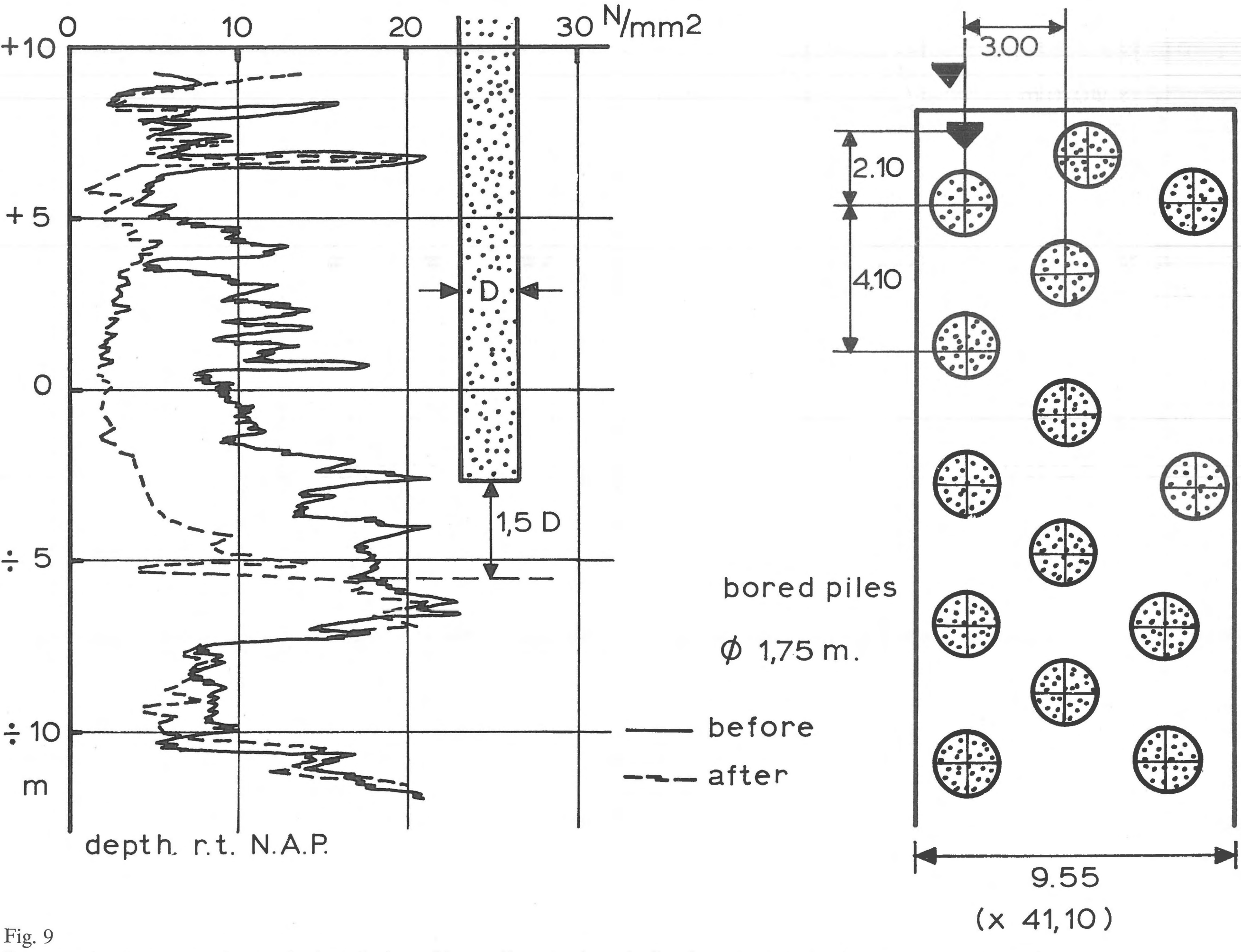

Fig. 9
Decrease in stress level due to the installation of large-diametre bored piles for a bridge pier in coarse gravelly sands.

and thus also temporary negative friction in the already completed piles at short distances. Heave in sand layers will not take place, but such a layer can be pushed upwards by an underlying impervious clay stratum.

PILE SHAPE AND FRICTION

Another factor that greatly influences the stress pattern in the soil after the installation of displacement piles is the pile shape. Usually prefabricated piles have a uniform cross section. This means that the underside of the pile has to displace the soil; afterwards the pile shaft moves deeper into the already formed hole. Therefore the soil, while being pushed aside, exerts a very large grip around the lower end of the pile. This grip results in large shear forces along the pile's surface, thus creating much friction around the pile's lower section. Further penetration of the pile through a given soil layer means increasing strains and thus decreasing frictional forces (BEGEMANN, 1954). The decrease in friction in clays results from the remoulding action and will usually restore practically the original level after full consolidation has taken place. This aspect is usually called 'set-up'.

In sands, however, the increasing penetration will result in a reshifting of the grain skeleton around the shaft. Furthermore the pile's surface is not perfectly even and smooth while during pile driving as well as during vibratory installation also small transversal movements of the pile shaft are taking place. So the soil immediately surrounding the pile is progressively brought into a looser state, while the compacted soil at a little larger distance from the shaft is in itself stable. The horizontal effective stresses acting perpendicular to the pile surface at a given depth are therefore decreasing with increasing pile penetration. The final result is a friction distribution as given in figure 11. The radial effective stress distribution is also shown (VESIC, 1970).

The most effective section of a friction pile in sand is always its lower part with a length of 5-8 pile diametres. Increasing the pile length does not add very much, because this additional length should be considered as being at a level where the pile's friction is rather low (Fig. 11) (BEGEMANN, 1969).

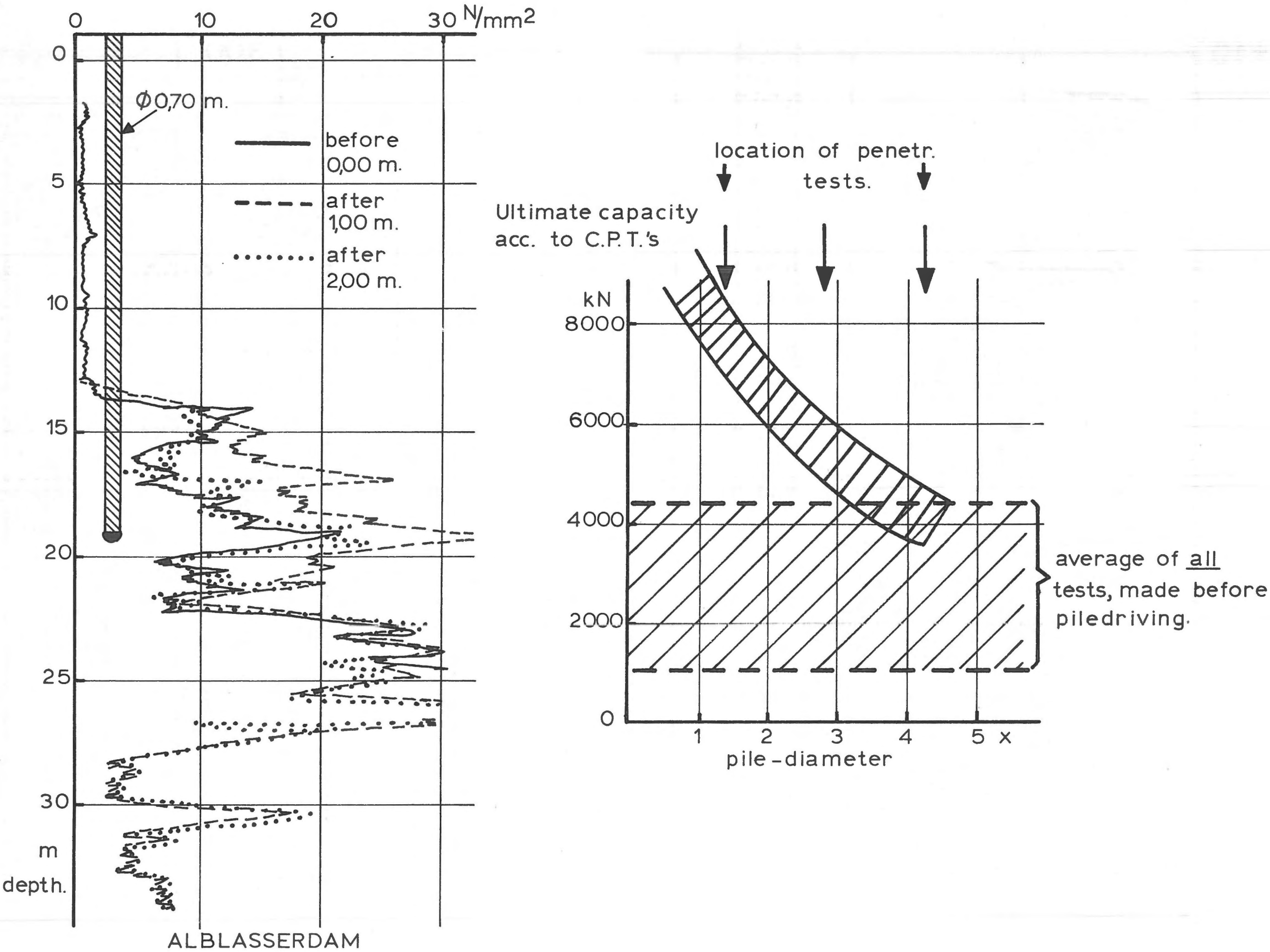

Fig. 10
Increase in stress level due to the installation of a driven pile, as recorded by cone penetration tests. At the right the increase in pile-bearing capacity is given as a consequence of the increased soil strength.

A much more effective friction pile is a pile with a wedge shape, like the well-known timber piles driven with their top ends down. Such piles create lateral soil displacement over their entire length during driving. This lateral displacement automatically leads to the maximum possible total stress perpendicular to the pile's surface and thus to maximum friction, especially in sandy subsoils where consolidation as a function of time does not occur. Remoulding of the soil is adequately compensated.

During the actual use of a wedge-shaped pile as a friction pile under tension, one would expect an adverse effect of the pile shape. However, the load which should be allowable, must of course lead to only limited pile deformations, so that the wedge shape cannot be considered to be disadvantageous in this respect.

For steel H-piles or open-ended steel tubular piles the amount of soil to be displaced by the pile is very limited. Therefore the lateral loads exerted by the displaced soil against the pile shaft are much less than for massive concrete piles or closed pipe piles. Thus the skin friction mobilized will also be substantially less for such open piles.

For bored piles the usual pile has an equal cross section over its entire length. There are, however, piles with underreamed bases. Such underreams will affect the existing stress distribution in the surrounding soil to a greater extent than the standard pile, so that the gain in bearing capacity is certainly less than one should expect. For impervious clays these foot extensions will be the most advantageous (RIZKALLAH, 1973).

INSTALLATION SEQUENCE

The above considerations are only valid for single isolated piles. A foundation will, however, make use of a number of piles and the installation of a pile has therefore also its influence on adjacent piles, installed before. A displacement pile will force aside the soil and increase the lateral effective stresses. This influence is larger at close distances from the pile and decreases with increasing distance. Also in this case

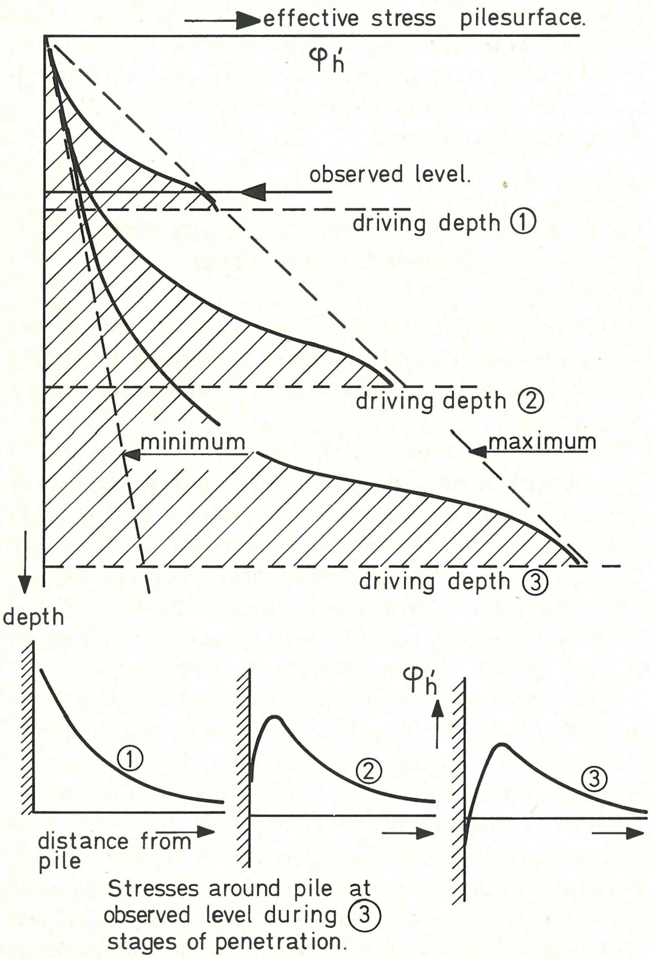

Fig. 11
Confining effective stresses around a driven pile at various levels during pile penetration.

shear stresses will be developed, but now in horizontal planes in front of the penetrating pile. The larger the number of piles, the further away their influence will be noticeable. Piles in position will experience larger lateral effective stresses against their front faces looking from the side of the piles being installed. Now the installed piles do not undergo any axial strain, so that any stress increase around their shafts will have a permanent character. The friction along such a pile is therefore very positively influenced by the driving of one or more neighbouring piles.

With bored piles the reversed effect must be expected, because the stress release generated by the boring operation will decrease the stress level around any existing pile within its sphere of influence.

FRICTION ALONG OPEN PIPE PILES

To illustrate even more clearly that stresses and deformations should be brought in harmony with each other in order to arrive at proper conclusions, one may take the following example of an open-ended pipe pile as used at a large scale in the North Sea for steel jacket-type structures.

It is customary in the pile design to assume that the friction between pile and soil at the pile's inside and the pile's outside is equal. At first sight this assumption sounds reasonable. Soil profile and pile surface are at least identical at both sides. Nevertheless this assumption is totally wrong, as will be explained now. Frictional loads exerted by the pile's outer surface are distributed into the ground surrounding the pile through shear stress generation. For long piles, shear stresses are able to transfer the frictional loads over substandital distances sideways from the pile. So the increase in stress level immediately around the pile is not very large.

Inside the pile, the situation is totally different. The frictional forces can only remain in the soil core which has a very limited cross section, so that the increase in stress level there will always be very substantial in case we assume the same friction per unit of surface as at the outside. This high stress level would result in substantial compression of the soil core and that in its turn would result in negative friction instead of positive friction. Such a contradiction is, of course, impossible. The inside friction will actually remain very limited, because the soil core must be compressed to a slightly smaller extent than the pile itself in order that the generated friction is still a positive one. As the allowable pile deformations are small it is a condition that the core compression is kept even a little smaller and this can only be obtained if the additional load taken by the core is also small.

Friction inside a pipe pile is therefore of an entirely different nature than outside. Inside for instance a plug can be formed at the tube's lower end. If that is the case, arching occurs which leads to extremely large effective stresses perpendicular to the pile's surface over a short length of the pile and thus to locally very large frictional forces. If this happens it can only be permanent if consolidation under the increased stress level cannot take place, so that this is applicable to granular soils in which excess porewater pressures do not occur. In clays long-term plugging cannot take place. The best way to prevent plugging is to aim at a low stress level inside the pipe which can be obtained by an increase in diametre of the tube shortly after the soil has entered. Another approach can be to decrease the friction between the tube's inside and the soil. This has been done successfully by injecting a bentonite slurry where the tube increases in diametre (Fig. 12). Experience has shown that the formation of a plug is a random effect which for one pile takes place and for the other not at all. It has been impossible so far to predict such a plugging behaviour. This effect can be fully compared with silos where also silo action shows usually a random effect.

FRICTION ALONG BORED PILES

It has already been mentioned that the relatively short period

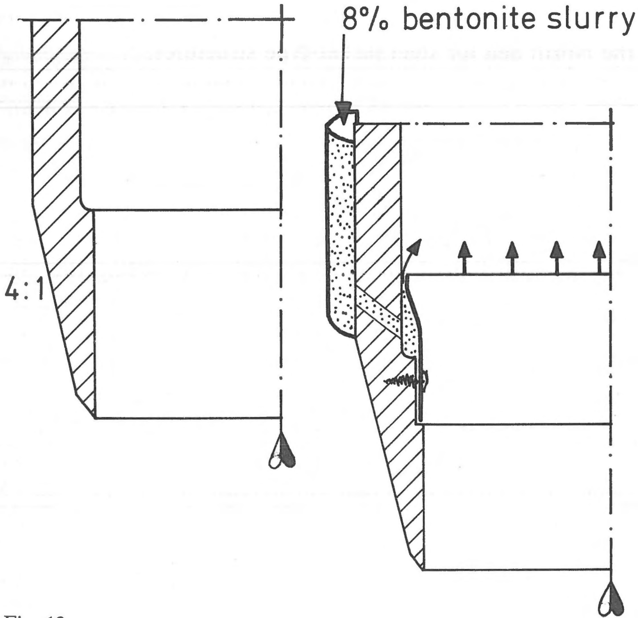

Fig. 12
Underside of open steel pipe pile with measures to decrease plugging effects.

during which the bore hole stands open while its walls are stabilized with a rather light bentonite slurry, a stress release in the surrounding soil takes place. In The Netherlands, where the bearing stratum is nearly always formed by Pleistocene sand deposits, this stress release is an immediate one. This sequence is followed by that of filling the bore hole with tremied concrete of high plasticity. The concrete with a unit weight of 24 kN/m^3 builds up a larger horizontal effective stress, as the sides of the hole act as the form work for the concrete. Actual measurements have shown that the total stress in the fresh concrete during placement is less than that according to a hydrostatic distribution. The rate of casting is slow, drainage of the surplus amount of water from the concrete into the surrounding soil leads quickly to increasing shear resistance in the concrete, while also the reinforcing cage may increase the internal friction. These factors contribute all to lateral concrete pressures from the pile onto the side walls which sometimes can be substantially lower than the hydrostatic ones.

These lateral concrete stresses are balanced by the groundwater pressures (which are known in magnitude) and the horizontal effective stresses (which can then be assessed). Based on these actually final horizontal effective stresses, the side friction can be computed. It will be clear that the friction values thus arrived at, have very little to do with the original soil conditions and stress level before the boring operations, but everything with the pile-installation process.

Did the bore hole stand open during a full night? Was the concreting interrupted for one or two hours? Such events have considerable effect on the amount of friction, the bored pile will be able to mobilize. An experienced crew is of great importance for the actual behaviour of the bored pile under its load. These factors get insufficient attention so far, but are more than worth obtaining it. As has been mentioned before, bored piles in clays are much less sensitive to their installation cycle as the effective stresses in the clay will only slowly change with time.

INCREASING THE STRESS LEVEL AROUND A PILE AFTER INSTALLATION

It has been explained that a high stress level around the pile will generate more skin friction than a low stress level in the same bearing stratum. The question is: how can we create a K maximum?

This approach actually is the reason for the success of grouted anchors or tie backs. For such anchors steel bars or bundles of steel wires are used while cement grout under high pressure is pumped between these and the soil. The result is a frictional force per unit of surface ranging between 500 and 1000 kN/m^2. Such a friction is a 10-fold of the friction generated by displacement-type of piles, so that apparently there is plenty of room for improvements in the field.

Such improvements already have occasionally been applied in the past. Large bored piles under two bridges in Argentine were grouted some time after they had been installed. Load tests before and after did show a drastic increase in bearing capacity (MORETTO, 1977). *This leads to the question why we are making it ourselves so difficult with pile installation by driving, vibrating and the like. Why should we not be able to place our piles in the easiest way possible and then grout a mantle around that part of the pile that reaches in the bearing layer?* Cement grouts do not penetrate into sandy soils, so that these are ideally fit to expand the pile diametre in such soils, thus mobilizing the largest possible frictional forces. Such an approach would certainly be more friendly for the pile itself, but also for the crews installing the piles and actually for the whole environment.

The high stress level can, of course, only be relied upon in case this level has a permanent nature. Therefore the proposed working method is to be limited to application in granular soils only, while even in such soils a new deformation of the stressed grain skeleton must be avoided. Deformations could result from a severe earthquake or from a deep excavation nearby or even from pile driving at short distances. Vibrations due to traffic cannot be considered strong enough to have such an adverse effect.

PILE BEHAVIOUR UNDER LOADING

It now is generally accepted that the displacements required to generate the full frictional resistance along the pile shaft, do not exceed 10-20 mm regardless the soil condition and regardless the actual pile size (Fig. 13). The deformation required to generate the full endbearing capacity depends on

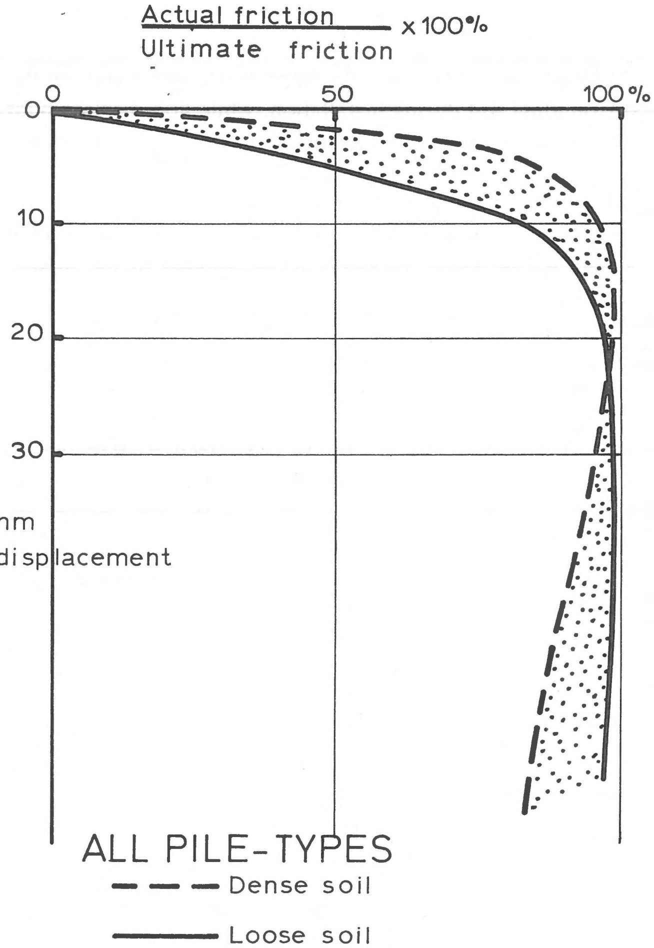

Fig. 13
Average friction-displacement diagram for foundation piles.

the size of the foot and on the degree of precompression of the soil directly underneath the foot. For impact-driven piles, the precompression is more than that for piles which were driven by a vibrator. For bored piles there is no precompression at all, but usually there is a decompression that develops in the period between the completion of the boring process and the beginning of the concrete placement.

In figure 14 different load-settlement curves have been given for the pile foot of driven piles, augered piles and large-diametre bored piles. As the observed settlements are always a linear function of the pile foot diametre the deformations have been plotted on the vertical axis as a percentage of this pile diametre. The results are then applicable for any diametre, within the range of practical experience. Using both graphs for a given pile learns the following:

(1) Driven timber piles with a tip diametre of 100-150 mm, as used extensively throughout The Netherlands (1977: 450,000 piles), react very stiff. The pointbearing capacity is mobilized after approx. 10-15 mm tip settlement which means that the full skin friction has then been generated as well. In compression and in tension the pile deformation will be more or less equal, although the failure load under tension will, of

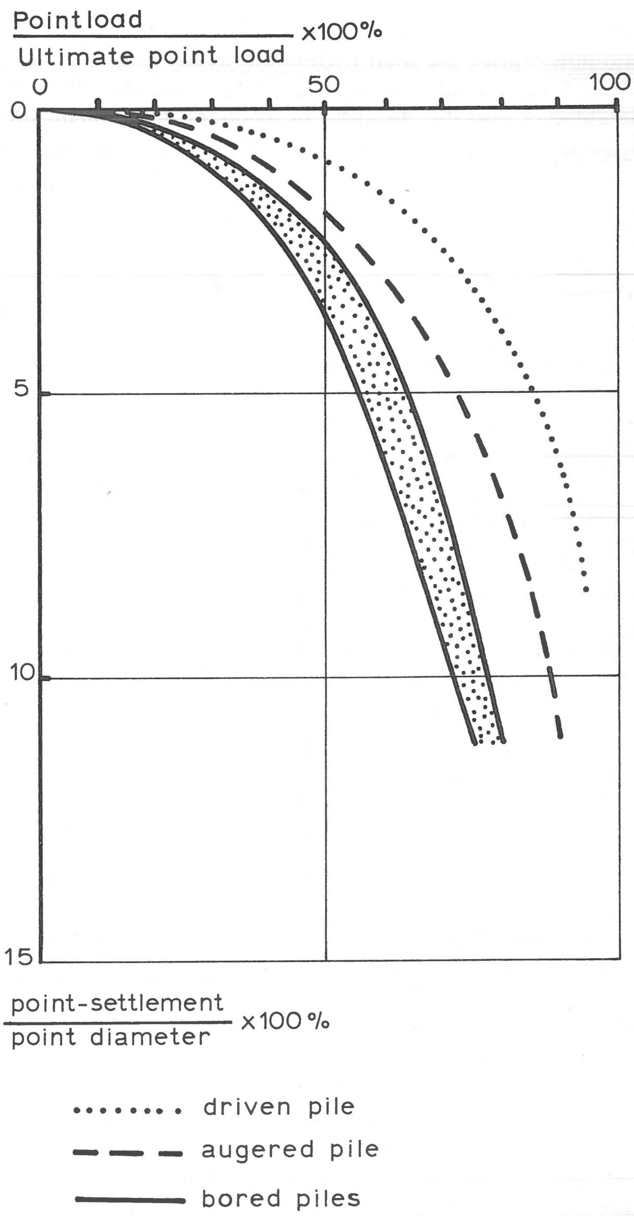

Fig. 14.
Load-settlement diagrams for the foots of seceral types of foundation piles.

course be smaller than that under compression. A factor of safety used against failure, being usually 1.7, wil apply to both skin friction and endbearing at the same time and to the same extent: generating the allowable pile load means a pile foot load of 60% of the ultimate value to which belongs a settlement of approx. 1.7% of the pile foot diametre which equals approx. 2.0 mm. The average shaft settlement in the bearing layer will therefore be 2.0-3.0 mm which also leads to approx. 60% of the ultimate value of the friction. A safely designed foundation with timber piles will therefore limit the building settlements to well under 10 mm!

(2) Large-diametre bored piles with a diametre of 1.00 m will

behave entirely different. When the pile foot has moved 10 mm downwards, the shaft friction has nearly fully been generated, but the pile foot still carries not more than 30% of its ultimate capacity. In order to generate the ultimate tip capacity a settlement well in excess of 100 mm should occur. In order to limit the settlement to the same maximum of 10 mm as for the timber piles, the foot load is to be limited to only 30% of the ultimate value. In that case the factor of safety is 3.3. But a settlement of the pile foot of 10 mm includes automatically nearly full exploitation of the maximum available skin friction, so that the factor of safety therefore is close to 1.00. Actually a single factor of safety for the pile as a whole cannot be applied to the same extent in skin friction and endbearing. Actually for large-diametre bored piles only the endbearing shows a margin; not so the skin friction!

(3) Augered piles lay with their behaviour in between both other pile types.

These observations should always be borne in mind when tests are carried out with small-sized piles, or model piles, in order to investigate their behaviour under loading. The differences in behaviour between such small-sized piles and actual piles may be substantial.

LATERAL SUPPORT

A phenomenon that gets little attention from researchers in the field is the lateral support that piles obtain from the soil surrounding the piles.

Under axial loading, piles always tend to move sideways. This effect occurs for the foundation as a whole into one and the same direction, unless the pile foundation obtains extra stability from groups of inclined piles or from a basement under the structure. The leaning of the piles in one direction will increase the effective lateral resistance at the face, but will decrease it at the opposite side. For a given lateral deformation, the increase in lateral pressure usually is larger than the simultaneously resulting decrease at the opposite side, so that this effect will lead to an increased axial skin friction. Everybody knows this from his own experience with the extraction of poles from the ground. Axial pulling leads in an easier way to the desired result than bending and pulling.

Apart from this leaning effect, most small-diametre piles will also show initial buckling effects under compressive loading, but the lateral support of the soil prevents this process from fully developing. Nevertheless the pile shaft will generate a lateral support of the soil that is according to a sine distribution in axial direction. As the elastic line of the pile need not stay in a single plane, a screw-like form is the most likely. This effect adds also to the average friction especially in the top layers where both aspects work together.

CONCLUSIONS

(1) Displacement type of piles improve the stress state in the surrounding soil during installation, while non-displacement type of piles will have an adverse effect.
(2) A foundation on many small-diametre piles will show less deformation than an alternative foundation on a small number of large-diametre piles. When an equal deformation is aimed at, the factor of safety to be applied to the larger-diametre piles should be substantially larger.
(3) Driven piles improve the stress state in the soil around their lower ends most, but increasing pile penetration during installation deteriorates this good result progressively.
(4) There seems to be room for increasing the stress level around the pile section in the bearing stratum after pile installation. Such an approach would enable pile installation techniques which are more friendly to the pile and its environment than the techniques usually applied nowadays.
(5) Theories for the prediction of pile capacity do not take into consideration in a sufficient manner the way piles are installed.
(6) Pile-installation techniques do not exploit the possibilities to improve pile-capacities.
(7) Researchers approach the problem of pile capacities as if this is a matter of arithmetics without studying in detail what really happens with the pile and the ground.
(8) The predetermination of pile capacities will require a lot more attention in the future. Using our knowledge about the geology of the underground, about the installation techniques to be applied and about the pile shape and the pile material, needs more attention than hitherto.
(9) Friction along the inside of open ended pipe piles has nothing to do with the friction along the outside. Plugging or not plugging is a random phenomenon.

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