For the wooden tapered pile was here found $\alpha = 1{,}0$, $\beta = 2{,}4$ and $P_u = P_E$, when the Rausch or the differens formula (given in Fig. 5) was used. If the formula $P_u = \frac{n \cdot K_3}{K_1} \cdot P_E$ is used, $n = 1$ as in pure sand $K_3 = K_1$.

CONCLUSION.

As a main result of this investigation was found, that the use of caissons would involve settlements of abt. 60 cm in 20 Years, whereas an anchored concrete flat slab resting on vertical wooden piles only would settle slightly, when driven down to the glacial clay with high point resistance.

Further was found that the Yoldia clay was especially favourable in giving a high friction for a minimum of driving. For a given pile resistance, $P_u$, tapered wooden piles require less driving effect ($P_E = P_u$: 2,6) than for untapered concrete piles ($P_E = P_u$ : 1,7), but generally the friction is halved during driving and full strength regained after few days (average $P_u = 2{,}1\ P_E$ ).

The uniform and homogeneous Yoldia clay was especially suitable for predetermination of the pile resistance based on loading tests borings, but also quick field sample test gave a reliable predetermination of both static resistance (2,1 $P_E$) and dynamic ($P_E$) resistance.

In glacial chalky clay was the predetermination of the friction less safe due to soft spots, but the point resistance is in agreement with the loading test borings. Generally the Rausch pile driving formula is found valid in sand and in sandy or chalky glacial clay but may in Yoldia clay be multiplied by $K_3/K_1$ = 2.1 in average.

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VII a 3

# SOME PILE DRIVING PROBLEMS

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In the Proceedings of the First International Conference on Soil Mechanics a description was given (1-1) of the method, nowadays used in the Netherlands, of determining the length and the bearing capacity of foundation piles by means of soundings. Moreover, in report 1-2 point resistances and frictional resistances, resulting from such a sounding, have been compared with those resulting from a large number of pile loading tests (loading and pulling tests on piles of various dimensions).

This method offers great advantages above the customary methods of test borings or the driving of test piles, which are not discussed here. Moreover the costs are considerably lower than those of loading tests. This explains why this method has come into general use in this country, so much so that up to date more than 5000 soundings have been performed.About the improvements of the apparatus during the last years and about the method of sounding under water, see the companian paper of Vermeiden (III b 7 ).

In some cases it is very simple to predetermine the desired location of the point of the pile and the safe point load from the results of soundings, especially when very weak clay and peat layers are underlain by a sandlayer of considerable penetration resistance (fig. 1a). When the ground surface has recently been raised or when this still must be done prior to construction, negative friction must be accounted for in the determination of the safe point load. Its magnitude can be estimated on the basis of the measured frictional resistances, although these cannot be rigorously applied to the driven pile, because in sandlayers an increased density and hence an increased friction may be expected, whereas in clay layers the structure may be disturbed and friction may decrease.

When the results of the sounding are not so straightforward as shown for instance in fig. 1b and 1c, a certain amount of scientific insight is required to determine the best or the most economical loaction of the point and the corresponding point resistance. Tests,performed in conjunction with the Bridge Bureau of the Rijkswaterstaat, at Alblasserdam,and the results of several other test loadings have considerably improved our insight in these matters.

At Alblasserdam the piles were driven exactly at the place where soundings had been taken before, and after driving and test loading another sounding was taken. From the comparison of 6 similar tests it was concluded that an increased density of sandlayers under the point, resulting from the driving, could only be ascertained over a very slight depth. It follows, that the sounding resistances, found at some distance under the future location of the point, determine the safe point resistance. Fig. 2 shows the results of a sounding together with the driving diagram of a pile, and the point load resulting from a loading test. Evidently the bearing capacity at the point is already influenced by the weak layers one meter below.

At pile driving works where the location of the point and the safe point load are predetermined in this way, practical difficulties often arise. These may be caused either by the heterogeneous nature of the soil or by the rigid adherence to a specified penetration per blow. These two causes are closely interrelated. It is obvious that the costs of soundings preclude an investigation by means of a sounding at every pile. Therefore, in order to take account of the heterogeneous nature of the soil and to acquire an insight into the soil conditions from place to place, the average penetration at the last series of blows is registered.

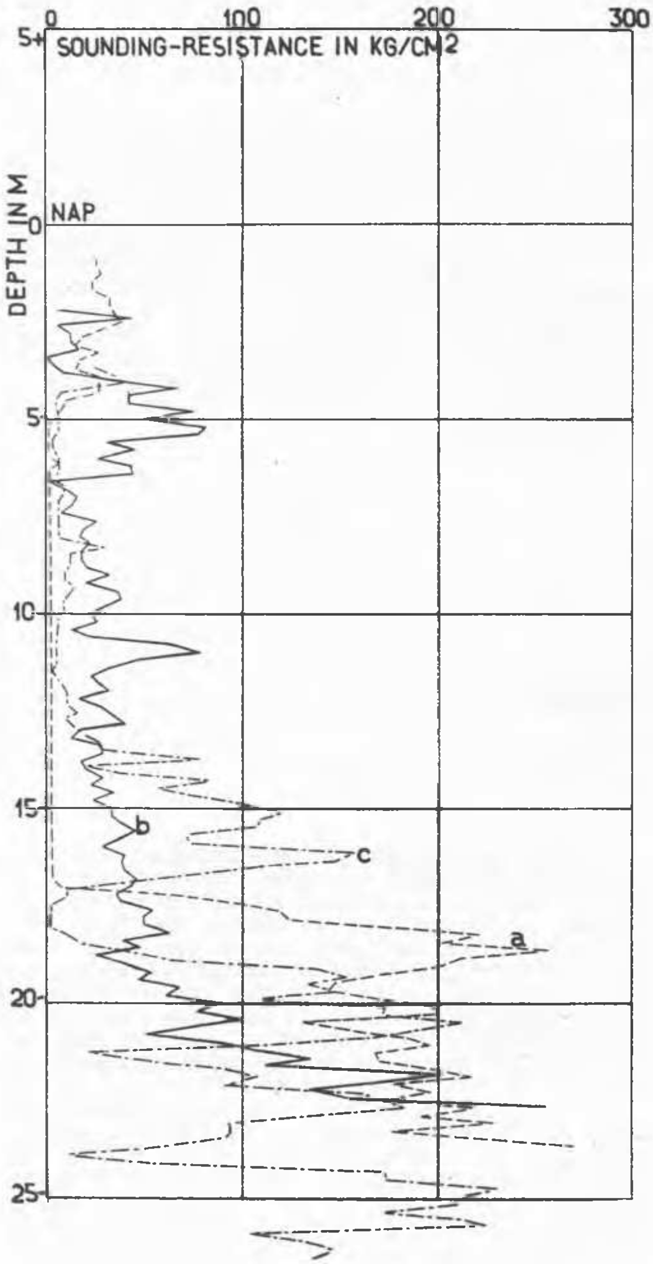
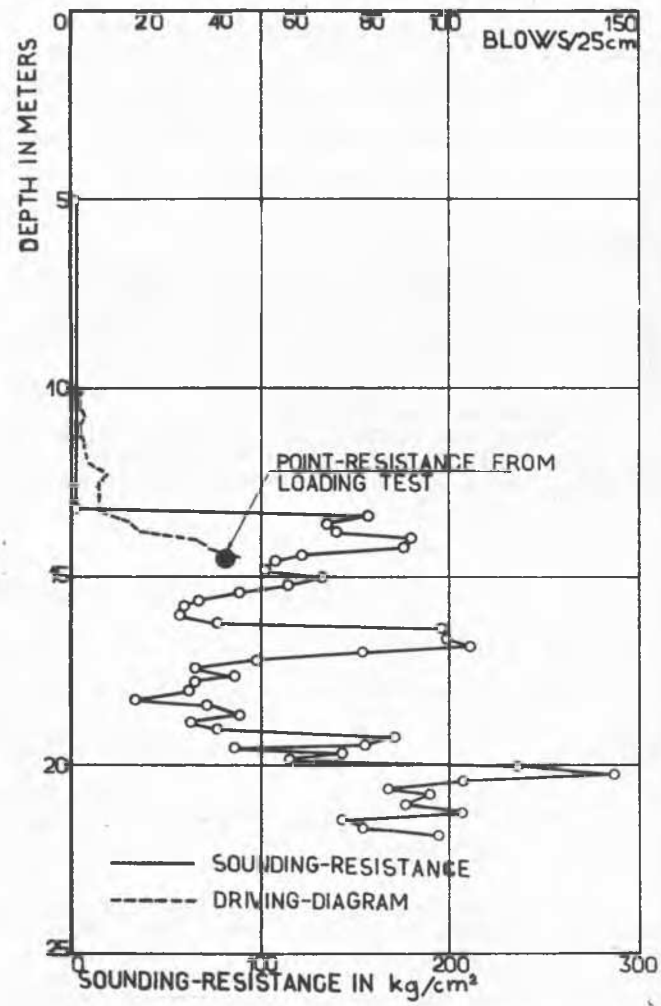



FIG.1




Point resistance for pile No 4-Alblasserdam-in comparison with sounding results.

FIG.2

That is why the local Building Inspection often specifies for piles of given dimensions and with a specified bearing capacity, a certain maximum penetration based on pile driving formulas.

It is well known that pile driving formulas depend on such a variety of factors, that on the whole it is not very well feasible to determine the safe load with the aid of such formulas. Many writers have expressed themselves to this effect. Nevertheless, to assess the relative value of the resistance of the soil, the method of the average penetration per blow is generally applied, the more so as there does not seem to exist another economical method to attain the same end. As a result the piles are mostly driven till a specified penetration is obtained and, as this penetration not only depends on the point resistance, but on the frictional resistance along the surface as well, this means that, especially in areas where the sounding resistance varies, the point may find itself within a layer of slight resistance. Moreover, as we shall show below, the penetration depends also on various other factors and therefore it is not a reliable starting point. In such a case then, the preliminary soundings have not served their purpose, the piles may have to be extended by extension pieces, or, if longer piles have been brought on the site, a large proportion of them will be left sticking out of the ground.

An example is furnished by a work at Rotterdam. For the design of the foundation of a structure 5 soundings were made, the results of which are shown in fig. 3. These show that the soil was fairly homogeneous, except at sounding no. 3 where a lower resistance was found. Down to a depth of 15 to 16 m - NAP weak layers of low resistance were encountered and under these a sandmass where the piles would have to find their resistance. Based on these results it was proposed to place the points of the piles at about 17 m - NAP and to allow a point bearing pressure of 30 kg/cm². It was also noted that the very rigid layer at about 25 m - NAP would be difficult to reach with ordinary means, as evidenced by the total registered sounding resistance (point resistance + friction) of more than 5000 kg.

On the basis of these data it was decided

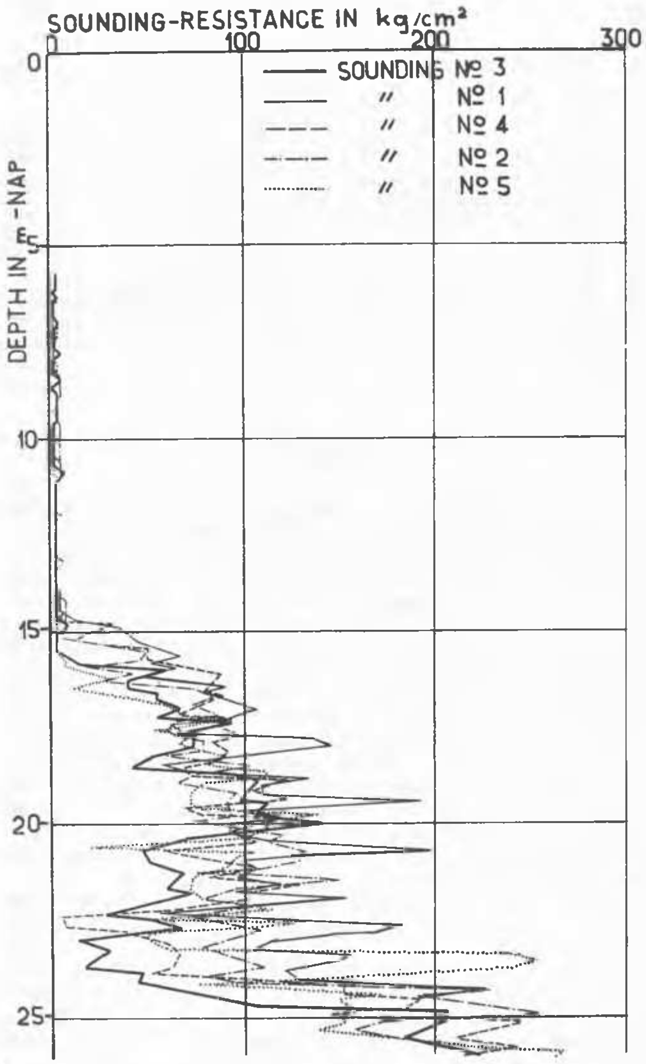
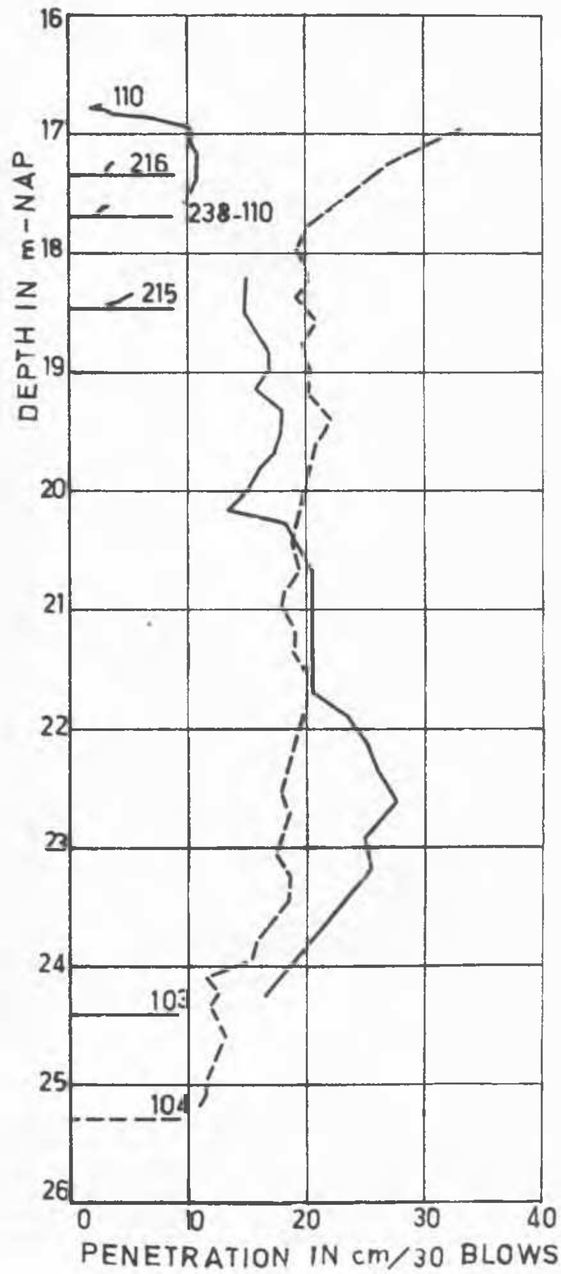



FIG.3




Driving diagrams for piles No 110, 216, 238, 215, 103, 104.

FIG.4

to use "Arkel" piles (hollow round centrifuged piles), 40 cm in external diameter, with a point enlarged to 60 cm diameter over a length of 1 meter. These piles were designed for a safe load of 60 tons, so that the pressure at the point worked out at only 20 kg/cm$^2$, leaving a large margin of safety, the more so as the excavation for a large cellar under the structure would counteract any negative friction by relieving the ground pressure.

The driving was done by means of a 4000 kg steam hammer (weight of the moving parts) in a heavy steel Dutch pile driving frame. The Building Inspection specified that for a maximum load of 65 ton, the penetration, with a height of fall of the hammer of about 60 cm, should be not more than 8 cm at the last 30 blows. Where an extension piece on the pile was used, this figure was reduced to 4,5 to 5 cm per 30 blows, because of a certain loss of energy.

At first the work proceeded favourably. Penetrations per 30 blows were registered of 0,7 to 4,5 cm for pile lengths of 10,5 and 11 m. The points came to rest at about 16,50 m to 17,50 m - NAP.

The difficulties started from pile no. 92 after which the piles had to be driven to below 19 and 20 m - NAP. Very unfavourable results were obtained at piles nos. 103 and 104, which were driven to 24,40 and 25,30 m - NAP.

In fig. 4 the driving diagrams of these 2 piles are shown together with some diagrams of other piles. Because it was suspected that this deterioration was caused by the local soil conditions, it was decided to carry out another 4 soundings (nos. 6 - 9) and one boring (see fig. 5). Sounding no. 6 was worse than the previous ones, no. 7 gave better results, whereas no.8 and 9 were similar to the previous soundings. The boring showed that the layer, where the points were located, consisted wholly of sand, growing coarser with the depth. Afterwards the size distribution curve and the permeability of the 4 samples were also determined (fig.6.)

Various possibilities were considered in order to find an explanation for this sudden change in driving results but no acceptable solution presented itself. The pile driving

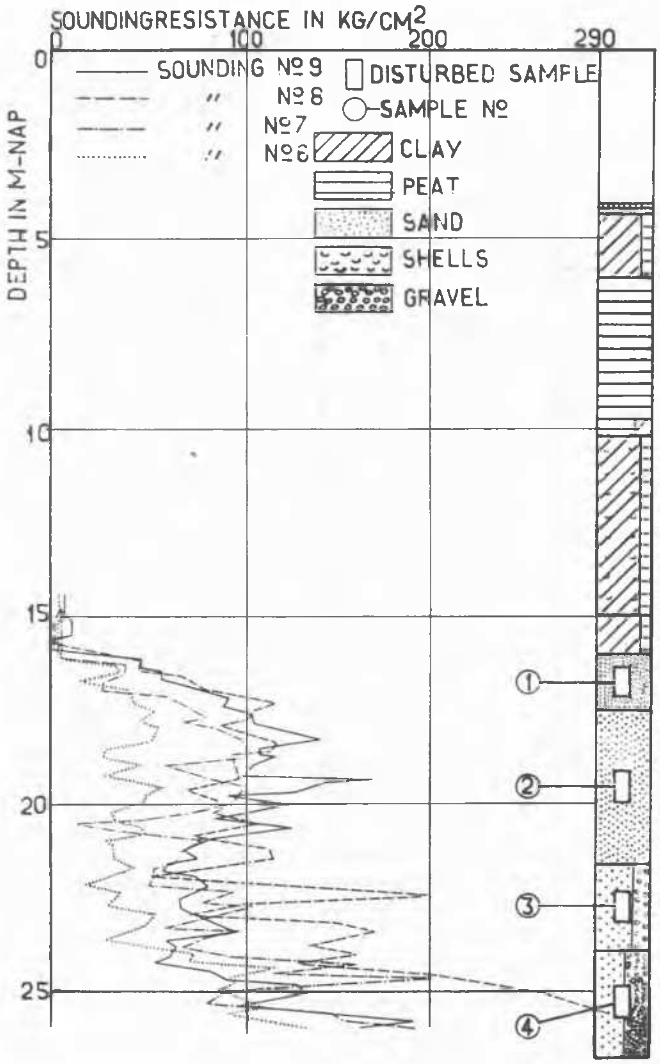
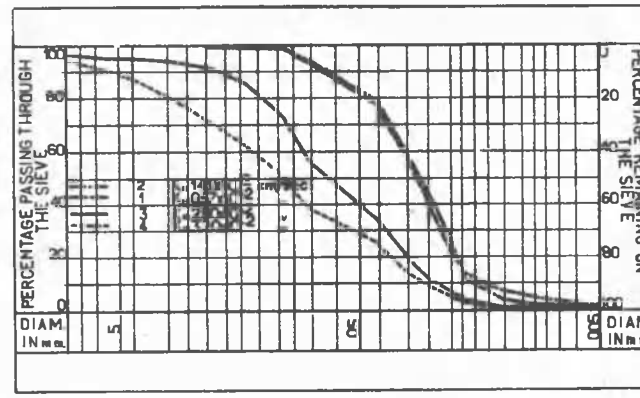



FIG.5




Particle size summation curves.

FIG.6

was continued and the results of the following 5 piles were still very bad, but after that the resistance in the upper layers improved and the pile driving could be completed, be it with some piles added. Fig. 7 shows the progress of the pile driving and the final location of the point.

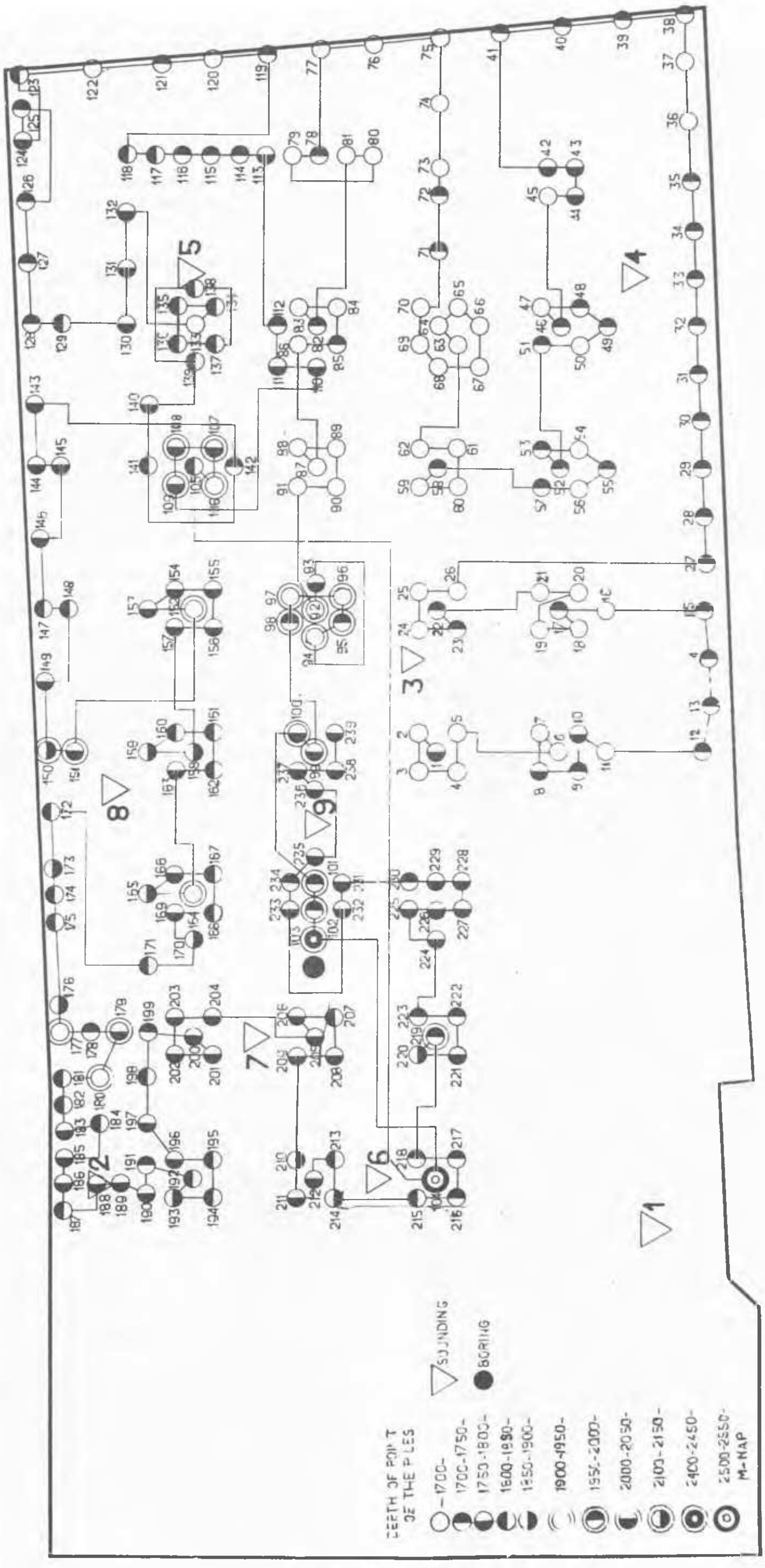
For piles with an area at the point of about 2800 cm$^2$ and with a load of 60 tons the average point pressure is about 22 kg/cm$^2$. It can be seen that even for the most unfavourable sounding (no. 3) this would mean a safety factor of 2 at a depth of 17½ m, that is, if no account has been taken of the friction along the pile surface. Therefore the difficulties arise out of the adherence to the specification of the Building Inspection, requiring a definite penetration per blow.

Now what happens during the driving of a pile? As the hammer hits the pile head, the pile receives an impact and is pushed into the ground. For this to occur, it is necessary that a certain volume of soil particles with water is displaced. These soil particles will tend to bring about an increased density of the surrounding sandmass. What it amounts to is, that a volume of water must be displaced, equal to the volume of the pile as far as it is driven into the sand. Excess pore water pressure will result. Consequently the water will flow towards regions of less pressure but, before that can happen, dependent on the permeability of the soil layers a decrease of the effective pressure in the sand takes place, because the weight of the layers above remains almost the same - apart from the friction forces along the pile, which are transferred to the adjacent soil - and this in its turn causes a decrease of shear strength and hence of penetration resistance. Thus high pore water pressures give rise to low penetration resistance i.e. unfavourable set per blow (other conditions being equal e.g. same friction along pile in cohesive layers above, same hammer weight, height of fall, etc.) There is a limit to the rise of water pressures viz. where they become equal to the weight of the soil mass above, plus the friction forces acting upon this soil mass via the pile surface. Beyond this limit the soil mass is lifted.

The water pressures will rise higher and the penetration per blow will be more in direct proportion with the speed of the pile driving progress and with the impermeability and hence with the fineness of the sandlayers. This has been observed many times in practice. These water pressures have been measured in 1939 at a structure at Rotterdam in the upper cohesive layers (fig. 8). The meters were placed just outside the work site and excess pressures of 5 atmospheres were observed during the pile driving. It is to be noted that these pressures drop relatively soon. This can be explained by assuming an expansion of the soil as a result of the decrease of effective pressure, as it is known that even a very slight drop in moisture content can cause any excess pressure to disappear.

At this particular pile driving work excess water pressures in the sandlayer have not been measured. Recently, however, the City Engineer's Department of Rotterdam have measured excess water pressures in sandlayers as well (see report) VII of ir. G. Plantema) .

As soon as the water pressures increase in the neighbourhood of the pile during driving, the water will tend to flow in several directions at the same time. In layers of coarse sand and gravel this stage will last only a short time. Thus, after a period of rest the resistance is bound to increase, as is very well known. In layers of fine sand or clay this stage will take some time, and, to have the same effect, the period of rest will have to be so much longer (see fig. 8). In view of these considerations it seems certainly acceptable that in such fine sandlayers the resistance can be so low. It might even be expected




Pile driving plan.
FIG.7

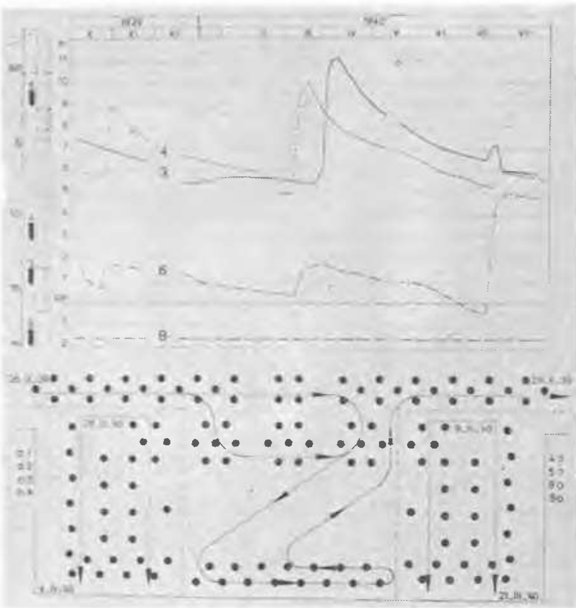


FIG.8

that in fine sandlayers the flow is so slight, that the excess water pressure, caused during the driving of the first piles, will still be present during the driving of the following piles, resulting in a gradually increasing pressure during construction and hence in a gradual drop in the penetration resistance with time. Therefore, in the case of the pile driving work mentioned above, it was suggested, that this was the cause that the penetration resistance dropped off after the 91st pile, but if this were so, then it is still hard to explain why it was still so low at piles nos. 105 to 109, where very unfavourable penetration resistance was observed after a rest period of 15 days, whereas the following piles gave more favourable results.

Personally I feel that, what we have observed here, may be associated with the phenomena we know from the investigations into the critical density. It is known that loosely packed sands, when subjected to shearing stresses, acquire a higher density, while excess water is pressed out and that thereby temporarily also the shearing resistance drops appreciably. Closely packed sands on the contrary increase their volume under the influence of shearing stresses, while water is attracted and thereby the shearing resistance rises.

When we re-examine, in the light of these phenomena, what happens during the driving of a pile into sandlayers, then we can expect increased density in loosely packed sands, simultaneously with the occurrence of excess water pressure. The amount of this pressure depends in the first instance on the dimensions of the pile, on the number of piles per surface area, on the speed of the pile driving progress, and on the density and permeability of the soil. The density of the soil changes by previous pile driving in the proximity. The volume of the water to be pressed out is equal to the volume of the part of the pile in the sand. This excess water pressure causes the effective pressures (and hence the shearing resistances) to drop, so that a low penetration resistance is registered, which is by no means a measure of the bearing capacity of the pile. After a period of rest, when the excess pressure has disappeared, and the effective pressures and thereby the shearing resistances have been restored, the pile will have obtained an increased bearing capacity. It may be feasible during the pile driving to drain the excess water by pumping or by suction, either by means of a well, or by means of an arrangement in the pile itself, in order thus to get a better penetration resistance during the driving. In that case the registered resistance would correspond better with the ultimate bearing capacity of the pile.

On the contrary, in closely packed sands, the occurrence of shearing stresses will cause a volume increase i.e. a looser structure of the sandmass. The ground surface may consequently rise, and the gain in volume thus obtained may be greater than the volume of the displacement of the pile in the sand. For not only the expansion of the displaced sand but also the expansion of the sand round the pile which is subjected to shearing stresses, will contribute to the ultimate increase of volume. Excess water pressure cannot occur. On the contrary, "negative" water pressure can be expected, with a correspondingly greater effective grain pressure and hence an increased registered penetration resistance.

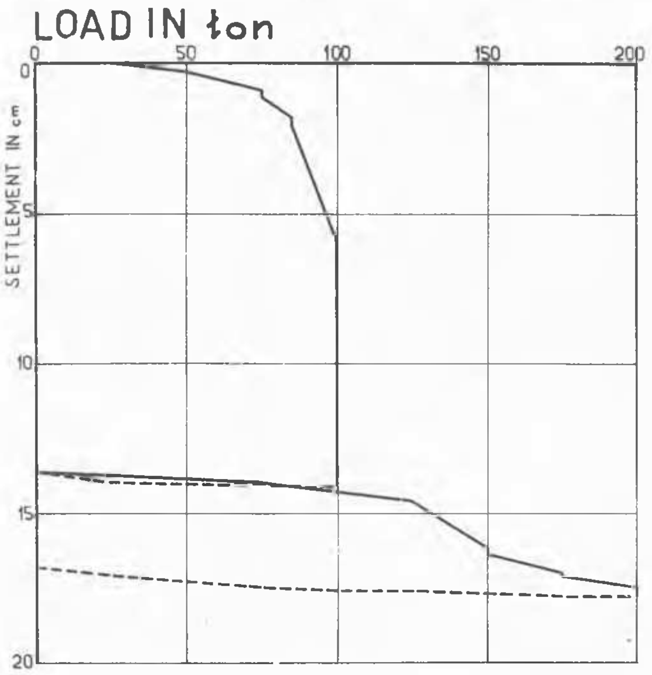
In actual practice one can expect every transition phase between these two cases as in general all soils are very heterogeneous, both in horizontal and in vertical direction, and moreover by the effects of previous pile driving in close proximity. This is demonstrated in pile driving works by the considerable difference in depths reached by contiguous piles. It is interesting to note that the excess pressures measured by the City Engineer's Department at Rotterdam decreased again during the driving from a certain depth downward. From this point the registered penetration resistance should therefore improve.

The conclusions to be drawn from the above considerations are the following:

1) Pile driving formulas based on a certain definite penetration per blow cannot provide a measure for the bearing capacity of piles driven into sandlayers.
2) Pile driving in weak cohesive layers and in loosely packed sandlayers causes excess pore water pressure and thereby a temporary considerable drop in penetration resistance of the soil. After some time, as soon as the excess pressure has disappeared, the penetration resistance rises again.
3) Pile driving in closely packed sandlayers causes an increase of volume, a decrease of pore water pressure and thereby a temporary rise of penetration resistance.
4) Other conditions being equal, driving in loosely packed sandlayers results in an unfavourable penetration resistance, while in closely packed sandlayers a very favourable one is obtained.
5) Therefore it is required to get some foreknowledge of the densities of sandlayers in which foundation piles will find their bearing and to this end soundings are as yet the most suitable means.

At some other works, where, for the sake of economy, the piles were driven down to the sandmass but not penetrating within it, other difficulties were experienced. It appeared there that, during the driving of piles in the proximity of previously driven piles, the latter were lifted, sometimes over 10 cm. What could be the cause? Three possibilities were considered viz.:

1) Lifting is caused by the displacement of the soil in the weak upper clay and peat layers.




Load settlement curve

FIG.9

The previously driven piles are subjected to additional frictional forces in an upward direction. Because they are not driven to within the sandlayers, they can be lifted easily with the adjacent soil if the friction exceeds the weight of the pile; they will now be free from the rigid foundation layer and their bearing capacity will be based solely on friction.

2) As a result of increased pore water pressure the clay and peat layers will expand. Expansion may also occur, owing to a decreased load e.g. when excavating a site for the building of a cellar. The expanding soil layers may lift the pile by means of the friction along the pile surface.

3) An increase of volume of closely packed sand, caused by the driving of the pile point into it, might cause a similar occurrence under previously driven piles. In this case the pile point will still be resting on the sand, which however, will get a looser packing.

At the pile driving work where the sounding of fig. 1a and the water pressures of fig. 8 were measured, these phenomena were more distinctly observed in a part of the work, where an excavation was made for a cellar. After completion of the driving all the piles were pressed down with a static load. An example of such a loading is given in fig. 9. At a load of 100 tons suddenly the pile started to subside quickly, but it recovered its support again at a level about 14 cm below.

From these observations it may be concluded, that possibility 3 needs not seriously be considered, because in that case the sudden subsidence under 100 tons would not have happened. Next, the fact that most of the lifted piles were situated in the deepest part of the excavation does not necessarily mean that only expansion may be the cause. Observations during a long period of the levels of the pile heads at such a pile driving work, will improve our insight into this phenomenon and lead to the right remedial measures.

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VII a 4

## EFFECT OF VIBRATION ON PILES IN LOOSE SAND

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SUMMARY.

Theoretically, piles supported in a mass of saturated loose sand should be subject to additional settlements from vibration and piles driven into such a mass should show small driving resistances because of reduced friction between pile and soil resulting from partial or complete liquefaction of the soil mass under vibration or sudden shearing forces. This theory has been proved by a series of tests made upon several types of friction piles supported in such a sand mass which were vibrated while carrying a static load.

A sand mass subjected to vibration tends to increase in density with a corresponding decrease in voids, the individual grains adjusting their positions downwards under the influence of gravity. In a mass of saturated sand below ground water level, such increase in density must be accompanied by expulsion from the mass of a portion of the water contained in the voids. Expulsion of this water results from a transitory increase in the Neutral Pressures and decrease in Effective Pressures. In a mass of very loose sand subjected to sudden shearing forces or to heavy vibration, the rate of volume change may be so great that in the area affected, the effective pressures may be reduced to zero, temporarily causing complete flotation of the sand grains, that is liquefaction of the soil mass.

Driving a pile into a mass of loose saturated sand sets up both sudden shearing forces and vibration, causing a decrease in volume of the sand mass. Theoretically, there should be a decrease in the Effective Pressures reducing the intergranular friction of the soil mass and the friction between the soil and the pile. Under such conditions, the pile may drive