SECTION VII

PILE FOUNDATIONS, PILE LOADING TESTS

GENERAL REPORT

J.L. KERISEL (France)

We recall that this section includes three sub-sections:

a) Settlement and bearing capacity of piles
b) Horizontal pressures on pile foundations
c) Special problems.

It is the first sub-section which has inspired the largest number of communications, 22 in all, from 9 nations; only three communications have been submitted in respect of sub-section b, of which two are from Holland; and finally, sub-section c has brought together 4 communications.

Sub-section a. SETTLEMENT AND BEARING CAPACITY OF PILES.

It is difficult to give an account here of the considerable amount of work accomplished by the authors of the numerous communications received: the very classical nature of the subject has not discouraged those taking part and on the whole the conclusions submitted enrich in a very substantial manner the already abundant data we possess on the matter.

The authors of the reports have felt the necessity of trying to get away from trial and error methods that have been in use for many centuries and if a few reports describe field methods, which incidentally are very interesting, (Use of long spliced piles - J.B.O. Hosking - Australia; Use of cone shaped pile of the Ragmond-pile type - Radzimir Pietkowski - Poland) the authors on the whole have sought to draw from the observed facts the deductions which will enable them to demonstrate conclusions with a general bearing.

The final aim is in every case the same: to determine the total bearing capacity of the pile. And for doing so there are three methods:

Deduce the bearing capacity.

1) From the penetration obtained by driving.
2) By direct loading of the pile.
3) From the physical characteristics of the soil.

1st. Method.

The question as to the value that should be placed on driving formulae is in truth one of the most hackneyed in the science of soil mechanics. The mathematical justification of each one of these formulae is part of the theory of shock between two bodies with or without interposition of a soft body. As the dissipation of energy is not directly measurable, the result is that each formula can only be justified by cross checking with the results of one of the two other methods, and particularly the direct load method.

Here are the main observations contributed in this field.

Mr. Lawrence B. Feagin (U.S.A.) notes that the test load required to produce a settlement of 0.02 ft. in the test pile exceeded the bearing value indicated by the Engineering News formula x); with few exceptions the piles were driven in deep deposits of sand, varying from fine to coarse.

Messrs. Gregory P. Tscheborarioff and L.A. Palmer think that a pile driving formula which gives a low value for dry sand gives too high a value for plastic clay.

In a fine alluvial sand mixed with shells and a little clay Messrs. L. Bjerrum, C.H.R. Ostenfeld and W. Jonson (Denmark) have noticed that Eytelwein's formula gives the most applicable results, while the Engineering News and Brix formulae give values 40 to 60% too low.

Without endeavouring to find out which among the empirical pile driving formulae is the best to apply to a special hammer and to special soil conditions Mr. T.K. Huizinga (Holland), Mr. Willaim F. Swiger (U.S.A.), Mr. C. Franx (Holland) and Mr. G. Plantema (Holland) all consider the same question: are pile driving formulae reliable? They are inclined, independently of each other and for the same reason, to answer this question in the negative.

Pile driving in weak cohesive layers and in loosely packed sand layers causes excess pore water pressure and thereby a, temporary, considerable drop in the penetration resistance of the soil. After some time, as soon as the excess pressure has disappeared, the penetration resistance rises again.

Thus, pile driving formulae based on a certain definite penetration per blow cannot provide a measure for the bearing capacity of piles driven into sand layers. (The City Engineers Department of Rotterdam has made some investigations on the excess pore water pressure. The water pressure in the sand layer was recorded by an electrically operating pore water pressure cell during the driving of several piles. - See reports of Mr. G. Plantema and Mr. C. Franx).

Divers other considerations or observations connected with the subject are added to this conclusion, which, to us, appears to be indisputable.

Mr. Huizinga feels that the disagreement between pile driving formulae and bearing test values is closely associated with the phenomena of critical density.

"It is known" he says "that loosely packed sands submitted to shearing stresses, ac-"quire a higher density, while excess water is " pressed out and that thereby, temporarily, "the shear strength also drops appreciably. "Closely packed sands, on the contrary, in-"crease their volume under the influence of "shear stresses, while water is attracted, "and thereby the shear strength increases.

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x) Engineering News Formula: $R = \frac{2\ Wh}{s + 0,1}$

Symbol	Definition	Unit
W	Weight of striking parts of hammer	Pounds
h	Height of fall of hammer	Feet
s	Penetration of pile per blow	Inches
R	Allowable static safe load on pile	Pounds

"This pile driving in closely packed sand "layers causes an increase of volume and ex- "cess pore water pressure, and thereby a tem- "porary increase of resistance to penetration."

It was therefore necessary for Mr. Huizinga to obtain some previous knowledge of the densities of sand layers after piles have been driven in.

We agree with Mr. Huizinga: the knowledge of the soils density after pile driving, is not only necessary for appreciating the degree of exactitude of this first method, but it is also necessary for the third method (calculation of the bearing capacity from the physical characteristics of the soil). Does the coefficient of apparent friction tg φ become equal to what M. Caquot x) calls the most probable friction coefficient $tg\varphi = \frac{\pi}{2} tg\psi$, $tg\psi$ being the friction coefficient of the material of which the grain is composed?

This question is answered to some extent by Mr. L. Bendel (Switzerland) who draws a comparison between the penetration of a sounding needle in the neighbourhood of a pile, before and after it has been driven. The same author, as did also Mr. Huizinga, has studied the movements of a pile that had already been driven while other piles are being driven near by.

Finally, Mr. W.S. Housel and Mr. J.R. Burkey have localized, in their report, the zone wherein the structure of the soil was recast in the neighbourhood of a driven pile. They found, more particularly, that a clay deposit was completely disturbed or remoulded at a distance of 3 inches from the pile, but was not measurably affected at a distance of several feet from it.

All these indications, which are already extremely valuable, would in our opinion be profitably completed by systematic experiments.

2nd. Method (Determination of the bearing capacity by direct loading test).

The most interesting report the subject has been submitted by Mr. Plantema (Holland) and by Mr. Franx (Holland).

The authors dwell on the advantages of the new pile sounding apparatus which is used to a large extent in Holland and in Belgium. It is a sort of small telescopic pile whereof the central portion ends in a cone of 10 sq. cm. section with its guiding rod sliding in an outside tube of which the external section is the same as that of the cone base; so that by bringing loads to bear on the rod one can ascertain the degree of toe resistance at each depth.

This telescopic pile system which eliminates skin friction has been used in experiments with single test piles by a large number of other authors.

Mr. Plantema and Mr. Franx ask themselves: Is the cone resistance comparable to the toe resistance of the piles whereof the area is so much greater? The answer is in the affirmative: the authors think that the figures given by deep soundings can, in fact, be extrapolated for toe resistance.

We agree with this conclusion; having arrived at the same conclusion as a result of our experiments with telescopic piles of different diameter in 1936 and 1937 xa).

A second question raised by Mr. Plantema is the following:

What percentage of the cone resistance should be taken in order to keep the settlement within certain limits?

The author notes that the load-settlement diagrams are, to a great exent, the same when the loads are plotted in percentages of the ultimate bearing capacity of the pile toe. He concludes therefrom that for Dutch soils the 2 centimetre settlement rule for tests is of value and that according to the diagram the load required for this settlement is very near the ultimate bearing capacity.

Although we cannot deny the value of this conclusion in so far as it applies to Dutch soil conditions, we cannot say that we agree with the above general conclusion which amounts to saying that the settlements, and also the load settlement relations, must be independent of the toe area. The Boussinesq formulae show clearly that this conclusion cannot be accepted in a general way.

A large number of authors, Messrs. A. Mortensen, Bjerrum, Chr. Ostenfeld and W. Jonsen (Denmark), and Mr. Thornley (U.S.A.) consider that as a complement to the direct loading test, it is possible by measuring the strength of resistance to extraction of a pile to find out the effort exerted on the skin of the pile which comes into account in the total bearing capacity.

To us, this method does not appear to be above criticism: the mathematical theory of the coefficients of passive pressure shows clearly that the passive pressure stress which is exerted from the bottom to the top on a vertical plane (and opposes the downward movement of the pile) is quite different from that exerted from the top to the bottom (opposing the extraction of the pile). One and the other are in the ratio of 3.4 to 1 for $\varphi = 15^{o}$ and of 12 to 1 for $\varphi = 30^{o}$.

3rd. Method. Determination of bearing capacity from the characteristics of the medium concerned.

Messrs. Gregory P. Tschebotarioff and L.A. Palmer have endeavoured to check the agreement of bearing capacities measured by the second method with the results of a formula whereby the bearing capacity is equal to the surface of the upright section multiplied by the cohesion.

Mr. Frederick J. Converse (U.S.A.) tries, for his part, to verify whether the bearing capacity of a group of piles can be calculated by multiplying the perimeter of that group by the average shear strength of the soil.

To us, these formulae do not appear to have any justification from the physical or mathematical point of view. We believe that the known facts available today permit the correct calculation of the toe term and of the skin friction term, in terms of the physical characteristics of the soil. We have shown (ibid.) that the two terms are not independent of each other, the first compensating the insufficiency of the second and perturbing the latter in the neighbourhood of the toe. So that the distribution of stresses along the lateral surface (skin) is not parabolic for a non telescopic pile but entails a reduction at the bottom of the latter.

This phenomenon is noted by Messrs. L. Bjerrum, Ostenfeld and Jonsen (Denmark). - It is also this that causes Mr. Chellis to say that the friction does not increase with the depth.

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x) Equilibrium of Masses with Internal Friction. - Caquot Gauthier Villars, Paris.

xa) Annales des Ponts et Chaussée - 28 rue des Saints Pères, 1939: La Force Portante des Pieux (The Bearing Capacity of Piles)

As there is compensation between the two phenomena, the essential thing is to know if the sum of the two terms given by the correct mathematical theory really coincides with the maximum load that the pile can bear. This is the study which has been systematically carried out by M.M.J. Florentin, G.l'Hériteau and Farhi (France) in their very interesting tests on small scale piles and using electrical extentiometres with SR 4 Baldwin rheostat. They find a fair measure of agreement with theory when assuming for the toe term an angle φ superior to that of the lateral medium, which is normal. The authors also do not find an integral parabolic distribution for lateral (skin) resistance but a parabolic truncated distribution of a certain area at the lower part, the corresponding effort being made by the toe.

Such research with the SR 4 looks as though it should be very fruitful. It is outlined by Messrs. Gregory P. Tschebotarioff and L.A. Palmer who note a resistance to extraction, in non coherent media, equal to 1/4 of the corresponding resistance in the loading test. The corresponding correct test conditions with the SR 4, especially so as to counteract the affects of moisture, are given in great detail by Mr. L. Leroy Grandall. And finally, let us recall the experiments of a similar nature published in France by M. Cambefort in "Travaux" (July and August 1947)

Before finishing with sub-section a, we draw attention to a communication, of a mathematical nature, by Mr. G.W. Glick (U.S.A.) which is a very searching study of the buckling strength of long toe bearing piles in soft ground.

The effect of vibration on piles in sand of low specific gravity entailing additional settlements because of reduced friction between pile and soil resulting from partial or complete liquifaction of the soil, by William F. Swiger.

- The influence of construction methods (especially of grouting at the bottom of piles) on the bearing capacity of piles, by Dr. H.C. Giovanni Rodio (Italy).
- Prescriptions in order to obtain the best casting of cased concrete piles with a guarantee of the elimination of all failures in their sectional constitution, by Sr. F. Derqui (Spain).
- Study of the best use of the material used for the piles in relation to driving possibilities, by C. Franx (Holland).
- The comparison of the extent of disturbance produced by driving piles into plastic clay to the disturbance caused by an unbalanced excavation, by Messrs. Gregory P. Tschebotarioff and James R. Schuyler.
- The causes of the lifting of piles in the neighbourhood of a pile freshly driven in, by Mr. Huizinga (Holland).
- A special case of negative friction, by MM. L'Hériteau and Florintin (France).
- A special case of chimney failure, by Mr. K. Khalifa (Egypt), showing one of the disastrous results of inadequate knowledge of soil mechanics and foundation engineering.

SUB-SECTION b.

Sub-section b deals with horizontal earth pressure against a row of piles.

Although this phenomenon is of general occurrence, its effects are more especially evident in Holland where the roads are usually made on sand fills. This fill increases the water pressure in the underlying clay and peat layers and may result in substantial horizontal pressure against bridge abutments and, generally speaking, against any substructure built on piles.

Mr. Franx makes a systematic study of the means to avoid the harmful effects of this phenomenon while Mr. E.C.W.A. Geuze makes a quantitative analysis of it, in terms of the hydraulic gradient in the soil mass and the results of slow shearing tests on undisturbed samples.

Meanwhile, Messrs. L.A. Palmer and James B. Thomson submit a method for solving the problem of horizontal interaction of earth and single piles by differential equations.

SUB-SECTION c - SPECIAL PROBLEMS.

The two most interesting reports submitted under this sub-section are the following:

Effects of drainage by well points on pile foundations, by Messrs. A. Plomp and W. van Mierlo (Holland).

The positive friction acting along the pile skin decreases after drainage by well point and may eventually change into negative friction.

In three series of characteristic tests, the authors have determined (in terms of the water table level) the respective settlements of the pile and the level of the point where positive friction changes into negative friction.

Level control in buildings by means of adjustable piling, by Sr. Gonzalez (Mexico).

The problem considered is a result mainly due to the peculiar conditions of the subsoil in Mexico, which is highly compressible (the settlement varies between 10 and 20 centimetres per year). The adjustable system allows the pile to pierce through the slab and girders of the foundation by means of screws capable of supporting a cross tie which rests on the pile.

The author shows that the principal advantages offered by foundation on piles with controlable levels are as follows:

1) One or more piles can be easily adjusted to even up or to change their loads, and it will be possible to know the magnitude of the load and to keep it within specified limits.

2) It will be possible to obtain uniform levels throughout the building as it is possible to bring a slanting building back to the plumb line by means of simple and precise operations.

We should like to see these studies carried further, and we should be interested to know the costs of this new technique.

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SUB-SECTION VII a

SETTLEMENT AND BEARING CAPACITY OF PILES

VII a 24

DISCUSSION ON PAPER VIIa 5

A.J. COSTA NUNES (Brasil)

The author of this paper underlined the fact that the allowable settlements of a foundation must necessarily depend on the type of construction and on the actual distribution of the differential settlements.

The author's observation that the fixation of a maximum settlement which will not be exceeded in the load test, without taking into account the peculiar conditions of each problem, may result in "many unnecessary troubles" is a very important one.

In this connection, it would be useful to emphasize that the prescriptions that establish the allowable working load of the piles as a fraction, generally the half, of that corresponding to a fixed settlement, for instance 1/2", seem even more inadequate.

It is really difficult to imagine the relation existing between the settlement of a single pile under twice the working load and both total and differential settlements of the foundation, which are, ultimately, the important thing.

The solution given by the author of adopting as admissible load a fraction of the limit of elasticity, obtained from the load-settlement curve, should not be generally adopted. The reason is, that in most cases that limit of elasticity does not come to evidence in the actual curves, which present a noticeable curvature from the beginning of the test (see figures 1 and 2 reproduced from reference 1).

As an example or evolution in this field we may mention the 1939 and 1940 editions of the D.I.N. 1054 Standards regarding the allowable load of the soil and of the pile foundations 2.3) The first edition adopted, as allowable load of a pile, a half of the limit of elasticity, but the 1940 edition of the same Standards puts down the working load as a fraction of the rupture load.

From a statistic investigation made, 4) based on 100 loading tests, it is concluded that the part of the load settlement curve limited by the working load shows generally an important bend. The deviation of the actual curves from the linear interpolation, around 20% of the cases, exceeded 6% of the total settlement, as fig. 3 (Ref. 4) shows.

At present it would seem that the best way for determining the allowable load of a pile is to adopt a fraction of the rupture loading and to investigate, by computation, the settlements of the soil to be expected under the total load of the construction.

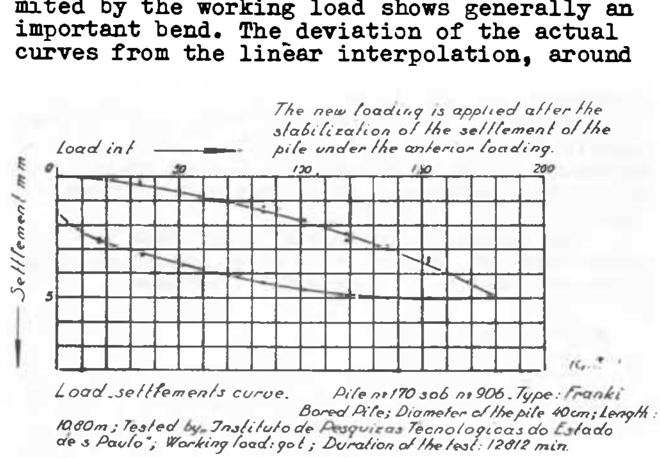

FIG. 1

REFERENCES.

1) Costa Nunes, A.J., Determinação da capacidade de carga de uma fundação em estacas - (Determination of the bearing value of a pile foundation (Revista Municipal de Engen-

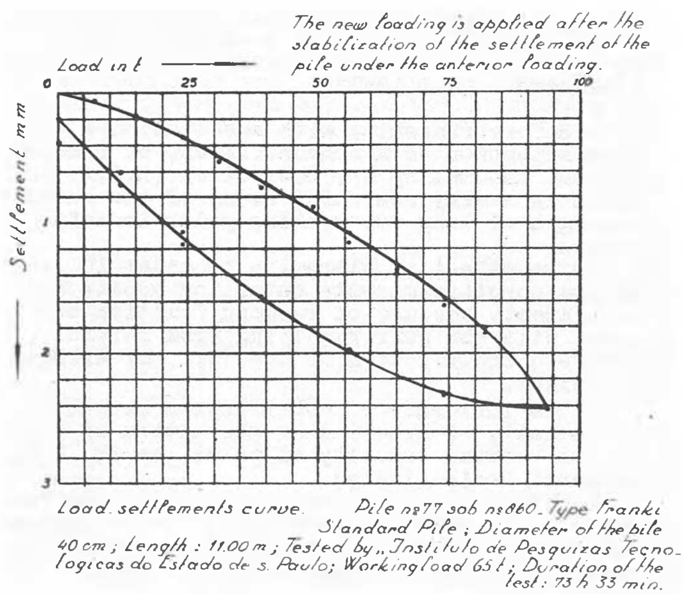

FIG. 2

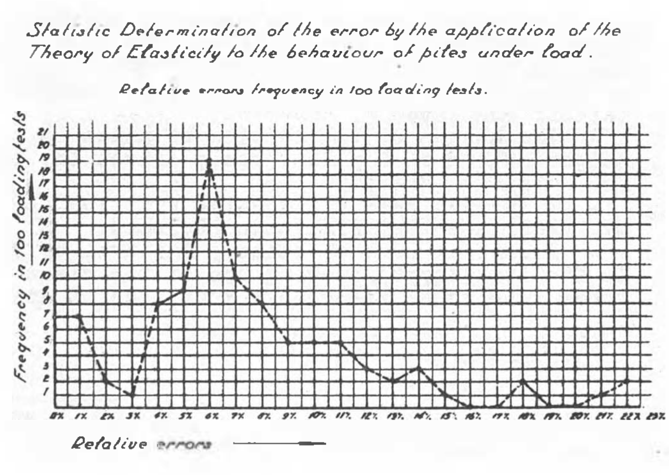

FIG. 3

naria - Jan. 1943 (Brazil)
2) Richtlinien für die zulässige Belastung des Baugrundes und der Pfahlgründungen (D.I.N. 1054, 1939 - Zement Heft 26-1939)
3) Richtlinien für die zulässige Belastung des Baugrundes und der Pfahlgründungen - D.I.N. 1054, 1940 - Zentralblatt der Bauverwaltung - 25, Sept. 1940 -
4) Costa Nunes, A.J.- Distribuição da carga sôbre um bloco de estacas (Load Distribution on the piles of a pile footing) - Symposium de Estruturas (Structures Symposium) - Vol. II - July 1944.

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VII a 25

DISCUSSION ON PAPER VIIa 3

G. PLANTEMA (Netherlands)

Regarding the report "Some pile driving problems" (VIIa 3) Mr. Huizinga writes about waterpressures in the sandlayers during the process of driving piles into them, I would make the following observations.

1) On page 188, 2nd column at the end of the 1st paragraph, Mr. Huizinga says: "There is a limit to the rise of waterpressures, 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 forces acting against the soil being lifted do not only consist of the weight of the mass of soil + the friction of the pile along this mass of soil. It is augmented by the friction of the rising soil along the adjacent mass of soil that keeps resting on the sandlayers.

To give an impression of the waterpressures that would have to become manifest in order to cause a lifting of the soil, the following figure 1 may serve, which was obtained after a simple calculation. It follows from this that if the equilibrium of the pile is only considered without the surrounding soil being lifted, an overstress of waterpressure of abt. 265 t/m^2 is required to raise the pile. If a mass of soil around the pile with a radius of 1.00 m is considered, an overstress of 50 t/m^2 is required; with a radius of 3 m an overstress of 20 t/m^2 would be required. These are impressive values, indeed. Now after this calculation let us go back to reality. It then appears that the overstress of water pressure is considerably lower than that stated in the values mentioned.

On the occasion of a great number of measurements made by the Civil Engineers Department of Rotterdam the maximum overstress was found to be 7 t/m^2, which pressure I thought very high already.

In fact, the effective pressures in the sand around the pile point were reduced by as much as 70%. However, these waterpressures were measured at a distance of 0.50 m from the pile. The waterpressures strongly decreased with the distances from the pile. At a distance of 3.50 m an overstress of as much as 0.50 t/m^2 was indeed ascertained.

If, for instance, this figure is compared with a pressure of 20 t/m^2 which is required for lifting the soil, the phenomenon of the soil being lifted by the overstress of the waterpressures in the sandlayer as mentioned by Mr. Huizinga is very unlikely to occur. Moreover, if such a phenomenon did occur it would not be of any importance, as, water being hardly compressible at all, a minimal lifting of clay and peat layers above the sandlayer would already be sufficient to reduce the waterpressure to normal values.

2) On page 190, Mr. Huizinga develops the main theme in his report, the explanation of the difficulties met with in pile driving.

I regret I cannot at all fall in with his views in this matter either.

The fact of the matter is Mr. Huizinga says that it can be expected that in fine sandlayers the flow of the water overstress of the water pressure, caused by the first piles being driven in, remains in the sand, so that when the following piles are driven in there is already an overstress. This will cause a gradual increase of the overstress of the water in the sandlayers and the set per blow will gradually increase on continuing the pile driving work. Now, this line of thought is quite incompatible with the reality as measured by me by means of an electrical waterpressure cell described in another report (VIIa 15). These measure-

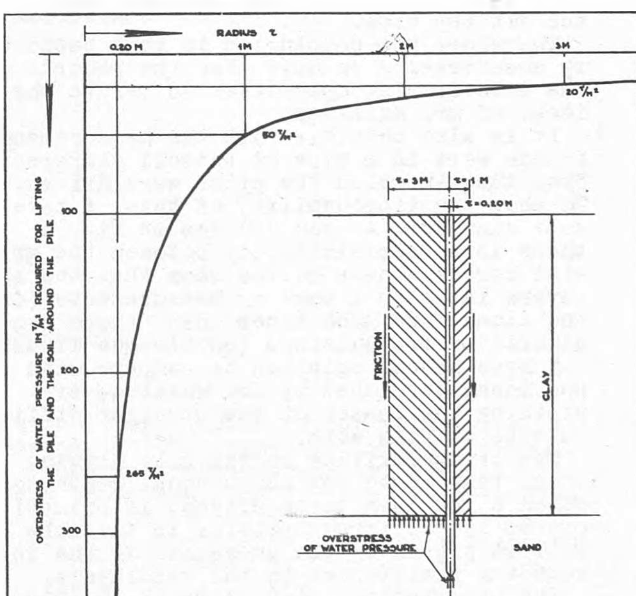

FIG. 1

ments have revealed that

a) the overstress of the waterpressure after from 5 to 15 minutes has entirely disappeared. (Exceptionally, a time of 45 min. has been ascertained, but this was in a spot 0.50 m distance from the pile).
b) at a distance of 4 m from the pile no. waterpressure at all can be measured. Consequently, the overstress caused by pile no. 1 being driven, will have entirely disappeared when driving pile no. 2 is started.

The pause between driving two piles in fact takes abt. an hour. Besides the night-interval with a normal 8 hours' working-day is not less than 16 hours. An overstress of the waterpressure continuing for this period, which is indeed much longer than the 5 to 15 min. interval observed, may be thought to be out of the question.

In addition, the overstress measured during the pile driving at the distance at which the following pile would have to be driven in, is of no importance, let alone its being of any significance after time intervals of from 1 - 16 hours.

Moreover, it is not only my measuring results that refute Mr. Huizinga's trend of thoughts the results of the pile driving work dealt with by him in itself are also contrary to the view he takes.

a) A contemplation of fig. 2 reveals indeed that the gradual drop in penetration resistance as mentioned by Mr. Huizinga is in fact not a gradual drop but a sudden drop. 91 Piles were driven to the normal depth. With an average the piles no. 92 - 103 were driven 3.00 meters deeper.
b) After driving pile no. 104 there was a pause in the operations for 15 days. During this period one can surely expect that the overstress in waterpressures will have entirely disappeared. But on the contrary on recommencing the pile driving the piles (no. 105 - 109) were driven to the same depth as previously, with the same poor set per blow. The piles no. 110-239 (the last) were driven again to a higher level of 18.00 m - N.A.P. with a gradual decrease of the set per blow.

Therefore the conclusion is that besides my measurements we have also the records of the pile driving operatives to refute the ideas of Mr. Huizinga.

It is also possible that the measurements I made were in a type of subsoil different from that in which the piles were driven. To show the improbability of this, I have some examples. As you can see in fig. 3 there is a fair similarity between the grain size curves. These curves show that the sandlayers in which I took my measurements (dotted lines) are much finer than those considered by Mr. Huizinga (continuous lines).

I have my own opinions to compare with the ideas developed by Mr. Huizinga explaining the causes of the peculiar difficulties of this work.

The irregularities in the pile driving work, that is to say the unequal depth to which piles have to be driven, is primarily caused by the irregularities in the subsoil as proved by the uneveness of the deepsounding resistances in the sandlayers.

The bad results of sounding no. 6 in Mr. Huizinga's report would have been similar, even if made before the pile driving work began. That there are great differences between the results of deepsoundings from one place to another is not so uncommon in the municipal area of Rotterdam; excellent examples I can show in figure 4.

In the one case two soundings were made at a distance of 10 m, in the other at a distance of 15 m apart.

You will see clearly that it can be only the marked differences in the quality of the subsoil that affect the pile driving work.

It is quite another thing that in driving the pile an overstress in the pore water is caused. However, a good intertretations of this phenomenon is necessary.

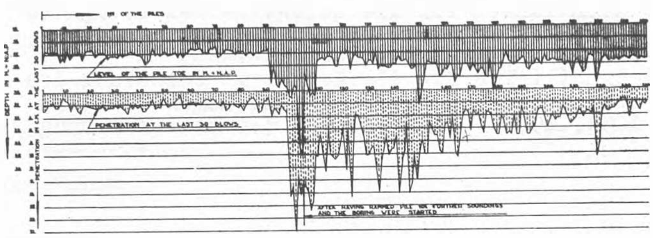

FIG. 2

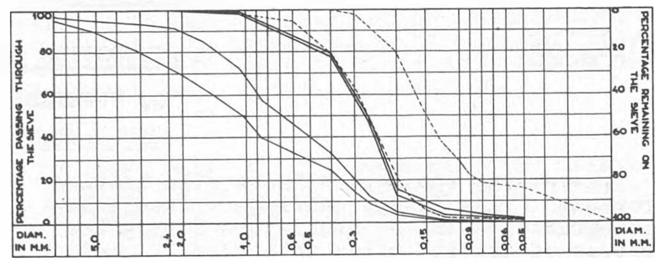

FIG. 3

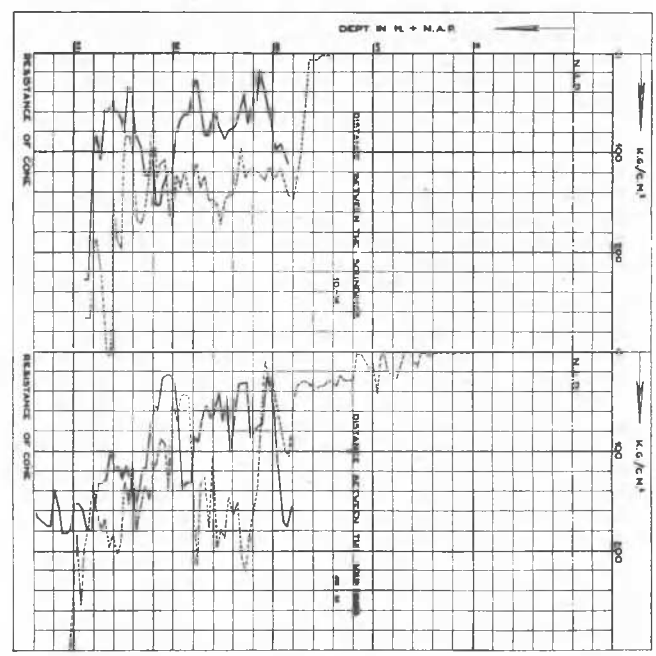

FIG. 4

3) On page 90 in the 2nd column Mr. Huizinga suggested a method of maintaining a good set per blow.

The significance of this is not clear to me.

Because the overstress after driving soon disappears, the bearing capacity for a static load is not influenced, so that it is not at all necessary to pump out the water. The suggestion of Mr. Huizinga is of use only where one is relying on the use of pile driving formula. But these formula, according to Mr. Huizinga, are not reliable. In addition to the above objection the boring of tubes to pump out the water in the sand is not desirable because the disturbance of the sandlayers diminishes the bearing capacity, so that this cure is worse than the complaint.

After some of my principal objections to the report of Mr. Huizinga I wish to turn to the report of Mr. Swiger (VIIa 4). Mr. Swiger has measured an increase of settlement during the vibration, but this increase stops the moment the vibration stops.

The increase of settlement he ascribes to the overstress of the pore water in the sand-layer which causes a temporary diminishing of the bearing capacity. It is very interesting that the measurements of the settlements fully agree with my measurements of overstress. Here I have seen that directly after the vibration (pile driving) the overstress disappears, so it is quite acceptable that also the increase of settlement ceases directly after the vibration. Moreover, I also measured an increase of settlement when testing a loaded pile during the driving of piles around it.

Finally I wanted to suggest a method which can predetermine the extent of the danger when driving piles next to an existing building on piles.

The method is to drive a trial pile near the building and to measure the overstress of water around the pile and this is the measure of the decrease of the effective normal stress under the existing piles.

Then when the bearing capacity of the subsoil is known, perhaps from deepsounding records, you can estimate the consequences of the decrease in effective normal stress.

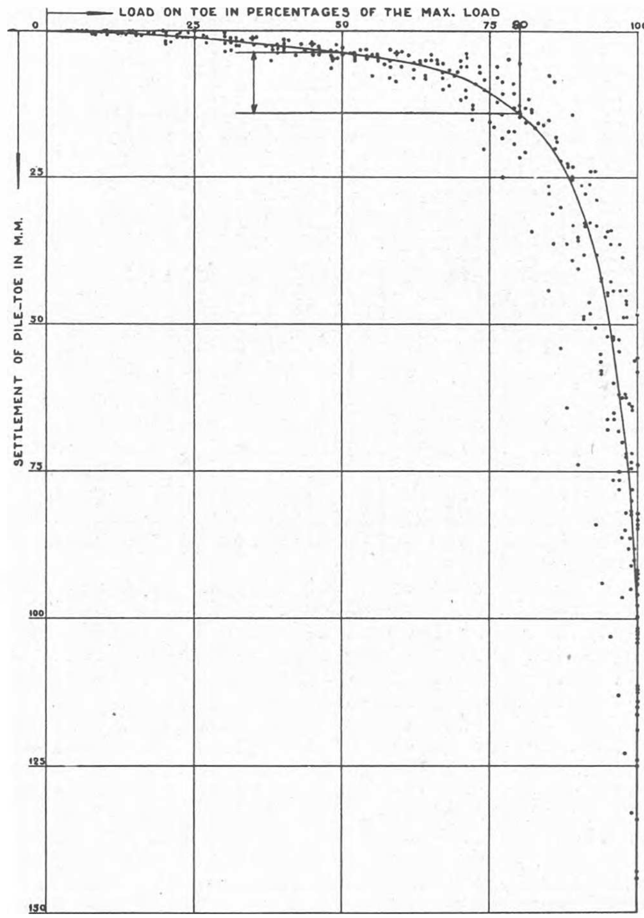

FIG. 5

For this purpose you can use the diagram no. 8 in my report VIIa 12.

If before driving the pile point pressure is 50% of the sounding resistance, that means the maximum bearing capacity of the subsoil, and after pile driving perhaps 80%. you can derive from the curve the increase of settlement (figure 5).

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VII a 26

DISCUSSION

G.C. BOONSTRA (Netherlands)

With great attention I studied all the papers of the section VII, and I should like to make some remarks about the report of this section, composed by Mr. Kerisel, and about some contradictions in a number of papers.

In the papers of the Netherland authors the great advantages are shown of the new pile deep-sounding apparatus for determining the point resistance as part of the bearing capacity of piles. Mr. Kerisel speaks in his report of a small telescopic pile. I remark that this small steel pile is not telescopic, in the sense of somewhat tapered, but cylindrical.

When Mr. Plantema and Mr. Franx in their papers state that the figures given by deep soundings can be extrapolated for determining the toe resistance of a pile, this statement seems to be somewhat in contradiction with the theories of stress distribution and settlement in the soil, and more especially when Mr. Franx gives in another paper a formula for the ratio between the toe resistance and the cross section of a pile.

I want to emphasize that this formula has no theoretical but only practical value in Holland, for, as the writer states, this formula gives a practical limit for bearing capacity of piles with regard to driving possibilities and to the usual driving plants in Holland. This formula is only empiric! This remark is also not a critical one but only a warning for foreign collegues.

Mr. Kerisel in his report says that the 2 cm settlement rule (one inch), usual in Holland as the result of hundreds of pile loading tests and again affirmed by the beautiful tests of Mr. Plantema, cannot be accepted in a general way, according to the relation between the toe area and the settlement, (just like the relation between surface and the settlement of a raft foundation) and the stress distribution computed from the Boussinesq formulae. The practical engineer has to look for a compromise between practice and theory; he knows that the Boussinesq formula is a mathematical formula, valid for a homogeneous, elastic and isotropic mass. Hardly any soil satisfies to the Boussinesq conditions.

On the contrary, not only in Holland, but as a result of a great number of pile loading tests in other countries, which I studied, we may state that the one inch settlement rule in all those loading tests gives nearly always the ultimate bearing capacity of a pile.

I may refer to the many loading tests described in the papers of the first International Congress of Soil Mechanics and to the few papers with regard to pile loading tests in the Section VII of this Congress, viz. the papers of Mr. Giovanni Rodio and of Mr. Housel and Mr. Burkey.

In my own practice I tested a lot of piles, practically all with the same result, that a total settlement of 2 to 3 cm , meant the bearing capacity; also my other collegues, who dealt much with pile foundation, like Mr. Franx. As Mr. Plantema states in one of his papers I believe that a settlement of this size means loss of equilibrium in the sand layer under the toe, in which the driven pile has formed a zone of high density with increased C value.

Therefore I hope that the next Congress may give us more information with regard to this subject.

Similar questions rise with regard to the difference between the value of the skin friction as part of the bearing capacity of a pile.

Many loading tests are followed by a pulling test and the resistance found from this has been considered as the skin friction equal to the pushing test.

In Holland we also made a lot of such tests and as the skin friction in most cases and for the greatest part has been developed in cohesive layers we concluded to equality of up- and downward friction.

Mr. Kerisel means that this method does not appear to be above criticism, due to the mathematical theory that the passive pressure exerted from the bottom to the top on a vertical plane is quite different from that exerted from the top to the bottom.

For $\varphi = 15^{0}$ he means that the ratio is 3,4 : 1 and for $\varphi = 30^{0}$ not less than 12 : 1.

I cannot believe that this theoretical view on the subjects is right and that the skin friction is only a question of passive earth pressure. To a certain degree it may be true for non-cohesive soils but for cohesive soils the difference between the values for skin friction, measured from pulling and pushing tests, may be much less than Mr. Kerisel means.

The results of pushing and pulling tests on piles without significant toe resistance do not show such great differences and although it is possible that in non cohesive soils as sand or gravel there may be some difference, in this question there also may be stated a great difference between theory and practice.

Some authors tell us in their papers for this Congress that the downward and upward skin friction are equal, like Mr. Mortensen, Mr. Bjerrum, Ostenfeld and Jønson, on the contrary Mr. Chellis states that there is a great difference. Mr. Tomby in his paper "Pile test programs" states that the skin friction can be determined by a pulling test and that the direction has no influence on the friction; also Mr. Converse in his paper over the "Determination of pile bearing capacity from sub-soil investigation and laboratory tests." Mr. Tschebotarioff states that in non cohesive soils pull-out tests give lower values (ratio 1 : 4) for the skin friction than pushing test, but this conclusion is also based on laboratory tests.

It is probable that concerning this question laboratory tests do not agree with real scale tests. I may refer to the remark about this subject in Prof. v. Terzaghi's address to the conference on yesterday.

No more it is sure that the results of the skin friction measured with the deep sounding apparatus with a very small surface can be extrapolated to the surface of a pile.

I ask in this meeting "Who is right"?

For the study of the pile problem it is of the greatest importance to get more information with regard to the value and distribution of skin friction for different types of soils and piles.

Another question is the high values for skin friction, given in some papers, sometimes up to more than 10 tons per m^2, much more than we ever found in Holland in firm layers.

Are the tests reliable, for instance the tests with the electrical resistance strain gauges, described by Mr. Grandell? He found skin frictions up to an average of about 15 tons per m^2 over the pile length, even much more for the upper part of the pile and he concluded that there was no point resistance at all. He gives no further explanation for this conclusion.

In a compiling study of pile friction values Robert Chellis also gives very high values of 16 tons per m^2 and more for friction in coarse grained soils as sand and gravel.

It should be of great interest to know in which way these extraordinary high figures have been found, especially when the author states that point bearing has been neglected when reporting most loading tests.

This statement is of no value for the circumstances in Holland, where pile foundations have been applied in almost all the structures and where hundreds of pile loading tests have been made, showing much lower values for the skin friction, even in very firm layers. The above mentioned remarks show that some problems on pile foundations have not been solved. Between the second and third Congress a lot of investigation work has to be done, both for the practical and the laboratory engineer.

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VII a 27

WRITTEN ANSWER TO MR. PLANTEMA (PAPER VIIa 25)

T.K. HUIZINGA (Netherlands)

Engaged in some organizations matters I could only partly participate the meeting of section VII. So when I entered the meeting room I heard only the last part of the above mentioned discussion of which Mr. Plantema did not give me any notice beforehand. As no time was available for free discussions I could not reply and so I take this opportunity to do so.

As to point one: I agree with Mr. Plantema that also the friction of the rising soil along the adjacent mass of soil must be reckoned upon. I did not mention this as the phenomenon is still more complicated since the stresses by deformations of the whole soil mass come also into account. As tc his calculation for the necessary pressure for lifting the pile I will bring in remembrance that some publications tell us about the rising of a pile after the hit even so, that it is difficult to keep the pile down. Further I agree with him especially where he says that: "the effective pressures in the sand around the pile point were reduced by as much as 70%". This is the principal phenomenon on which I have called the attention in my paper. These changes in effective pressures are due to volume changes in the sand where critical density phenomena play a part too and the results depend on the original void ratio of the soil, its permeability, the number and volume of the piles and the speed of pile driving. Therefore pile-driving formulae are of minor or no value.

As to point two: principally our opinions do not differ as can be concluded from my report page 190 column 1, but so I cannot agree with Mr. Plantema's conclusion that the records of the pile driving operatives refute my ideas. For a single pile the driving records depend very much on the water pressures in the soil and the rise of these pressures as well as the question whether these pressures may accumulate or not depends on the above mentioned factors and not only on the marked differences in the quality of the subsoil.

As to point three: I proposed this test for showing those who believe only in pile driving formulae that the driving results can be changed by decreasing the water pressures. For practical purposes reliance on results of deepsoundings will be preferable.

I will finish by thanking Mr. Plantema for having called your attention once again to my paper. Collaboration will give a solution for the mentioned problems.

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VII a 28

WRITTEN DISCUSSION ON PAPER VIIa 3

E. DE BEER (Belgium)

With much interest I read this paper of Mr. Huizinga. Also in Belgium there are sometimes descrepancies between the results of deep penetration tests and the dynamic penetration under a hammerblow. These discrepancies do not occur in very compacted sands but only in sand with a more or less loose compacity. I agree completely with the explanation of the author concerning the different behaviour in very strong and in looser compacted sands against dynamic and static penetration.

But the author does not give the explanation of the different behaviour of the piles in the case at Rotterdam considered in the paper. It is worthwhile to notice that the piles were not driven exactly at the place of the deep penetration tests, and that the tests with lower values than the mean (no. 6 and 3) are arbitrarily located throughout the area. Thus it seems possible that the resistance of the sandlayer encountered is very different from place to place. Perhaps the author can give some further information concerning the geological history of the encountered sand-layer.

In a part of the Flanders exist the so called "Flandrien" which is a thick sand-layer of fluvio-estuarine origin. This formation has a very variable compacity and deep-soundings situated as near as 0,50 m. can give rather different results at the same depth.

For instance fig. 1 gives the results of 4 deep-sounding tests and 4 borings performed in this layer at the distances indicated in the fig. 2. The comparison of the results of these tests indicates that the constitution of the layer encountered is erratic and that it must be considered probable that very different dynamic penetration results shall be found at very short distances.

Thus if the layer encountered at Rotterdam should have an anologous origin, it should not be surprising that at near spots the compacity of the sand is different at a same depth, on very near verticals. A rather low difference of the compacity gives already a great difference in static penetration (see for instance the difference between the deep-sounding 6 and 9) (fig. 5) but still a much greater difference in the dynamic penetration.

The author has paid most attention to the possible discrepancy between the dynamic penetration during driving and the deep-sounding penetration in not very densely compacted sands.

Another discrepancy can arise during pile-loading tests. Indeed in Belgium the insurance companies are customary to specify that the movement of the head of the pile may not exceed 2,5 mm. under the effective load, and 5 mm. under one and a half times this load. These excessive requirements are generally easily satis-

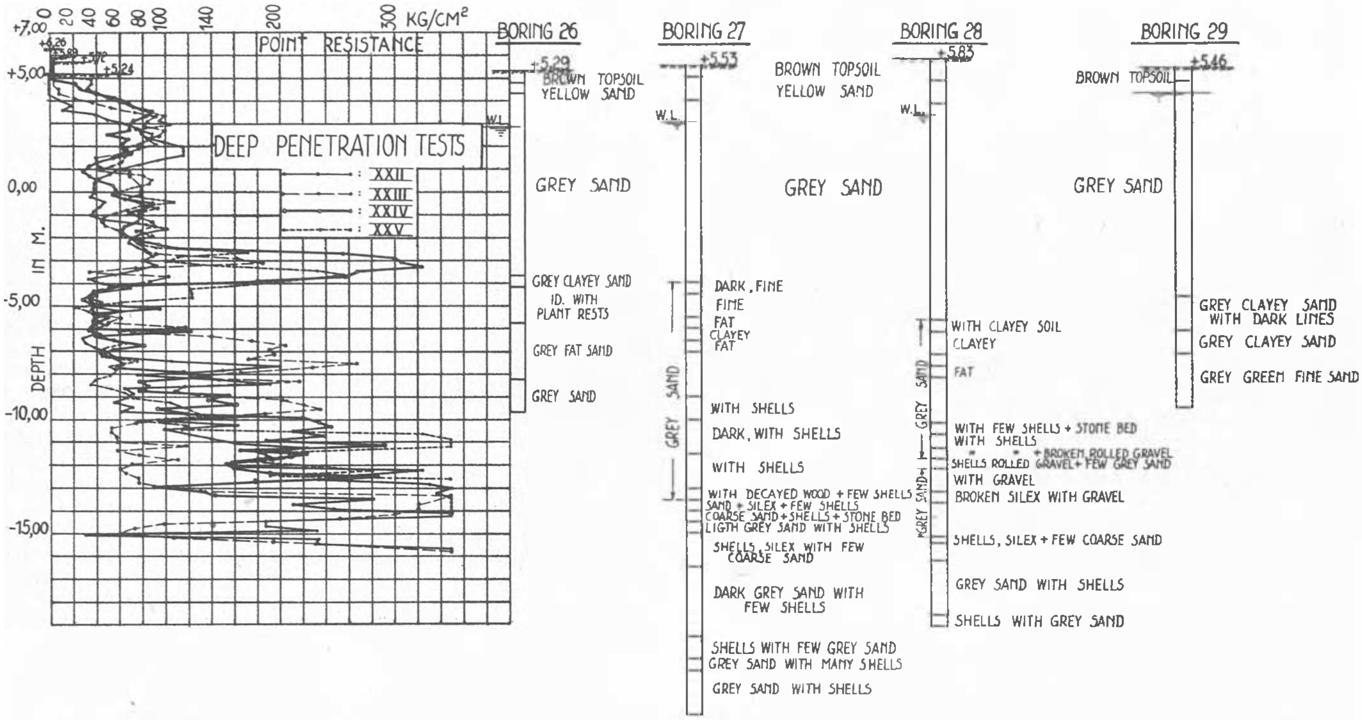
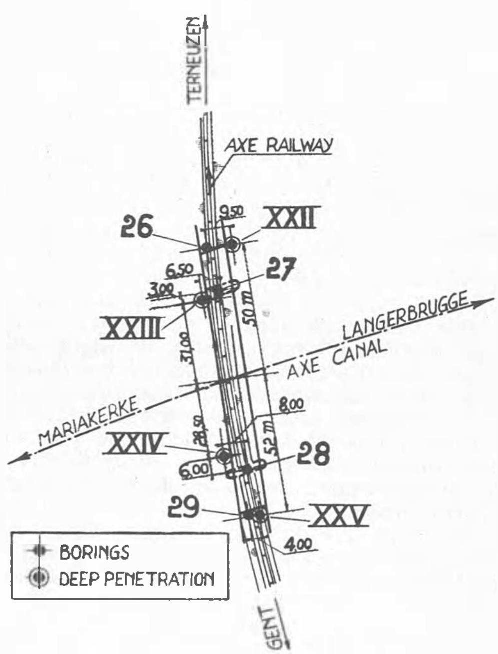


FIG. 1



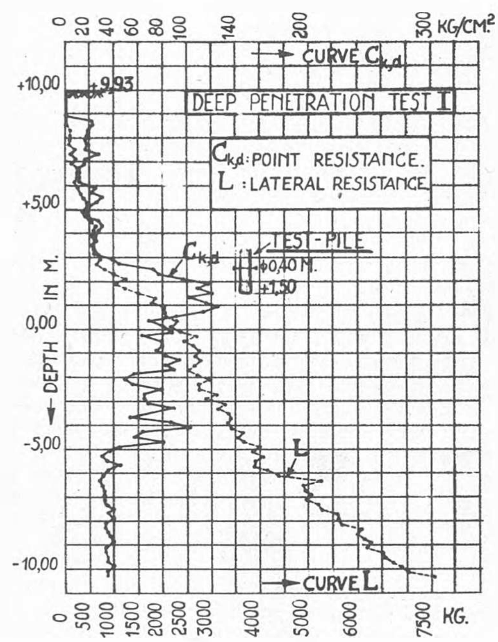


FIG. 2

FIG. 3

fied when the pilepoints are located in very dense sand layers, but can not be satisfied when they are located in less densily compacted layers In case of such layers the excessive requirements concerning the settlements, bring generally to use much longer piles than necessary.

For instance fig. 3 gives the results of the deep sounding test performed at Kortrijk. They indicate the presence of a thick sandlayer of medium to rather loose compacity underlain by the tertiary Ypresian clay. From the deep sounding test it is deducted that it is advisable to have the pilepoints located in the sand layer.

But as the compacity of this layer is not very high, it may be expected that the records of the dynamic penetration during driving and even the excessive requirements imposed on a loading test will give negative results. A pile ⌀ 0,40 m. was driven with its base to the level + 1,50. The mean penetration during the last 10 blows was 1 cm. per blow. The results of the pile loading test are given in fig. 4. Taking

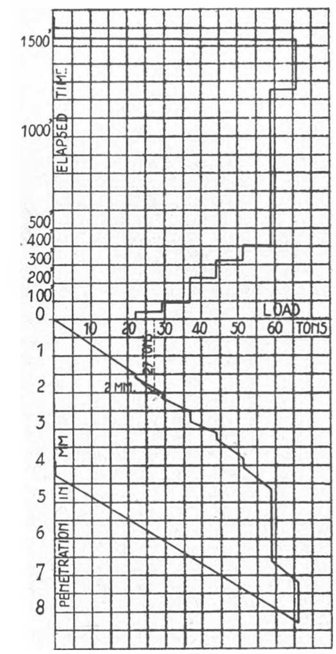


FIG. 4

into account the requirements imposed by the Insurance-companies the load on the pile may not exceed 27 tons, to be compared to the load of 40 tons deducted from the deep-sounding tests. The results of the loading test itself indicate that the requirements for the settlements are excessive, when the pile is to be subjected to static loads only. It is seen that here again a good use of the results of the deep soundings is far better, safer and more economical than to follow strictly blind rules as those concerning the tolerable settlements of the head of the pile during a loading test.

The conclusion is that in not very strongly compacted sandlayers the bearing capacity of piles deduced from the dynamic penetration during driving and even that deduced from an excessive limitation of the settlements during a pile loading test, is much lower than the bearing capacity which can usually safely be tolerated on piles supported by such layers. An exception is when the piles are not subjected to a static loading, but to vibrations or alternating loadings. In such cases the unfavourable results of the resistance against dynamic penetration must be taken into account. At the contrary for static loads the results of deep-penetration tests can be relied upon.

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CONCLUSION

J.L. KERISEL (France)

I merely wish to add a comment to what Mr. Plantema and Mr. Boonstra said in their interesting observations.

Mr. Plantema spoke of over-pressure in the liquid medium owing to pile driving. He refers to Mr. Huizinga's report, and agrees as to the reality of this phenomenon, but debates its permanence and its localization in space.

It is impossible to debate this question, but I think everyone will agree to recognize the necessity of investigating this phenomenon and of following in Mr. Huizinga's and Mr. Plantema's footsteps.

As for Mr. Boonstra, I would reply in all friendliness that by a telescopic pile I meant an apparatus, such as the "sounding cone apparatus" employed in the Netherlands, i.e. one composed of two tubes sliding one within the other. I see that this word, translated into English, gave rise to a different interpretation, but we agree as to the type of apparatus. The only point at issue is the different interpretation of one word in two languages.

I want to confirm to Mr. Boonstra in another way that in my view lateral resistance on pulling is different from that exercised by the soil on the lateral surface when pushing the pile.

In this connection I should like to submit two particulars:

1. Most authors, in their experiments, take as the measurement of the resistance exercised by the soil on the lateral surface when pushing the pile, the difference between the total force and the force on the point, both observed successively by means of an instrument similar to the "sounding cone apparatus". This calculation is not correct. The friction at the lateral surface is greater than this difference, because when in the "sounding cone apparatus" a force is exerted only on the point, the resistance thus measured is greater that the portion only of the resistance of the point, if the same force had been exerted on an ordinary pile of the same dimensions (of Florentin's, l'Hériteau's and Farhi's experiments).

2. For soils with a small angle of friction the lateral resistances on pushing and on pulling do not differ much mathematically (in the case of a liquid they would be equal) and it is this which leads experimentators to assume a would-be equality. I am convinced, however, that carried out in a sand with a large angle of friction, the experiment will yield entirely different results.

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