Session 8

Pile Foundations

Fondations sur Pieux

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J. Trofimenkov, Chairman

INTRODUCTION

On behalf of the Organizing Committee for this X ICSMFE I would like to welcome you to Session 8 on the subject of pile foundations. This is, of course, a very large and complex subject.

The topics for today's discussions suggested by the Organizing Committee and chosen by the General Reporter and myself are the following three:

1- Use of stress-wave theory in prediction of pile performance, including comparisons between the bearing capacity determined by stress wave measurements and results from static load tests.

2 - Friction piles, including change of soil properties after installation of driven and cast-in-place piles.

3 - Behaviour of pile groups, including comparisons between settlement of single piles and pile groups.

In these topics there are the largest uncertainties and several points that could be discussed.

As regards the order of our discussion, we shall first have the General Report.

Three topics will be discussed in turn, about 60 minutes for each topic. In these 60 minutes we shall have two discussions by panelists and about 5 discussions from the floor. After discussing of each topic we shall have some short comments or summary made by officers of the panel.

The wide use of pile foundations all over the world has made this problem quite important. During the past 20 years the quantity of piles used has increased by several times and reached several tens of million each year. Cross section of piles and their length increased considerably, as well as allowable load per pile, which reached more than 100 t for driven and several hundred tons for cast-in-place piles. The expenses involved in pile foundations are considerable. Saving of a few percents of these expenses by increasing allowable bearing capacity of a pile and improving constructional technique will give considerable economical effect.

In spite of long and wide use of piles, many problems of pile foundations have not been solved as yet.

I will touch two of them which are connected with today's discussion.

First. It is a pity that up to now we have no generally accepted definition of the term "bearing capacity of a pile". Even if we have the curve of a pile load test, which more often has no vertical tangent, the determination of the ultimate load of a pile is quite vague.

There exists a lot of recommendations of how to determine this ultimate load, but no one is generally established. There is additional complication, in that the diagram of a load test is time dependable and depends on the procedure of the load test. From these two conclusions follow: in our today's discussion, we should try to define the term "bearing capacity of a pile" when we use it, and in our future work we should speed up the preparation of our Society's recommendations on pile load test procedure. They should not be very rigid, but merely a set of minimum requirements, which will help us in the evaluation of the test results.

Second. It is important to remember, that a group of piles shows completely different behaviour from a single pile. But in pile foundations, piles work almost always as a group of piles. That is why any investigation, theoretical or experimental, on a single pile should not be applied directly to a pile foundation. Most of the recommendations on the relating of the results of a load test on a single pile to the behaviour of a pile group are based on elastic solutions which are often not adequate. Now are justified so-called efficiency coefficients. Settlement observations on existing structures could help tremendously to rational design and construction of actual pile foundations. And we would very much appreciate these data in our today's discussion.

A.F. van Weele, Panelist

WAVE EQUATION FOR DRIVEN PILES

Mr. Chairman, my contribution will be applicable to piles installed on land in non-cemented soils. Although it was quite some time ago, many of you will remember that the Engineering News Record has once published an article about pile-driving formulae, in which they explained a great many of these, maybe as many as one hundred. In Holland Mr. Huizinga did the same around 1950 for his book "Soilmechanics". Unfortunately for you, this book was written in the Dutch language, so you cannot use it and one of its interesting diagrams. The diagram I mean is given in figure 1. It has been of great help to us in many instances. It shows

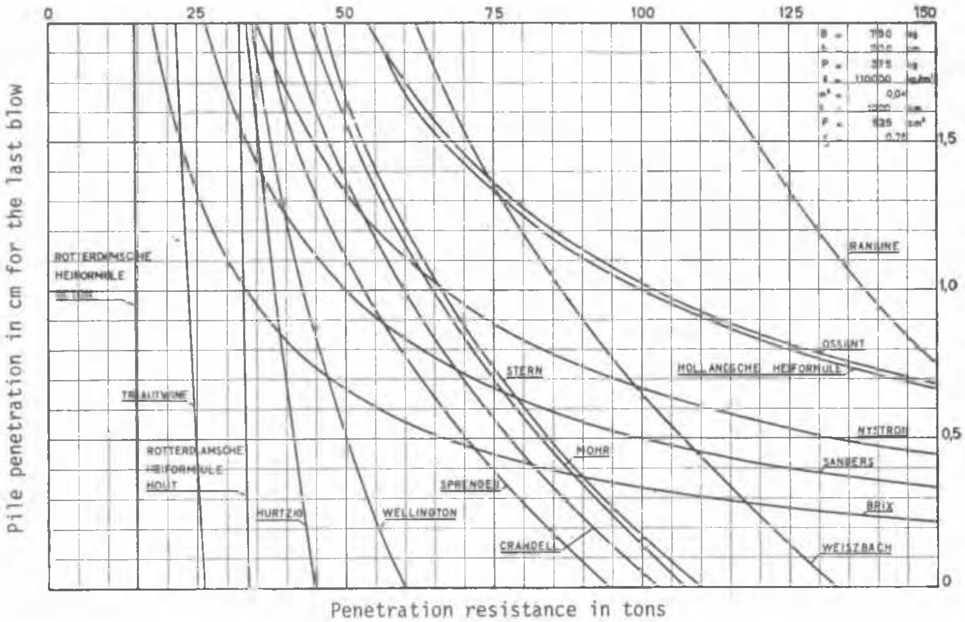

Fig. 1. Relation between penetration resistance and pile penetration per last blow according to various driving formulae for a timber pile driven with a free fall hammer.

the relation between the pile-penetration per blow (usually taken as the average of the last 10 blows) and the ultimate bearing capacity, as given by 14 different driving formulae. So I decided to present it to you.
So let us assume that your pile ended with a penetration of 5 mm per blow and that the desired pile-capacity amounts to 600 kN. The diagram immediately tells you that in that case you should use the formulae of Brix. It is just at the safe side. For another pile, however, you need 1000 kN while your penetration is as much as 10 mm. Now you recommend the Dutch driving formula or that of Ossant. The diagram learns you other interesting things: The Rotterdam formula arrives at 300 kN anyway no matter what your pile penetration is ! Apparently a formula made by or for the supervising authorities. Mr. Rankine seems to be the most optimistic person in the group. Was he a contractor ? Sprenger, Crandell, Stern and Mohr show little differences. Which 3 of them copied the results more or less ?
It will be clear that for a given pile, a given driving assembly and a given soil-profile, only one single relation between pile-penetration and bearing capacity can exist. Out of the 14 possibilities shown, I think all 14 will lead to a wrong answer. Nevertheless such driving formulae are still in use. Also in the present contributions several authors still mention such an approach. In my opinion is what they do maybe appropriate to them but certainly not to us. Why ?

Our profession is one in which knowledge is to be combined with experience. The piledriving formulae are mainly based on experience and are therefore mainly valid, if at all, under given local conditions. Their results can certainly be in good agreement with the results of static loading tests and so justify their existence. They should, however, not be recommended to others on the false basis of knowledge or theoretical backgrounds for other locations where either the soil-conditions are different or the pile-type or the driving means. This would lead to the kind of confusion shown to you by the Engineering News Record or Mr. Huizinga's example.

This example is only one of the many that apply to the foundation problems, we have to solve daily. We must be aware of the fact that our soils, even when we describe them as "stiff green clay" or "uniform medium dense fine sand" show an enormous variation in properties and thus also in behaviour. A great deal of the difference between a good and a bad soils-engineer, lies in his experience with comparable problems in comparable soil-conditions.
A good soils-engineer is always aware of the great many factors, influencing the outcome of any foundation solution to be judged by him. The ideal homogeneous- and isotropic soil with a bi-linear elasto-plastic behaviour, forming the basis of many theoretical approaches, is certainly a great exception. So far I myself did not come across any such soil.
The step from the driving formula to the wave-equation method was a fundamental step forward to decrease the contribution of experience and to increase the contribution of knowledge. It has resulted in a theoretical approach of assessing the dynamic resistance in a much more appreciable way. It does, however, not mean that we can be satisfied and tell our colleagues that we have solved our old problem once and for all.
We should namely not forget that:
1) According to several contributions to our todays session, the driving resistance may increase considerably after driving, by allowing the pile and even more its surrounding soil, some rest. We all have experienced that an increase in pile-capacity with time after installation, takes very often place although not always.
2) We should neither forget that piles are mostly installed at short relative distances. Neighbouring piles installed later than a given pile, will compact the soil further and in sands, generate an increase in lateral effective stresses. This effect may increase the bearing capacity of that given pile considerably after driving as has been indicated by Lindqvist and Petala from Finland.

Only these two facts make it obvious that if we want to use wave-equation methods for a clear judgement of pile bearing capacity or pile integrity, we should only apply this method by redriving piles after the corresponding section of the whole foundation has been completed. We should stop to think that with the driving of a pile, its static bearing capacity can automatically be recorded. What maybe can be recorded, is the pile's dynamic resistance, which mostly is a different item. Of course, there will be cases where someone will be lucky enough that the differences between the two are small. Don't use, however, such a case to suggest to others that such a resemblance is common practice. It simply is not true !

If we use the wave-equation theory either along the lines given by Smith in 1960 or by the Case Western method as now further explained by Santoyo and Goble, our approach remains to be a very rough approximation of what happens in reality.
We still have to guess the values of a number of soil parameters such as quake and damping for friction as well as for end-bearing and also the distribution of the shaft friction along the pile. In order to make things even more complex, you will recall that Prof. Vesic has shown that the shaft friction in a given soil-layer has not a fixed value but decreases with increasing pile-penetration. Further is in the usual theoretical approach assumed that each hammerblow is given exactly centrically and also that the soil surrounding the pile remains absolutely motionless. Everyone having experienced only one single piledriving job, knows that the reality is different so that our theoretical model certainly needs further refinement.
In fact basic research is needed as to how the shear resistance of different soiltypes reacts, when a contact surface moves fast (0,50 - 5,0 m/sec) and that only during a few milliseconds. If we would understand this phenomenon we may be able to refine on our soilparameters or we may even need to adapt our mathematical model. The information on negative and positive waterpressures as given by Möller from Sweden, contributes to a better understanding of the behaviour of dense sands, although model tests have their limitations.

Let us not concentrate on our lucky jobs, where the actual conditions were apparently so favourable that the theory could be applied in such a way, that its outcome was in good harmony with the pile-behaviour. The lapse in time between our conferences is certainly long enough for each of us to generate at least one such a happy occasion. No, I would prefer to set our policy for the next 4 years. What are our most important blank areas in knowledge ? What are we going to do about it ? How can we co-ordinate in order to prevent duplications and omissions ?

Who will instrument thoroughly a steel casing tube which is to be used for the installation of driven uncased cast-in-situ piles ? Such a tube can be used for hundreds of piles so that the instrumentation can be used during a long period. End resistance and friction distribution as well as friction-changes during penetration can then be obtained directly and compared with the limited information which is usually gathered at the piletop. A disadvantage is that redriving is only possible for the first pile of each day provided the tube reached full penetration the previous day or even earlier. From tests with full scale instrumented piles, it is known that it should be possible to correlate pile displacement through pilevelocity with pile-acceleration by means of differentiation and/or integration.

1) Who has measurements which were so accurate that this correlation was possible without substantial "streamlining" of the read-outs ? Please let him speak.
2) Who has done or has planned to do research as to what exactly happens between pileshaft and surrounding soil during one single hammerblow ?
3) How important is excentricity of the dynamic load, travelling through the pile in the theoretical model ? Does it influence side friction ?
4) Can we really discover the differences between a pile of acceptable quality and another one which is just below standard, if we use the dynamic method for integrity testing ? It may be easy to note the difference between unharmed and fully broken piles, but where exactly lies the borderline for actual piles ? In Holland we can live with the presence of tensile cracks in precast piles under compression. We have never been punished for it. Such tensile cracks, however, can lead to strong reflections raising doubts about pile-quality.

I do hope that these topics will get the attention they deserve during our conference as well as during the coming 4 years so that we may learn the progress made at our next conference.

M. Appendino, Panelist

DYNAMIC LOAD TESTS INTERPRETATION

The number of contributions in this Conference as well as in recent specialized meetings brings us to judge that dynamic testing of piles is becoming a subject of great interest. Let me also refer to papers in the "Numerical methods in offshore piling" Conference-London,, 79 and in the "Application of stress-wave theory on piles -Stockholm 80 and Colorado 81 - Seminars.
The load for dynamic testing may be originated by the driving hammer, either on driving operation or on redriving, or by dropping a weight or by a periodic force. The last two loading procedures are suitable particularly for not-driven piles, such as caisson piles.
The following applications are so possible :

1. stress control during driving operation,
2. driving hammer efficiency control,
3. pile integrity control,
4. drivibility determination,
5. static bearing capacity determination.

I will limit my contribution to interpretative problems connected with the determination of static resistance and stiffness. This is based on a two steps procedure consisting of :

1. Determination of motion and forces along the pile either from the analysis of reflected forces measured at pile top or from measurements along the shaft and at the pile tip.
2. Elaboration of outputs through a model for soil-pipe interaction.

The model usually consists of a set of elasto-plastic springs connected with viscous dampers, which can represent both the dynamic and the static soil interaction. Static soil parameters such as resistance and stiffness (or Quake) do not necessarily coincide numerically with those obtainable from a dynamic test. A correct elaboration requires a due consideration of the way by which the loading is applied.
Table 1, comparing testing procedures, points out the large differences between static and dynamic approaches

TABLE I

TEST	DYNAMIC				STATIC		
LOADING	DRIVING	REDRIV.	FALLING MASS	SYNUSOIDAL	C R P	M L	CYCLIC
RATE	$(1-2)\ 10^{6}$ KN/SEC			10^{2} KN/SEC	(2-10) KN/SEC	$10^{-2} - 10^{-1}$ KN/SEC	2-10 KN/SEC
DURATION	$(5-10)\ 10^{-3}$ SEC			10^{-1} SEC	30/60 MIN	HOURS-DAYS	MIN-HOURS
REPETITION	$10^{2} - 10^{3}$ CYCLES	$1 - 10^{2}$ CYCLES	1 - 10 CYCLES	$\sim 10^{3}$ CYCLES	NONE	FEW	10-20 CYCLES PERSTAGE 5-10 STAGES
INTENSITY	FAILURE	FAILURE ?	LOW TO DESIGN LOAD	VERY LOW	FAILURE OR UP TO 1,5 - 2 THE DESIGN LOAD		

as well as the important differences existing among the tests in the same approach. We must expect from Table 1 that :

Under dynamic loading

- soil reaction must include inertial effects in soil mass, resulting at the pile boundary as a stiffness increment and an energy dissipation (elastic waves radiation), Appendino (1980),
- undrained failure always occurs, even in sands and consequently pore pressure variations from static conditions are possible, see e. g. Appendino (1980), Möller and Bergdahl (1981),
- soil parameters must be expressed in terms of dynamic soil properties, Appendino (1979),
- on driving, soil is displaced laterally, thus creating a remoulded zone around the pile shaft, Vijayvergive (1980), and high positive pore pressure occur,
- repeatitive loadings produce soil initial resistance and stiffness degradation, Möller and Bergdahl (1981), Mizikos and Fournier (1981) as to produce liquefaction,
- when shaft resistance is fully degradated then shaft damping also vanishes, Mizikos and Fournier (1980), as energy can no longer be transmitted to the soil.

Under static loading

- soil resistance and stiffness depend on test duration as a consequence of consolidation and creep, Bergdahl and Hult (1981),
- resistance and stiffness depend also on the failure criteria used to interpret load test,
- soil resistance and stiffness depend from the time elapsed between the end of driving and the static loading, as a consequence of strength recover in cohesive soils, Bergdahl and Hult (1981), and stresses redistribution in sands,
- resistance and stiffness may be modified by performing the dynamic redriving test in advance, Balasubramanian (1981),
- large uncertainties exist in splitting shaft results resistance from tip resistance, even if pile is instrumented, see e. g. Appendino (1980, 1981).

Then we must also ascertain that enough energy and force are supplied to fully mobilize soil resistance. A small permanent set cannot be a positive proof, because it may also be the consequence of non-linear soil deformability. If full strength mobilization is not obtained, then the dynamic test gives only pre-failure soil properties, usable to determine the initial settlement v/s load relationship according to Davis (1981) or Van Koten and Middendorp (1980) procedures.

Correlations may be obtained to convert values from dynamic tests to static ones or viceversa. The simplest ones are given by the ratio between dynamic and static soil parameters resulting from tests performed on the same pile, De Beer et al (1981), Santoy and Goble (1981). This may be a correct procedure for routine driving control when a large number of piles is involved, provided that the soil is sufficiently homogeneous, driving technique is constant and initial soil properties are not strongly modified. General correlations make a step forward. They may be expressed by complex models describing modifications of soil properties occuring during the test, Voitus Van Hamme (1980), Matlock and Foo (1980), or simply throug corrective coefficients, Vijayergiva (1980), Appendino (1979).

Data required by these models or corrective coefficients may be determinated in laboratory with the calibration

TABLE 2

SOIL	COHESIVE			GRANULAR		
TEST	1	2	3	1	2	3
$R_{s,d}/R_{s,st}$	$.5c_R$	$c_R - 1.1c_u$	$c_R - c_u$	$1^{(x)}$	1	1
$R_{t,d}/R_{t,st}$	1.5-2.0	2.0-.5	1.5-2.0		$.5-1.5^{(x)}$	

$R_{,d}$ = resistance for dynamic loading

$R_{,st}$ = resistance for static loading

c_R = remoulded cohesion

1 = driving, 2 = initial redriving or dropping weight, 3 = redriving

(x) = smaller ratios occur if excess pore pressure accumulates

cell for instance, Möller and Bergdahl (1981), Steenfelt et al (1981), Mizikos and Fournier (1980).
The expanded cavity theory is useful to express the driving effect on shaft interaction, Leifer et al. (1980), Carter et al. (1980), Wroth et al. (1980), or on the pile tip, Appendino (1979). Table 2 gives indicative coefficients for soil resistance.
Procedures based on corrective coefficients cannot be very accurate; on the other hand the complexity of the problem involves imprecision. Moreover we must remember that static tests or bearing capacity formulae cannot give precise results as well. We must consequently accept a reasonable estimate of static resistance, which I think it may be expressed by $\pm$ 25% .

R. Dahlberg (Oral discussion)

VERIFICATION OF PILE CAPACITY THROUGH DYNAMIC MEASUREMENTS
Verification de Portance de Pieux par Mesures Dynamiques

About one year ago, Petrobras had the first Brazilian deep water platform, the Garoupa PGP-1 platform, installed in the Campos Basin in 120 m water depth. The soil consists of an upper calcareous sand layer, about 20 m thick, overlying a silty clay. Between about 86 and 90 m there is a dense calcareous sand layer.

The 36 foundation piles, 1219 mm in diameter, were designed for a penetration of 85 to 90 m below mudline with the tip slightly embedded in the calcareous sand layer. Limited knowledge of the engineering behaviour of calcareous sands and some doubts regarding the insitu values of the undrained shear strength of the clay made the predicted pile capacity uncertain. With the aim to verify the capacity of the piles as intalled, some of the piles were therefore instrumented during driving. The instrumentation work was carried out by Det norske VERITAS under contract with Petrobras. Measured force and acceleration in the pile top were then used as input to a dynamic analysis known as CAPWAP (see Goble et.al., 1970 and Santoyo and Goble, 1981). The result of the CAPWAP analysis are the magnitude and location along the pile of both static and dynamic resistance forces.

In order to get a best possible estimate of the static pile capacity, measurements were taken both during initial driving and after two months of set up. The redriving was carried out on three piles using a Menck 5000 hammer which had been "warmed up" immidiately before by driving an another, non-instrumented pile.

The measurements showed a considerable increase in the CAPWAP predicted static capacity in the course of the two months of set up, a factor of 3 to 4. This increase was sufficient to verify that the piles were able to carry the loads with the required margin of safety. In Fig. 1, the CAPWAP predicted total capacities and load transfer distributions are shown for three blows during redriving as compared to what was found during initial driving. It is interesting to notice the gradual decrease in soil resistance in the course of the redriving. After some hundred of blows, the resistance was of the same magnitude as during initial driving.

It is, however, important in the further use of the CAPWAP results to realize the difference in failure mode when a pile is being driven as compared to normal static loading. During driving of open-ended steel piles, there will be mainly frictional resistance since the pile most likely will behave as non-plugged. During static loading a soil plug may form giving end resistance over the full cross-sectional pile area. These considerations are important when converting the CAPWAP results to static pile capacity. Unfortunately, this conversion will always have to involve some amount of engineering judgement in the case of open-ended cylindrical steel piles. However, without the instrumented redriving and CAPWAP analysis, there would have been much more doubt regarding the actual capacity of the Garoupa piles. In the case of high capacity, offshore piles dynamic measurements during redriving with subsequent analysis (e.g. by the CAPWAP method) is the only practical and economical approach to verify the capacity of the piles.

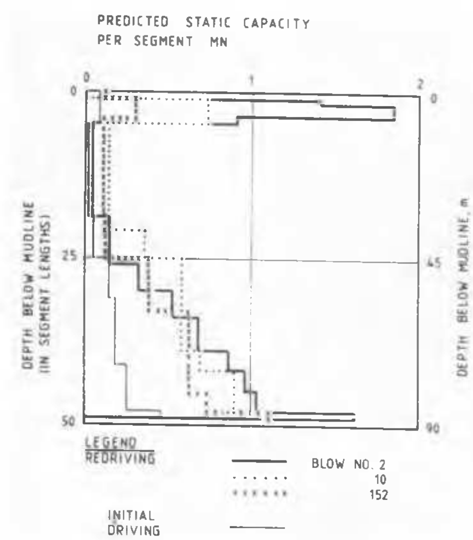

Fig. 1 Results of CAPWAP analysis

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T.Iwanowski and B. Larsson (Oral discussion)

A NEW DISC-SPRING CAP FOR EFFICIENT PILE DRIVING

For more effective pile driving the elastic disc springs have been applied as a cushioning member between the hammer and the pile. The construction has been succesfully tested first on a model scale,and later full scale units production has been initiated. Actually 27 units are utilized, and their behaviour verifies analyticaly computed high performance.
Principles of behaviour of the disc spring cap have been derived from the impact and wave propagation theories where the special material properties of piles have been considered. These indicated that in order to obtain the highest pile penetration, the impact wave of rectangular shape should be applied to a pile. Such an impact wave can not be accomplished by traditionally used wood or plastic packed caps, where nonelastic or elastic nonlinear properties cause a very unfavourable impact wave. High amplitude stress peak at the front of the conventional cap impact wave often causes damage to the pile, especially when reflected from the pile bottom tip as a tensile stress. Short duration of the impact, being a result of unfavourable elastic properties and a large energy losses in the conventional cap could not be sufficient for efficient pile driving. The new disc spring-cap generates an impact wave, which is close to a rectangular shape. An adjustable prestressing force of the spring stack in the disc-spring cap assures enough high forces required for pile penetration into the soil, but doesn´t cause overstressing of the pile material (Fig.1.). Much more of hammer energy is transferred to the pile. Measurment of the stress wave in the pile during driving performed by means of PiD Piling Analysis System indicated that 75% of hammer energy can be transferred by the spring cap to the pile, comparing with 50% efficiency of the conventional cap. This results in an increased pile penetration into the soil. Comparative driving tests performed on drop and diesel hammers equipped with disc-spring cap and conventional cap showed 50-150% increase of pile penetration per blow.

The very stable dynamic properties of the elastic springs give much more accurate data for control and computer simulation of the pile driving process. The disc-spring cap construction decreases noise and vibration level during piling. A 5-8 dB(A) decrease of noise level at distance 7 m from the driven pile has been measured frequently.
Recapitulating, there are the following advantages of the disc-spring cap comparing with the conventional cap:

- 50% more of hammer energy transferred to the pile
- reduced risk of pile damage
- 50-150% increase of pile penetration into the soil
- good durability and retaining of constant characteristic during long time operation
- better blow centering
- decreased noise and vibration level
- adjustability of the disc-spring cap to different piling conditions

Thus,the application of the impact and wave propagation theories on pile driving gave a new, very efficient tool for modern pile driving.

Fig.2. The disc-spring cap for 3 T drop hammer

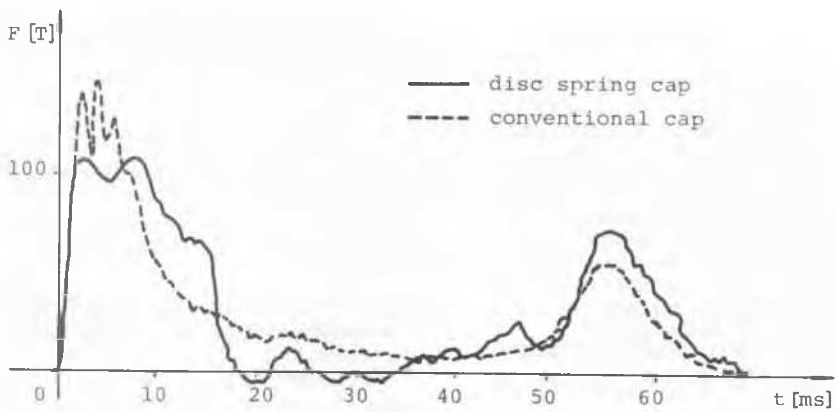

Fig.1. Force measured 1 m from pile head during driving.
Concrete pile 270 mm, length 87 m.
Drop hammer 4 T, drop height 0.5 m.

H.S. Lacy (Oral discussion)

DYNAMIC PORE PRESSURE DURING PILE DRIVING IN FINE SAND
Pression Interstitielle Dynamique pendant Battage de Pieux dans Sable Fin

The interesting paper on Dynamic Pore Pressure (Moller & Bergdahl, 1981) describes the measurements of changes in pore pressure when a 2cm model pile was driven into a 23 by 40cm cylinder containing dense fine sand. Below the pile tip, initial increase in positive pressures from dynamic loading of the sand quickly become negative as the sand distorts in shear. Along the side of the pile negative pressures are built up in amounts that decrease with distance from the pile tip. This is followed quickly by a positive pressure in dense sands, but not in very dense sands. The effect of variation in effective stress and soil gradation was not investigated. The authors concluded that the negative pore pressure phenomenon explained the Swedish experience of high driving resistance which suggested high load capacity which could often not be demonstrated by load tests.

The purpose of this discussion is to present an example of high positive pore pressures that were measured during driving of piles in compact sands and sandy silts, causing very low driving resistances. This site, in Syracuse, New York, is underlain primarily by soft to stiff slightly underconsolidated clay as shown in Figure 1.

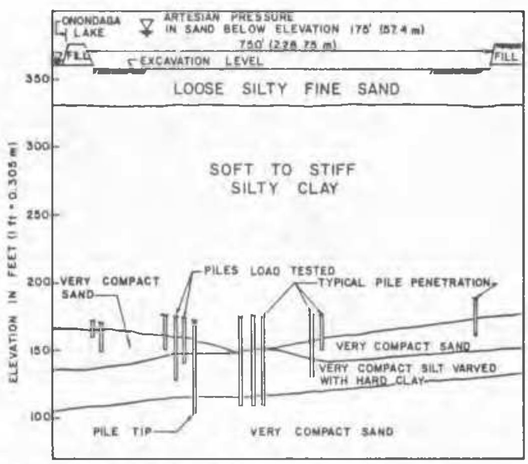

Fig. 1 Typical profile through site

Tanks constructed at this sewage treatment plant around 1960 have settled up to one meter. The pile bearing material is at depths ranging from 62 to 77 meters and consists of layers of very compact sand and silt. Artesian pressures in this confined aquifer resulted in a static head six to seven meters above the three meter deep excavation.

Figure 2 illustrates a marked increase in Standard Penetration resistance as Boring No. B-1 extended from the clay into the compact sandy silt. However, no increase in driving resistance was observed at this level when piles about 1.5 meter away were driven. By this time, a piezometer had been installed with a seal in the clay above the lower sandy silt.

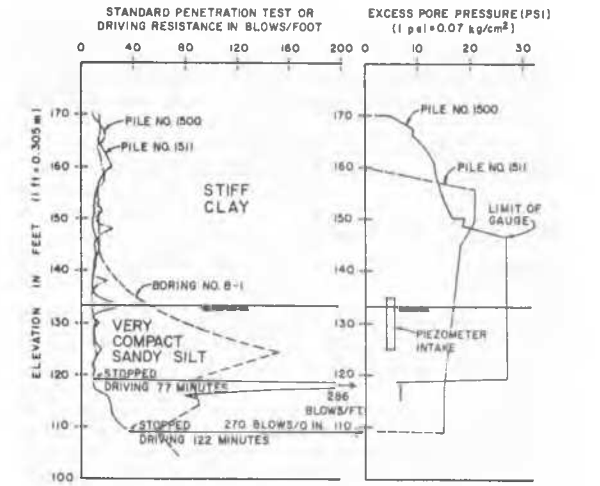

Fig. 2 Excess pore pressure during pile driving.

Pore pressures started increasing while the pile tip was in clay eight to 11 meters above the top of sandy silt. There is apparently some temporary increase in pore pressure in this confined aquifer due to displacement of the clay by these 0.35 meter square concrete piles. While the precise horizontal position of the piles with respect to the piezometer is not known, it appears that pore pressures adjacent to the pile approach the effective overburden stress causing a quick condition in the sandy silt. The excess water pressure dissipated over periods that vary from ten minutes to two hours with some sustained excess pressure for 24 hours. Where pile driving continued in the same area for several days, a small sustained excess pore pressure was maintained. Low driving resistances were observed in the bearing strata erratically throughout the site with no consistent trend with respect to gradation which varied from fine to medium sand to silty fine sand to sandy silt. Redriving the piles shown in Figure 2 after one to two hours following initial driving resulted in near refusal. This result is similar to those by Moe et. al and is a common occurrence in eastern United States. It is suggested that the generation of positive pore pressure during pile driving is more common than generally recognized. As a result of these tests, a program was developed to verify the integrity and capacity of piles driven to founding level at low penetration resistance. This program is described in a paper presented at this conference (Lacy, 1981). The static resistance of the piles was demonstrated by load tests and by monitoring the elevation of the fully loaded tanks over the past two years.

The consistent high driving resistance observed during Standard Penetration tests, using a 3.1cm diameter sampler is a marked departure from the general low, but highly variable pile resistance in these same soils. The soil sampler is similar in size to the model piles used by Moller and Bergdahl. It is suggested that the results of model tests may not always be consistent with the performance of soil around pile tips when high energy hammers are used.

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G.G. Meyerhof, Co-Chairman

INTRODUCTION

As the Chairman mentioned this morning, it is my pleasant duty to open and chair the afternoon session as the Co-Chairman of the Session 8 on Piles.

We have two main topics for discussion: Friction piles, including change of soil properties after installation of driven or bored piles, and the behaviour of pile groups. In the first part on single piles we have subdivided the discussion into two sections, the first section dealing with vertical loading and the second section dealing with horizontal loading.

In each case we have six discussors who have given their names before today. I know there have been additional names added, but I am sorry that there is no time to add these. Anybody who is not able to speak can send this discussion on the special prepared sheets by the 1st of September of this year to the Organizing Committee for publication in the last Volume of this Conference.

As was indicated previously, each discussion topic will be introduced by a member or two members of the session panel. This will be followed by the discussors who appear shortly on a list. Each discussor has six minutes and no longer. Then I will make a brief summary of these discussions.

I will now ask Professor Mazurkiewicz to open the discussion on friction piles.

R.K. Mazurkiewicz, Panelist

BEARING CAPACITY OF FRICTION PILES
Capacité Portant des Pieux Travaillaints aux Frottement Latêral

INTRODUCTION

In the following two contributions on bearing capacity of friction piles are given, namely, the first (I) presented during discussion at the Session 8 on Pile Foundations and the second (II) prepared as introduction to discussion to this Session according to the original program.

CONTRIBUTION I

It is very difficult to discuss the problem of bearing capacity of friction piles. The reason? All of us, I am quite sure, have made at least once an own decision and predicted this capacity with more or less satisfying result. Not only this. Yesterday we discussed in one of our Committees the whole day only one problem, namely, the pile loading tests being a tool not only for checking our decisions but also for better and wider preparation of our predictions. This is also a way.

On the other hand, however, we may clearly state that the researches are doing their investigations writing and publishing a hugh amount of reports and papers and drawing a not known amount of conclusions, which may seem some times to be similar or even the same as these which have been made some 20 or 30 years ago. I would like, however, to state that there are significant differences between the earlier and today statements. These differences concern mainly the wide knowledge of the amount and kind of factors which influence the bearing capacity and which allow to prepare our prediction in a much better way and introduce parameters, which earlier have not been even mentioned.

I think that I can also say that we would be all very happy if we could clearly separate the skin friction and the end bearing. We must, however, in the contrary state that we are from day to day much further from doing this. It is, for instance, reported that the contribution of the end bearing to the total bearing capacity is significantly different for different kinds of piles in cohesive and noncohesive soils. Values at e.g. 15% are obtained for concrete piles in stiff clay, 25% for steel piles in stiff clay and 45% for concrete piles in sand. Additionally, it is many times assumed that the maximum end bearing and the maximum shaft resistance are mobilized at the same time. We have, however, found that these two components are mobilized at different displacements.

The problem becomes more complex and difficult due to the fact that the method of installation and the loading sequence change considerably the properties of the soil and thus influence the bearing capacity.

A large number of different methods to predict the bearing capacity of friction piles exist today. Also a large number of investigations has been carried out where different methods have been compared. Analyzing the situation in this field I would like to ask all of us today: Is it possible to recommend one or more methods which adequately predict the ultimate bearing capacity of friction piles under different conditions, and are we able to present general relationships which take into account the most important factors which influence the ultimate bearing capacity? My answer today is that we have not such a possibility and I must state that it is impossible to evaluate a method which would be valid in the whole world for all kinds of piles and all kinds of soils. We can, of course, use always coefficients which cover all the influences.

Analyzing all possible influences we may conclude that the bearing capacity and the settlements depend on the changes of the strength and determination properties of the soil during the driving, the type of loading, the deformability and depth of the different soil layers, size of the foundation, type and number of piles and the load distribution along the piles. As said before at present there does not exist any method which takes into account all these factors. Thus the following and I think main question may be raised, namely, how can such methods be developed: a) on the basis of investigations in the field and laboratory of pile performance, b) observation of actual constructions, c) theoretical investigations which consider the increase of the pore water of the effective pressures during driving as well as the subsequent reconsolidation of the soil?

Taking into consideration all factors which seriously influence the bearing capacity of friction piles one can rise a large number of questions, the answer of which could maybe allow to reach a more satisfactory result in this field. This is, however, a very long way. Therefore I would like to make one final proposal, namely, that we should prepare ourselves to the next conference in such a way that we today propose for all of us one or two problems which will be the subject of our research work in the next four years. I am convinced that the papers and discussions at the next conference will than lead to a result which may be introduced by all of us in the daily practice. From my side I would propose to make in the next years efforts to widen our knowledge on the following two problems:

1) Influence of cyclic loading on bearing capacity and settlement of friction piles taking into consideration the soil conditions around and under the pile and the vertical and horizontal stiffness of the pile-soil system.

2) Estimation of the reduction of the shear strength in cohesive soils due to remoulding during driving, taking, however, into consideration when estimating the ultimate skin friction the increase of the shear strength of the remoulded clay with time, the changes of pore pressure during the driving and the influence of the permeability of the pile material.

I do hope that introducing this proposition we will be able to reach the necessary level of our knowledge much much earlier.

CONTRIBUTION II

The interaction between friction piles and the surrounding soil is a very complex problem since the skin friction and the end bearing cannot be clearly separated. These two components influence one another during the installation and when a pile is loaded. Considering the nature of the soil (nonlinearity, heterogeneity, sequence and thickness of the different layers, etc.), the pile installation method (driven or cast-in-place piles, etc.), the pile material, the shape of the pile as well as the type and history of the loading (static, dynamic, cyclic etc.) it is evident that it is not possible to predict exactly the bearing capacity of friction piles at the present time.

It is, for example well known that the contribution of the end bearing to the total bearing capacity is significantly different for different kinds of piles (concrete, steel) in cohesive and noncohesive soils. Values at e.g. 15 % have been reported for concrete piles in stiff clay, 25 % for steel piles in stiff clay and 45 % for concrete piles in sand. Many times it is assumed that the maximum end bearing and the maximum shaft resistance are mobilized at the same time. It is, however, known that the two components are mobilized at different displacements particularly if the piles are short. The measured values will thus be lower than the predicted values.

The problem becomes more complex and difficult due to the fact that the method of installation and the loading sequence may considerably change the properties of the soil around a friction pile and thus influence the bearing capacity. We know at present almost all the factors which influence the pile bearing capacity and that a program of future investigations can be prepared which may lead to an improvement of the presently used calculation methods.

A large number of different methods to predict the bearing capacity of friction piles exist today. However, it is very difficult to recommend which of these methods is the most suitable one for different conditions and which should be used. A large number of investigations has been carried out where different methods have been compared. These methods are based on a total stress or an effective stress analysis, on dynamic or static penetration tests etc. The following general topic may thus be raised for discussion, namely:

Is it possible to recommend methods which adequately predict the ultimate bearing capacity of friction piles under different conditions, and can we present general relationships which take into account the most important factors which influence the ultimate bearing capacity? The answer can perhaps be found by comparing the methods used in different countries. Could this comparison be done by a special Committee of our Society?

In this connection a question may be asked about the application of the finite element technique (FEM) to the calculation of the ultimate bearing capacity and the settlements of friction piles. With this technique it is possible to study the load transfer between a pile and the soil, and time dependency of the loads acting on the pile. A soil model (e.g. nonlinear) is necessary which takes into account how the different soil parameters are influenced by the installation and the loading method.

It is commonly known that the ultimate bearing capacity of friction piles depends mainly on the soil conditions around and under the pile after the installation. The different soil parameters are normally evaluated by laboratory or field tests e.g. borings, dynamic and static penetration tests, vane and pressuremeter tests. Investigations indicate, however, that the results from laboratory or field tests of soils around and under the pile before tha installation of the piles cannot be used directly to calculate the ultimate bearing capacity or the settlements. The next question that can be thus asked is:

Can we recommend in-situ methods to determine the strength and deformation properties of soils which are required to predict the bearing capacity and the settlements of single piles and of pile groups e.g. the shear strength of the clay around a pile after reconsolidation or the shear strength of the relatively undisturbed clay at some distance from the pile surface?

The following additional questions can be asked in connection with soil and pile investigations:

1. Is it possible to estimate the reduction of the shear strength in cohesive soils due to remoulding during driving? Can we estimate the increase of the shear strength of the remoulded clay with time and resulting increase of ultimate skin friction?

2. How does the changes of pore water pressure during the driving affect the pile bearing capacity? What is the influence of the permeability of the pile mataterial? How does the local positive and negative pore pressures around a pile during the driving affect the behaviour? Is the pore water pressure during and after driving affected by hydraulic fracturing? Do we know enough about this phenomenon and what is its influence on the bearing capacity?

3) Is it possible to predict the compaction or the laterally displacement of cohesionless soils around a pile during driving or casting? Can the relative density of the soil and the pore water pressures be predicted?

Other factors can also influence the bearing capacity and settlement of the friction piles. Such as:

1. Heave around driven piles.
2. Reduction of the relative density of cohesionless soils after casting (bored piles).
3. Effect of adjacent piles.

The ultimate bearing capacity of friction piles depends on both the shape and the type of pile. For example open end pile is affected by plugging during the driving. These piles are often only partially plugged during driving. They become plugged at a later stage. Both the ultimate bearing capacity and the driveability are affected. The following questions may thus be raised:

1. Is it possible to define the conditions when plugging take place?

2. Is it possible to determine the influence of plugging on driveability, bearing capacity and settlements of a pile and of a pile group?

3. When should open-end or closed-end friction piles be used?

The bearing capacity and the settlements depend on the changes of the strength and determination properties of the soil during the driving, the type of loading, the deformability and depth of the different soil layers, size of the foundation, type and number of piles and the load distribution along the piles. At present there does not exist any method which takes into account all the factors which affect the settlements. The following question can be raised:

How can a method be developed that takes into account these factors: a) on the basis of investigations in the field and laboratory of pile performance, b) observation of actual constructions, c) theoretical investigations which consider that increase of the pore water and of the effective pressures during driving as well as the subsequent reconsolidation of the soil. Also the residual load and its effect on the load-settlement relationship should be considered.

The problems become more difficult when the problem to define the ultimate bearing capacity and the admissible load from pile load tests also is considered. Different definitions have been proposed based on e.g. a maximum total settlement, a maximum plastic settlement, a limited ratio plastic settlement/elastic settlement etc. This problem involves also the relationship between the general factor of safety and the probability of failure. Can we determine this relatioship?

When discussing friction piles also the phenomenon of "negative skin friction" or "down drag" which affects not only the load in the pile and the skin friction resistance should be considered. Which method should be used e.g. a total stress or an effective stress method to evaluate the negative skin friction? The relative displacement to mobilise fully the negative and the positive skin friction varies. The problem of negative skin friction is alone sufficient for an extensive discussion. It is mentioned here only as a part of the whole friction pile problem.

The prediction of bearing capacity of cast-in-place piles is influenced for example by:

The shape and the dimensions of the pile shaft (important e.g. for negative skin friction) and by the symmetry of the pile cross-section (location of the reinforcement). The calculation of the bearing capacity and settlements of cast-in-place friction pile is really a probabilistic problem both with respect to the soil conditions and the pile itself. Can we overcome this problem by using a reasonable prediction method?

Efforts have been made during the last years to determine the bearing capacity of piles under cyclic loading. The settlements of friction piles will generally be much larger for cyclic than for static loading. It is not yet possible to predict with sufficient accuracy the influence of cyclic loading on the ultimate bearing capacity. Thus the following questions may be raised:

1. What kind of investigations should be performed so that the influence of cyclic loading on pile bearing capacity, pile settlement and soil conditions around and under the pile can be considered?

2. How is the vertical or the horizontal stiffness of the pile-soil system affected when a pile is loaded cyclically?

The ultimate bearing capacity of friction piles is influenced by the sequence of loading particularly if the load change its direction from compression to tension. Answers to following questions are necessary:

1. How much do we know about the influence of the repeated loadings on the bearing capacity

of friction piles?

2. Is it necessary to introduce a residual skin friction when the bearing capacity is estimated?

3. Is it possible to explain the reduction of skin friction resistance in cohesionless soils due to repeated loadings by dilatation and contraction of the soil?

These questions indicate that many factors effect ultimate bearing capacity of friction piles and how difficult the problem is. Progress can be made only, I believe, when the work of all persons dealing with this problem is coordinated. The following suggestions are made:

1. To appoint a ISSMFE committee which will prepare a general program so that the relationships mentioned above can be determinated more accurately.

2. To suggest a coordinated program so that the questions raised above can be answered.

3. To request the National Societies to concentrate their pile research on certain problems.

4. To request the organizing committees of the different regional conferences to introduce as one subject a specific problem connected with the prediction of the ultimate bearing capacity of friction piles.

The collection of results by the National Societies could advance considerably our knowledge of the general relationships that govern the behaviour of friction piles to the next International Conference. In this way results can be obtained which would not be possible when everyone is working individually and without any coordination.

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O. Moretto, Panelist

LATERAL SKIN FRICTION IN STIFF CLAYS

My discussion will concentrate on skin friction in stiff clays and plastic silts, defined as those materials with an undrained shear strength larger than about 50 kPa. It will refer to the ultimate bearing friction capacity of both driven and bored piles determined by static methods.

While knowledge about the value of skin friction developed in soft to medium none sensitive clays has acquired a fairly reliable level, a large uncertainty exists when stiff clays are considered. This uncertainty is reflected in the extent of the spreading of the relation between skin friction and soil shear strength obtained from different test results at failure pile loading.

When total stress analysis is considered, the reduction coefficient to be applied to the undrained shear strength of the soil varies between 0.2 and a value somewhat larger than 1, according to the different authors who have reported pile tests results and/or analyzed the problem either in papers to this conference or elsewhere.

The same happens when an effective stress analysis is made. The values of the coefficient of shaft pressure K_s to be applied to match analysis with test results vary widely as well.

I will add to this confusion showing some test results of my own. They refer to cast in place and precast piles driven through soft into stiff clay and silty clay soils. The stiff soil is made up of redeposited wind blown material that has been preconsolidated by dessication. A very detailed site investigation was undertaken to determine soil properties. Sampling was performed with a 100 mm (4") fixed piston sampler for the soft clay and a 100 mm (4") double barrel Denison type rotary-push sampler for the stiff soil. A large number of triaxial tests were made to determine the strength parameters for skin friction analysis. High pressure chamber triaxial drained and medium pressure chamber consolidated undrained tests with pore pressure measurements were performed with the soil located below the pile tip for the point resistance analysis. Figure 1 shows the soil profile and the undrained testing results; Figure 2 and 3 the high pressure chamber and medium pressure chamber triaxial tests.

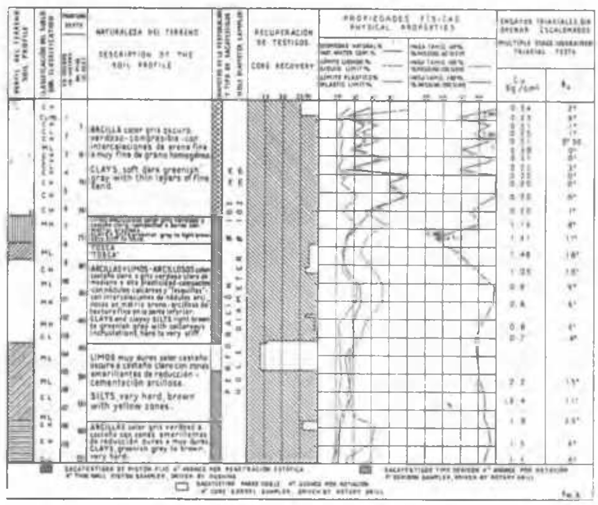

Fig. 1 - Soil profile, physical properties and undrained testing results.

Results will be examined for two of the piles tested, as representative of others as well. The first one is 37 cm in diameter, 13.50 m long, cast in place driving a casing close ended with a concrete point that is left in the ground when the casing is withdrawn while the hole is being filled with concrete placed with the help of a submerged vibrator. The second is precast, 30 x 30 cm, 8 m long.

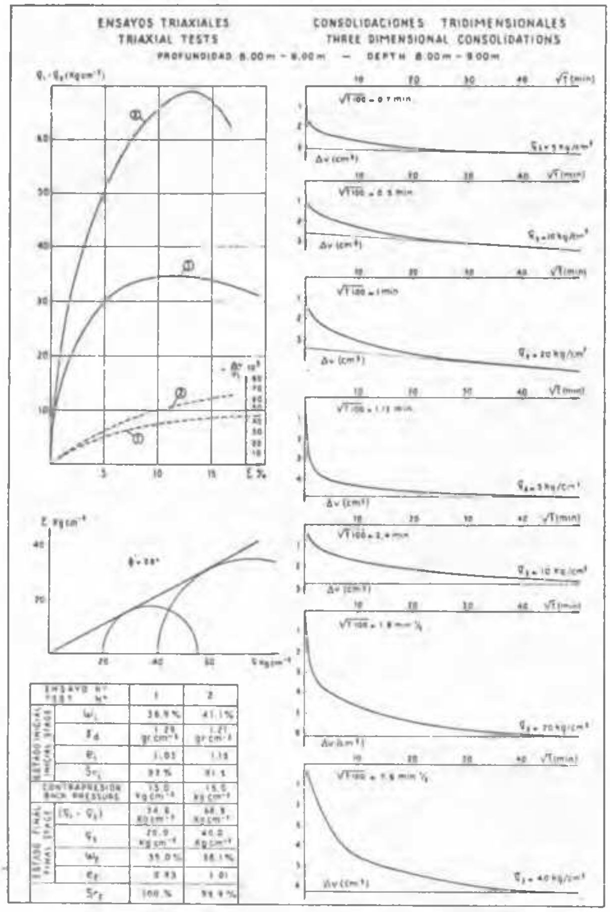

Fig. 2 - High pressure chamber triaxial tests of two typical samples of the stiff silty clay

Figure 4 shows the results of dynamic testings performed at the site of the first pile:

1) resistance to sampler penetration in a routine 75 mm (3") wash boring using a 50 mm (2") internal diameter sampler with plastic liners and a thin walled shoe which is driven into the ground as indicated in the figure; this is the most widely used method of sampling stiff soils in Argentina in routine subsoil investigations;

2) sounding with a 32 mm (1 1/4") drive rod with a cone tip;

3) pile driving record.

Figure 5 gives the same results for the second pile.

Figure 6 provides the results of the first test. Attention is called to the pull-out test results. A maximum resistance of about 130 metric tons was reached at first loading with a pile displacement of 15 mm. After unloading and reloading total friction decreased to about 80 tn.

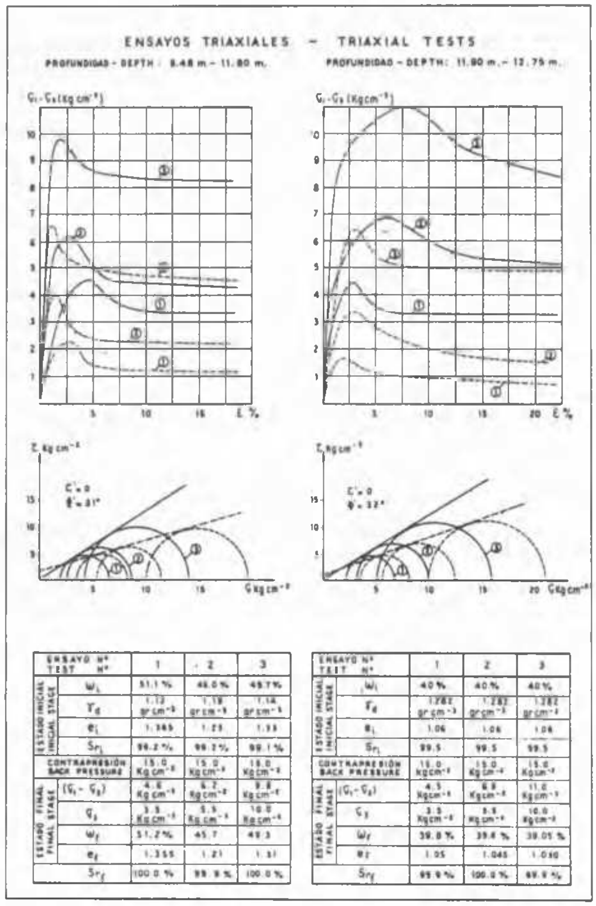

Fig. 3 - Medium pressure chamber triaxial tests on six typical samples of the stiff silty clay.

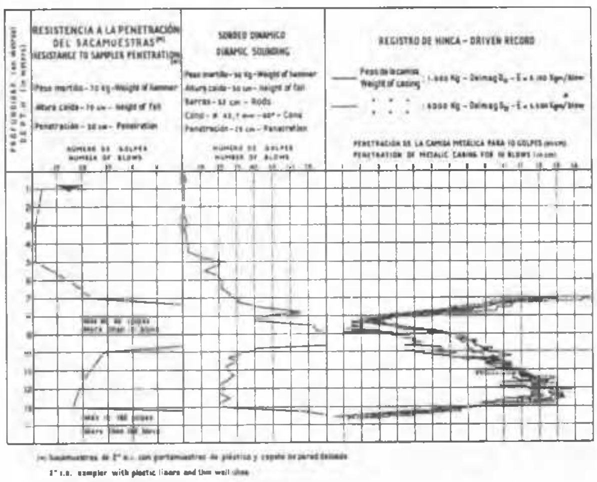

Fig.4 - Resistance to sampler penetration, dynamic sounding and driving record at the site of the first pile.

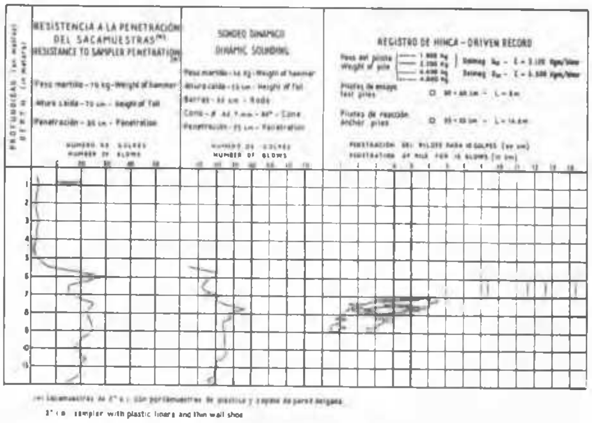

Fig.5 - Resistance to sampler penetration, dynamic sounding and driving record at the site of the second pile.

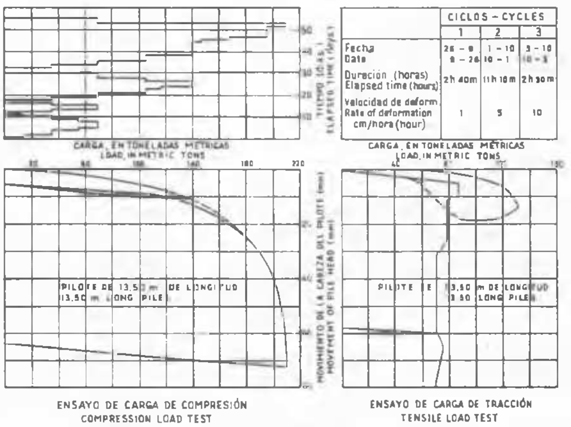

Fig.6 - Test results on the first pile.

Table I gives the point and friction resistance at failure calculated with total and effective stress parameters.

TABLE I

Friction and Point Resistance calculated with Total and Effective Stress Parameters

Point Resistance		Friction Resistance
$c_u : Ø_u$ (tn)	$c' ; Ø'$ (tn)	c_u (tn)
52	103	107

To match the maximum pull-out load measured, a reduction coefficient of the undrained shear strength of the stiff soil larger than one is required, while for the friction after unloading that coefficient lowers to 0.75. This last value would have been closer to one if the shear strength parameters had been determined on routine samples obtained driving the 50 mm (2") internal diameter sampler mentioned before, instead of carving them out of larger 100 mm (4") samples recovered by pushing a double barrel Denison type sampler.

Table I also includes point resistance calculated with total and effective stress parameters.

Figure 7 shows the results of the second test. Pull-out resistance reaches its maximum value for a 10 mm pile displacement and a load of about 100 tn. With further displacement, this load decreases to a much lower value of about 55 tn.

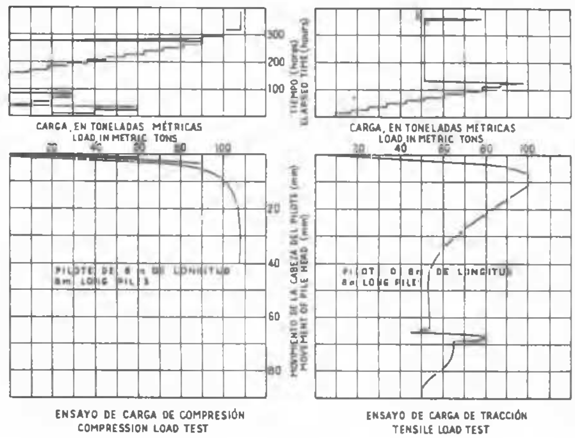

Fig. 7 - Test results on the second pile

Table II gives the point and friction resistance at failure calculated with total and effective stress parameters.

TABLE II

Friction and Point Resistance calculated with Total and Effective Stress Parameters

Point Resistance		Friction Resistance
$c_u ; Ø_u$ (tn)	$c' ; Ø'$ (tn)	c_u (tn)
12	35	63

To match the maximum pull-out load measured, a reduction coefficient of about 1.6 is required, while for friction after a very large displacement this value lowers to about 0.9.

On the basis of these and other field tests, we feel now rather confident that in the modified wind blown soils that underlay a large part of the Argentine plains around the city of Buenos Aires, which have been redeposited and preconsolidated by dessication, lateral friction at failure can be fairly well estimated by a total stress analysis with a reduction coefficient of one. Since we use the same coefficient for our none sensitive soft to medium clays, in our environs there appears to be no difference in response in lateral friction with clay consistency. In practice, on routine jobs, the shear strength of stiff clays and silty clays is determined on samples recovered by driving the 50 mm internal diameter sampler mentioned before. The results obtained are somewhat lower than on truly undisturbed samples. Therefore, on routine jobs, a reduction coefficient of one provides a calculated friction resistance which at failure is on the safe side and is applicable both to driven and bored piles.

This is not the case in other soil formations as shown by tests elsewhere. The well known investigation by Kerisel et al. (1965) at Bagnolet shows a highly decreasing reduction coefficient with increasing undrained shear strength. However, as has been stated by Kerisel (1973) himself, for the analysis of these tests, the values of c_u taken into account were those determined by vane tests. For stiff clays, vane tests values are often twice those obtained in triaxial tests. Had the analysis been made on the basis of triaxial tests results, the variation in reduction coefficients would have been much smaller.

The above considerations are pointing out that the different way in which the problem is approached may account for at least part of the existing uncertainties. Other factors may be involved, besides the soil interaction response. Sway during pile driving due to poor workmanship may affect skin friction in short precast piles by an unknown amount. The pile test results reported here give some indication of the very large importance that sway may have on lateral friction. The decrease in skin friction observed after loading and unloading to failure, or after reaching the maximum load, may be assimilated to the sway effect.

The nature of the pile material should have also an influence. Like in non sensitive soft to medium clays, steel piles should develop smaller lateral friction than concrete piles.

In conclusion, there is a great discrepancy in the test results performed in different parts of the world in different soil formations as to the lateral interaction between piles and stiff clays when the static method of analysis is applied. Similar discrepancies appear with other methods of analysis. A part of this discrepancy can be assigned to the different methods used for determining the soil strength parameters, another part depends on workmanship in pile driving and still another on the nature of the pile itself (steel or concrete). However, a sizable part appears to depend on the soil nature itself.

For bored piles in London clay, a reduction coefficient of 0.45 has been established after detailed and extensive investigations, while in the Buenos Aires modified loess we are counting on a coefficient of about one with a top value that for safety reasons we are at present limiting to c_u = 1 kg/cm2 = 100 kPa. Two factors may account for the difference:

1) the nature and structure of the soil deposit,

2) its initial state of stress.

The first factor would control the effect of surface remolding and internal drainage, the second one that of horizontal stress relaxation.

In any event, whatever the reasons for such an important difference, there is a general conclussion which can be stated with certainty: in stiff clays, there is no such a thing as a universal reduction coefficient nor a universal coefficient of shaft pressure. Soil formation, type of pile, method of installation and workmanship may influence such coefficients and therefore each situation should be evaluated locally in its own, by testing piles to failure.

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M. Maksimović (Oral discussion)

I would like to recall that Yamaguchi at Tokyo Conference had noticed that the ratio w/Q very often differs from the straight line which would yield the hyperbolic approximation of load-settlement curve. Some examples presented to this Conference, also, indicate that in some cases, the linear approximation does not hold in the whole interval of practical importance. At this point, I would like to introduce the following two assumptions:

1. After reaching yield, the shaft resistance becomes constant (nonlinearity + perfect plasticity), and

2. Inital portion of the base load-settlement relationship is linear.

In cases where the discussed plot is a straight line, the relationship between settlement and shaft and base resistances can be uniquely evaluated provided that one of the following unknowns is estimated or known:

a. Ultimate bearing capacity of the shaft or,
b. Ultimate bearing capacity of the base of the pile, or
c. The value of the settlement at which the shaft resistance begins the perfect yield.

Rather simple expressions can be evaluated in such case, but will not be shown here due to lack of space.
However, if the plot can be approximated with two straight lines, as in the example shown here, then it can be argued that at the displacement corresponding to the "kink" some significant change in the load transfer has taken place. If we take that at this settlement the shaft resistance arrived to its ulimate value which remains constant with further penetration of the pile, and assume smooth variation of the curve dQs/dw from inital value at zero settlement, to zero at yield of the shaft resistance, then the relationships between settlement and load components acting on the base and on the shaft can be uniquely derived.

The large diameter bored pile (D = 1.5 m. length 21.2 m. embedded in sand SP/SW in upper zone and SW with SPT resistance of N=6o at the base) shows in this interpretation typical bi-linear pattern as shown in Fig.1.

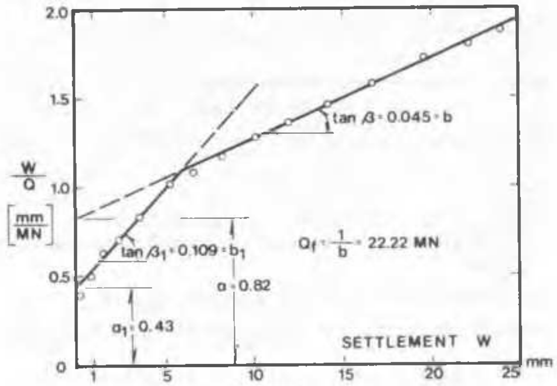

Fig. 1.

Following the analytical procedure described here in words for brevity, curves relating base and shaft resistances to pile setlement are computed and shown in Fig.2.

The pile loading test was carried out in a conventional manner, and the pile was not instrumented except for normal testing and it would be of primary interest to check the validity of the arguments presented here. Check in an approximate manner, is made for the ultimate values of base and shaft resistances with following results:

Shaft: $\tau_{av} / \sigma'_{vav} = 0.289$

This value is within the range ussually adopted for computation of shaft resistances of bored piles in granular soils.

Base:

Q_{bf} (assymptotic) = 1.1 Q_{bfM}

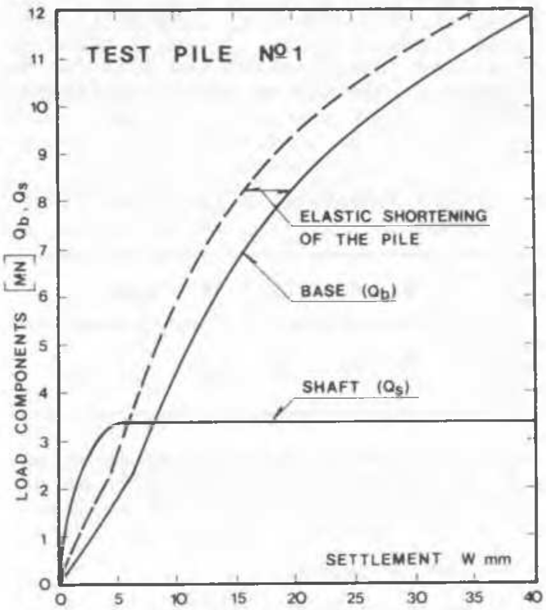

Fig. 2.

where Q_{bfM} represents the value computed by the method proposed by Meyerhoff in his well known Therzaghi Lecture.

At this stage we are normalizing curves obtained in this manner and using them in analysis of piles in conditions where the loading differs from the one in the case of a particullar test pile, like the negative skin friction,For this purpose, the appropriate computer program is developed and used in several practical cases.

R.K. Katti (Oral discussion)

I am enumerating certain observed behaviour of large diameter piles and small diameter piles in 2-3 locations in Indogangetic plane having fine to coarse sand deposits. On several jobs the ultimate pile load capacities are calculated based on either Berezantsev's coefficients or Meyerhof's coefficients for N_q. Only in the above 2-3 jobs we recommended use of 76 cm. diameter and 100 cm. diameter piles under columns having heavy loads. When the pile load tests were conducted on piles having diameter less than 56 cms, computed values of ultimate load capacity, factor of safety etc. with respect to structural strength etc. tallied closely with the computed values. However, for the same depth and same condition of bottom material the load settlement data with respect to ultimate load was alarmingly low. See Fig. 1. Even the computed ultimate strength was around 1.8 to 2 time structural strength, the load test values were much less. However, for $\frac{H}{D}$ > 24 or so there is some improvement.

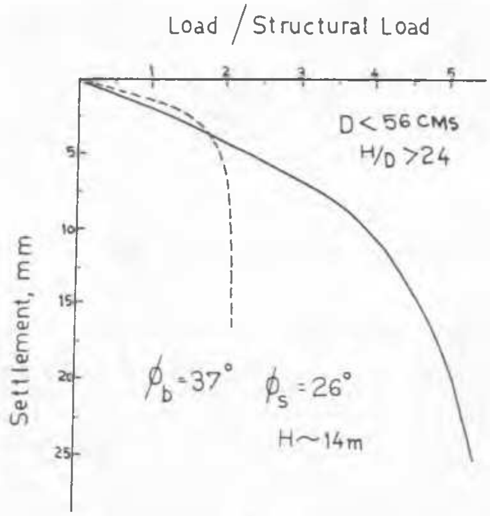

Fig. 1.

The chairman, co-chairman and Prof. Meyerhof may kindly give their interpretation on this

type of behaviour of large diameter piles. This has started affecting competetiveness of large diameter piles v/s small diameter piles in the same location and also for the same depth. This may be due to stiffness of the pile.

Safe Structural Strength of Piles

Shaft, dia cms.	40	45	50	76	100
Structural Strength, Q, tonnes	56	71	88	204	347

Ultimate strength of above diameter piles at around 14 m depth Ø' at bottom 37°, based on SPT and side $\emptyset_1$ value around 25° to 28°.

Shaft, dia	40	45	50	76	100
Safe structural load capacity Q_{safe}	56	71	88	204	347
Q_{ult}	170	210	260	380	630

Questions :

(i) Why large diameter piles behave differently compared to small diameter piles ?

(ii) Is there a need to change the N_q coefficients of Berzentsev and Meyerhof.

M. Vargas (Oral discussion)

The purpose of my intervention is to propose a way to use ordinary static load test on piles, made according to a carefully specified procedure, not only to determine the allowable loads, but also to separate point and lateral resistance, and evaluate the soil "deformation modulus" necessary for pile foundation settlement computations.

I think that this method, if proved reasonable, will be very useful in the developing countries where a large number of construction is going on, but sophisticate instrumentation is not always available.

In order to do so, I would like to suggest a very simples way to evaluate the point and lateral resistances of a pile by means of an ordinary load-test. Taking into account the results of load distribution, observed along the shaft and the base, during loading of instrumented piles, as shown on figure 1, it is possible to define the following two coefficients:

$$\alpha = \frac{Q_a}{Q_p} \qquad \beta = \frac{\int Q_z dz}{Q_a L} \tag{1}$$

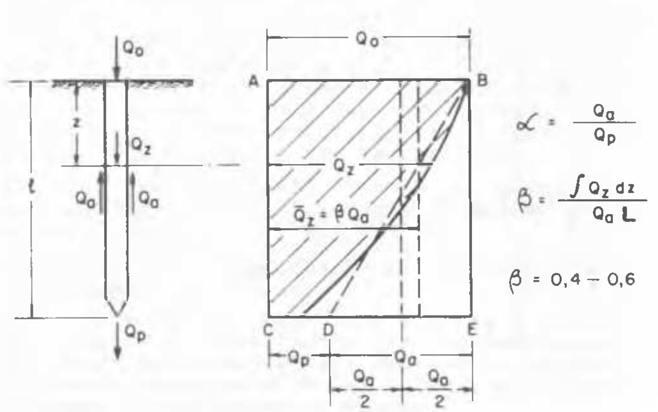

Fig. 1 Load transfer from a single pile

Where Q_a is the lateral resistance; $Q_p=Q_o-Q_a$, point resistance; and Q_z the load transmited from the upper to the lower section of a pile, at a depth z. The value of Q_o must be limited to less than that which corresponds to both ultimate values of point resistance and skin friction.

These two coefficients would define the load transfer from the pile to the soil. A statistical analysis of the published results of pile load transfer observations shows that the value of β can vary from zero to about 5. However, in great majority of cases, it falls between 0,3 and 1,5, with an average of about 0,4 to 0,6. The variation of the coefficient α is much more ample: from 0,6 to 15, with no apparent mean value.

Figure 2 shows a load-settlement curve obtained from a load-test on a pile, suggesting an approximate way to evaluate the elastic compression on the pile itself, by means of drawing a straight line, following the curve of discharge of the pile, from its working load to zero. The difference between the total

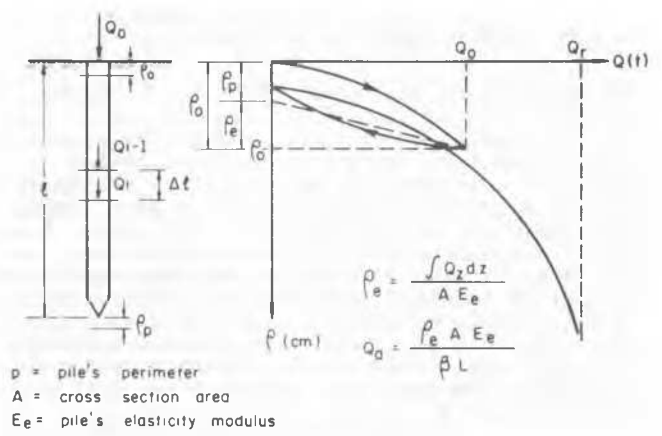

Fig. 2 Load-Settlement curve from a load test

settlement of the pile, at its working load, and the ordinate at zero load,on the straight line, as defined above, is obviously the elastic compression of the pile ρ_e.

Knowing the elastic compression and the modulus of elasticity of the pile material, it is possible to evaluate, approximately, the value of the lateral resistance of the pile Q_a, by means of formula 2. The value of β can be adopted more precisely if information is loccaly available; if not an average value of 0,6 can be used with a high probability to be correct.

$$\rho_e = \frac{\int Q_z dz}{A\, E_e} \quad \therefore \quad Q_a = \frac{\rho_e \cdot AE_e}{\beta L} \qquad (2)$$

where A is the cross section, L the length and E_e the modulus of elasticity of the pile respectively.

But the easyer way of estimating Q_a is by assuming that Q_z-curve, in fig.1, can be substituted by a streight line BC; then it would be easy to show that (in that case: $\beta = \frac{1}{2} + \frac{1}{\alpha}$):

$$Q_a = 2\left(Q_o - \frac{\rho_e\, A\, E_e}{L}\right) \qquad (3)$$

Once the values of Q_a, lateral resistance, and consequentially Q_p, point resistance, are known, two coefficients of reaction, respectivelly: of point resistance k_p, and lateral resistance k_f, can be defined as:

$$k_p = \frac{Q_p}{\rho_o - \rho_e} \qquad k_f = \frac{f_z}{\rho_z} \qquad (4)$$

i.e.: k_p is the ratio between the applied load and the settlement of the pile tip (diference between the total settlement and the elastic settlement) and k_f is the ratio between lateral resistance, per unit area of the shaft, and deformation of the pile shaft at depth z.

Since piles are, commonly driven through a soft surface layer until the tip reaches refusal at a hard substratum, it is assumed that the ground around a pile is composed of an upper layer with a modulus of deformability - E_f along the shaft of the pile, and a substratum - of higher stiffness, with a modulus of deformability - E_{ps}, below the tip of the pile.

If E_f is considered constant along the shaft of the pile, k_f would be also constant (see formula on fig. 3). In that case:

$$k_f = \frac{Q_a}{2\pi r L \rho_o} \qquad (5)$$

By means of formulas, derived in accordance with well known theories, as shown in fig.3, knowing the two above mentioned coefficients of reaction, k_p and k_f, it is possible to evaluate the "modulus of deformation" of the soil, around the shaft of the pile, E_f, and that corresponding to the ground just below the of the pile, E_p, respectively:

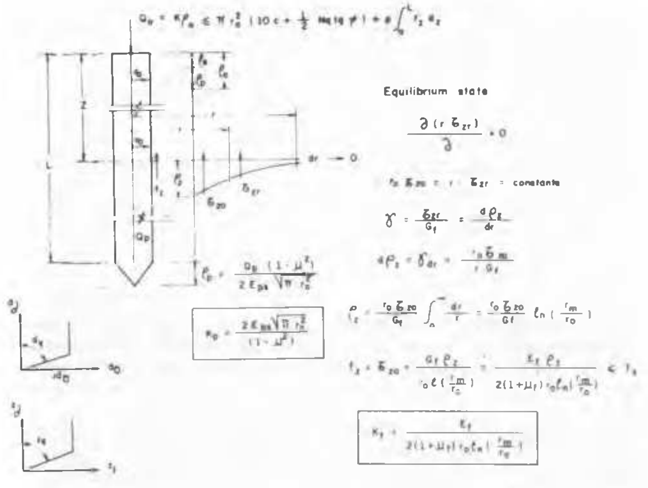

Fig. 3 Stress - Deformation around pile

$$E_{ps} = \frac{k_p(1-\mu_{ps}^2)}{2\sqrt{\pi r_o}} \qquad E_f = 2k_f(1-\mu_s) r_o \ell_n\left(\frac{r_m}{r_o}\right) \qquad (6)$$

Both values of the Poisson's coefficients μ_{ps} and μ_f can be taken as 0,5 and the ratio $\left(\frac{r_m}{r_o}\right)$ as egual to 3.

This procedure to evaluate soils "modula" (to be used, for instance, in the computation of the settlement of a pile foundation) seems to be more realistic than by laboratory tests on undisturbed samples. At least all the particular construction conditions and soil disturbances, due to pile driving and instalation, would be reflected in the results of a pile tests; what would not be the case as by means of laboratory investigation.

Shi-shen Lu (Oral discussion)

AN INVESTIGATION ON THE p-y CURVE METHOD

In a testing pile programme, I happened to measure the deflection curves and soil reactions of some piles. The displacement transducers and earth pressure cells were installed manually by myself in the boreholes. All the earth pressure cells used were calibrated in near-by bore holes prior to installing in the boreholes for the testing piles, satisfied results were obtained.

The deflections y_z at various levels along the pile shaft were measured during the application of the lateral loads H. The Fig. 1 below shows the H - y_z curves for the vertical pile No. 6.

It may clearly be observed in the figure that there are non-linear relationship between the lateral loads and the deflections at various levels except at depth beyond Z= 3.0 m., where the pile deflections are shown in direct proportion to the lateral loads. It was found in a previous analysis that the maximum bending moments of a pile were in direct proportion to the lateral loads provided the design load of the pile was not exceeded. Therefore, it may be concluded that there is a non-linear relationship between the maximum bending moments

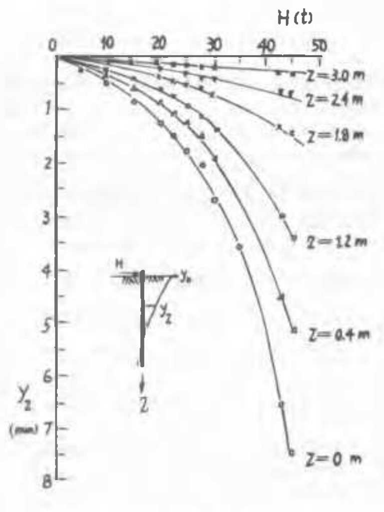

Fig. 1 H - y_z curves

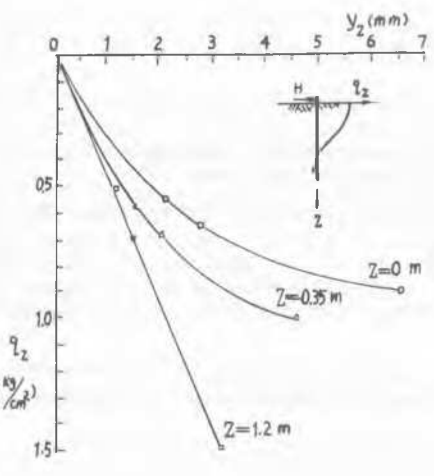

Fig. 2 q_z - y_z curves

and the deflections of the pile shaft for the upper part of the earth in front of the pile. It is observed also that the shapes of the H - y_z curve become more and more flater and the curve turns into a stright line at a certain depth, for instance, at Z= 3.0 m. for pile No. 6. This indicates that the plastic zone turns gradually into elastic one as the depth increases.

According to the measured values of pressure cells installed along the pile shaft, a set of curves as shown in Fig. 2 is plotted for three levels, namely Z=0 m., Z= 0.35 m., Z= 1.2 m., in which the values for Z= 0 m. are evaluated by means of the q_z - Z curves previously determined.

It is obvious from the curves that there is a zone of at least 0.35 m. thick within which the pile-soil system behaves elasto-plastically even though the pile deflections do not exceed about 1-2 mm.(for large diameter bored pile), It can also be noted that as the depth increases, the elastic behaviour becomespredominent, i.e., a linear relationship between q_z and y_z exhibits for depth Z equal to or greater than 1.2m.

It seems probable that even though the pile itself may behave elastically for loads smaller than the design load, or in other words, in the case that the bending moments are proportional to the lateral loads, a plastic zone does exist in the upper part of the earth from the very beginning of the application of the lateral load. I quite agree that in the analysis of a laterally loaded pile, the p - y curves are justifiable to represent the elasto-plastic behaviour of a pile-soil system.

A.S. Vesic, Panelist

BEHAVIOR OF PILE GROUPS

In spite of great progress achieved in the last ten years in understanding the pile-soil interaction, the load transfer phenomena involving even a single pile under the vertical loading are at best only partially understood. When it comes to load transfer phenomena in pile groups our knowledge is quite limited. The old efficiency formulae are largely speculative in nature, being based on incomplete observations and without a solid theoretical foundation. The efficiency charts based on model tests with pile groups place undue reliance on tests performed with very small piles. There are almost insurmountable difficulties in attempting to scale these observations up to the full-size range, particularly of the soil in question consists of a granular material, such as sand.

In previous considerations of this subject it was shown (Vesic, 1964, 1967) that comparing a nail size pile model with a large-size prototype in an identical, homogeneous sand mass, apparent ϕ-angle of sand at the small scale may be 10° higher, the apparent deformation modulus 30 times lower, and apparent rigidity index 30 times higher. If the pile material is the same, the small model pile may be 30 times more rigid with respect to the soil than the large pile. Sand under the pile tip may expand in shear at small scale and contract in shear at the large scale. The point resistance of the small model will be fully mobilized before the shaft resistance reaches its peak, while at the large scale the shaft resistance is fully mobilized before the point resistance starts being of any significance. Similar, though not as pronounced effects exist in the case of piles in clay. Altogether load transfer phenomena bear little similarity at two different scales. It should be added that testing in a centrifuge eliminates some but not all of these effects.

In recent years there have been numerous attempts to develop a consistent general theory of load transfer in single piles as well as pile groups, considering the soil mass as an elastic-isotropic solid. Following some early work by Pichumani and D'Appolonia, Jimenez Salas and others, Poulos and his co-workers have presented "solutions" for square and rectangular pile groups for a range of cases, examining the effects of group size, pile-spacing and length-to-diameter ratio of piles. Their work was further extended to the so-called Gibson soil. This approach, however consistent and pleasing to the mind, has some very serious shortcomings, that have been mentioned in the literature on several previous occasions (Vesic, 1970, 1977). Perhaps the most serious of these is the assumption in all these calculations that stress transfer in the pile-soil system starts from a completely unstressed state. According to this approach the load distribution between piles in a group and the group settlement factors are not affected by the method of construction. This contradicts known observations on full-scale groups and large scale models.

Thus, summarizing our present knowledge about behavior of pile groups, our primary source of information is the evidence from less than a dozen full-scale tests reported in the literature, not all of which had very elaborate instrumentation programs. A few apparent facts of general validity can be summarized as follows:

(a) the ultimate point load of a pile group is approximately equal to the sum of ultimate point loads of individual piles;

(b) the ultimate shaft load in soft clay cannot be larger than the sum of ultimate shaft loads of individual piles multipled by the ratio of outer perimeter of the group to the sum of perimeters of individual piles;

(c) in comparing shaft loads of piles and pile groups in cohesive soil, it is important to keep in mind that the dissipation of pore-pressures set by pile driving is slower for groups than for individual piles. The time needed for development of full capacity of a pile group may be considerably longer than that for single piles;

(d) the ultimate shaft load of a pile group of piles driven in sand or stiff clay may be larger than the sum of individual shaft loads. This is explained by the increased lateral compression caused by driving a greater number of piles within a relatively small area. The sequence of driving is important in this regard and there is evidence that later driven piles have higher capacity than these driven earlier. This effect appears to be more pronounced in loose sands; however the effect is nonexistant of possibly reverse in the case of bored piles;

(e) pile caps contribute to overall bearing capacity of the group to the extent that they are supported by competent soil outside the outer perimeter of the group;

(f) driving or jacking adjacent piles reduces the residual load of previously driven piles. This results in changes in distribution of skin resistance along pile shafts and may cause significant group settlement effect in predominantly shaft-bearing (friction) piles.

(g) Group settlements are always larger than those of individual piles carrying the same load per pile.

While the understanding of these facts has added enormously to our understanding of pile group behavior, there has been little in the way of improvement of our methods of analysis that would add to our ability to predict with some precision the actual performance of a group. A recent experience illustrates very dramatically the extent to which our ability to predict group behavior lags behind our ability to predict single pile behavior.

A series of loading tests on instrumented groups of 27cm diameter closed-ended steel pipe piles was recently completed on a site in Houston, Texas, under sponsorship of the U.S. Federal Highway Administration (O'Neill, 1980). The testing program consisted of six single pile compression tests, six single pile uplift tests, three nine-pile-group compression tests, as well as two subgroup compression tests, involving, respectively, five and four pile groups. All piles were driven through 6m deep, 20 cm diameter predrilled holes to a total depth of 13m in stiff clay and clayey silt deposits of the Beaumont and Montgomery formations. The piles in a group were joined by a reinforced concrete cap 1.30m thick suspended 0.90m above the ground. Details of elaborate pile and site instrumentation, which include strain-gauge circuits, extensometers, inclinometers, lateral pressure cells, piezometers and displacement gauges situated at different levels along the pile axis, can be found in the appropriate research report (O'Neill, et al, 1980).

Along with the testing program the FHWA contracted 11 foundation specialists (3 from universities, 2 from highway departments and 6 from prominent consulting firms) to make complete predictions of load transfer behavior of both single piles and the main nine-pile group. These predictions were delivered to FHWA prior to the completion of the testing and presented at a symposium held in College Park, Maryland in June of 1980. The prediction were followed by presentation of actual results of the test series, and a summary analysis comparing the predictions with experimental facts.

A summary of predicted single pile capacities is shown in Fig. 1, where the measured capacities are also shown. It can be seen that, in spite of great scatter of predictions, the average of all predicted capacities agrees quite closely with the measured values. A comparison of predicted and measured group capacities is shown in Fig. 2. It is obvious that the scatter of predictions of group capacities is wider than that for single piles; this can be explained by the lag in development of understanding of group behavior in general. Also, it is apparent that the averages of predicted capacities are slightly higher than the measured capacities, indicating a tendency in currently available methods toward overprediction of group capacities.

Looking at the spread of predicted versus observed capacities we have to ask ourselves whether the conventionally used safety factors leave enough room for a satisfactory design. This is a consideration that each individual designer has to consider for himself; however experience calls for some reservation about using safety factors of 2 with any method other than that based on measured resistances by means of a static cone penetrometer.

In spite of this and some other concerns, most predictions presented here are very good indeed. In making this judgement we must consider that, by the nature of things, predictions of behavior of deep foundations compared with similar predictions for shallow foundations possess an additional degree of complexity. In

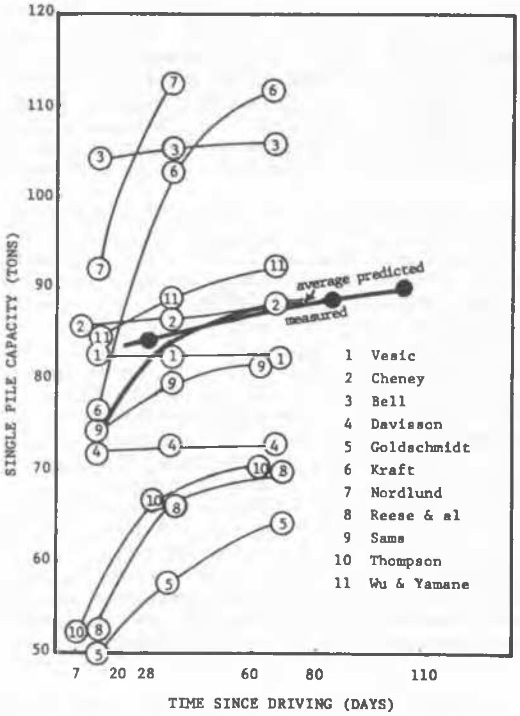

Fig. 1 - Comparison of single pile predictions

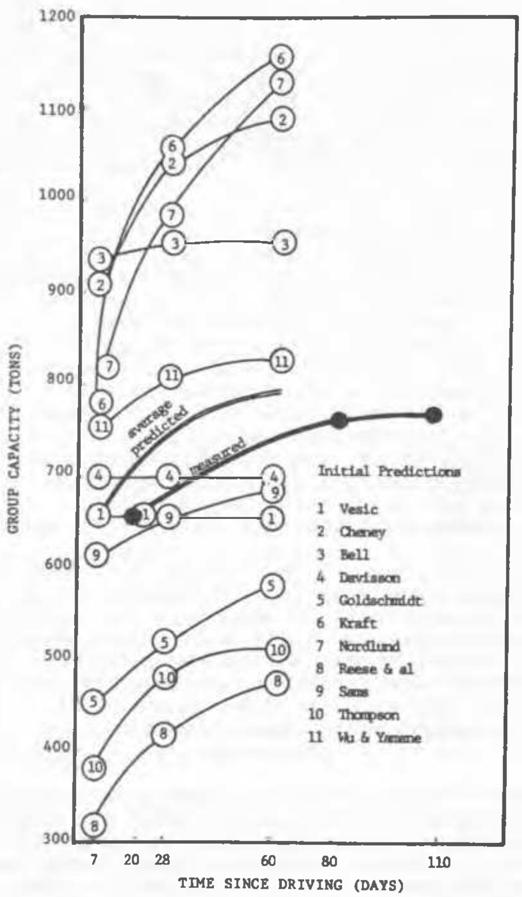

Fig. 2 - Comparison of group pile predictions

the case of shallow foundations we are dealing with uncertainity of proper evaluation of soil properties in presence of a variability provided by nature through the soil formation process; however we have relatively little to worry about the effect of construction procedures on bearing capacity and settlement and we can certainly determine stress changes in the soil mass produced by the construction process with relatively high accuracy. Not so in the case of deep foundations, where small variations in construction process may have major effects both on stress state in the ground and on structural behavior. Those working with probability in foundation design will recognize immediately that, in presence of two simultaneous uncertainties, the predictions for deep foundations should not be expected to be as good statistically as these for shallow foundations.

The preceding analysis provides, no doubt, a new insight into the state of the art of prediction of behavior of deep foundations. It is gratifying to find that our ability to predict appears to be better than most of us thought, certainly far better than it used to be just 10 to 20 years ago. A second significant finding which confirmed our earlier observations, is that piles in the group plunged individually into the ground, with no signs of speculated "block failure". Also, the behavior of these piles and pile groups in stiff clay was frictional. Measurements showed no major pore pressures generated by pile loading. Some were generated by driving, however they dissipated rather quickly and accounted very little for some observed increases of pile resistance with time. The observed distribution of shaft resistance with depth show, in this case, as in other cases of piles in stiff clay an almost linear increase with depth.

Next we should mention the importance of residual loads, which were again recorded in this study. Predictions did not turn out too well. It is obvious that we need to improve our methods of analysis of these loads, essential for proper prediction of pile load-settlement characteristics. This is particularly true for situations where loads are cyclic and the entire design rests on computed displacements. Examples of this kind appear almost daily in design of large off-shore drilling platforms, where the consequences of potential failures are extremely disastrous.

It was most instructive to find also how the load eccentricities may affect the group behavior in a very significant way. This experience points out to the importance of accuracy in construction, which in view of gradual increase of pile working loads in practice has to be more strict than we have been assuming in the past. It is somewhat disturbing that these "research" piles, driven through predrilled holes apparently had batters as large as 20 to 1, when they were supposed to be built as vertical. This points out to the need for closer construction control by the owners and designers, to insure that the actually placed pile groups indeed resemble those on construction plans.

A final remark is related to the need for caution in use of some of the unproven theories in deep foundation design, and particularly those that make extensive use of the Mindlin solution without descrimination. As was shown in test results presented by O'Neill (1980), the settlement factors from such theories are not even close to those observed in this study. This should come as no surprise if we consider how most of the tables and charts with group settlement factors are developed. Driven piles are often considered as just mathematical lines without due consideration of the process of pile placement in the ground. The oversimplification involved works sometimes, but may bring considerable errors in other situations. Thus, caution in applications of these computations is needed before improved theories are developed which would be free of such weakness.

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E. Togrol, Panelist

APPLICATIONS OF PILE GROUPS TO RESIST LATERAL LOADS

Piles are generally used as a group or a cluster. Yet it is the behaviour of single piles which is much more extensively investigated. The number of case records, measurements, test results available on the behaviour of pile groups, especially of those subjected to lateral loads is very limited. However, large lateral loads are to be resisted by pile groups as in the cases of piled foundations to bridge piers, trestles to overhead cranes, tall chinmeys,piles in wharves and jetties, piles used on hillside areas to arrest movement or slides.

Holloway et al in a well documented paper to this Conference (8/20) describe a lateral load test on a full scale pile group driven in loose to medium sand. They observe that the front piles are "punching" into the soil while the soil mass within the group tracks the piles to some degree. A computer analysis assuming the soil as an ideal elastic material with a constant modulus of elasticity was also carried out. The differences between the observed and predicted values of letaral deflections and moments indicate the need for a better understanding of load transfer behaviour within the soil-pile system. Whatever the approach and test design adopted for such a study, any attempt to trace relationships for the soil-pile system is bound to come up against formidable problems.

A further question concerns the practical assumptions to be adopted when seeking a satifactory design at a minumum expenditure. In this connection I would like to describe some successful applications of pile groups to resist large lateral loads. Bored and cast-in-place piles are widely used in Turkey, and during the recent years it has also been common practice to employ them for slope stabilization or as retaining structures. In estimating the resistance to lateral loads of the pile group the conditions (i) to secure the soil mass between the piles to "track" the group's movement, (ii) to create soil arching behind the piles were taken into account (Çamlibel, 1981).

A slide started to develop in 1955 at a critical section of Istanbul-Ankara railway near Tuzla. A number of remedies applied till 1961 without long lasting success. An extensive investigation carried out then by Professor Peynircioğlu showed that the location of the slip circle was seated in the heavily overconsolidated silty clay. The liquid and plastic limits of the clay were $w_L = 0.65$, $w_p = 0.28$ and the unconfined compressive strength q_u 4.6 kg/cm^2. The strength values obtained in laboratory were even higher. Yet the average shear strength along the slip circle was calculated to be $s_u = 0.1 - 0.2$ kg/cm^2. Attempts to control the ground water level was not completely successful since the lowest point on the slip circle was almost at the sea level and the location of the slide was only few hundred meters to the sea. Then it was decided to apply a structural remedy.

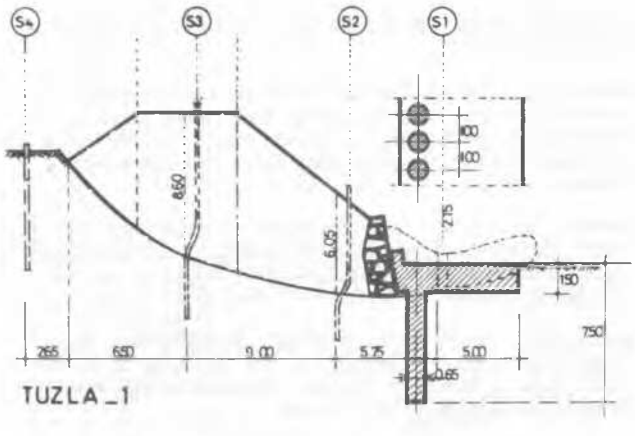

FIG. 1 - *Piled retaining structure at Tuzla.*

A piled retaining structure with a 5 meters wide top slab was formed in front of the already existing masonary wall (Fig. 1). At a section where soil conditions were found most unfavourable the structure was reinforced by a second row of piles (Fig. 2). Lateral loading tests were carried out to show that there exists sufficient factor of safety. The design was applied on a length of about 150 meters along the railway embankment and the movement was completely arrested.

Major fault lines in the area, make Bursa a town with high earthquake risks. In the construction of a hospital in Bursa in 1967 the problems arising from this first disadvantage are compounded by the second: the reports of active slides in the neighbourhood. A careful soil investigation revealed that the short term factor of safety of the site could be as low as 1.11. A structure based on bored piles was placed at the toe of the slope and the factor of safety of the slope (on which hospital buildings C1 to C4 rests) was reportedly increased to 1.40 (Fig. 3). The construction was successfully carried out.

A series of slides began in 1963 after a comparatively long period of rest in an area of size 230 meters by 400 meters near Istanbul airport (Peynircioğlu, 1969). Slides caused continual damage to the railroad and the road placed at the toe of the slope. An extensive soil investigation was carried out in order to understand the pattern of the slides. Existence of several old slides in different directions

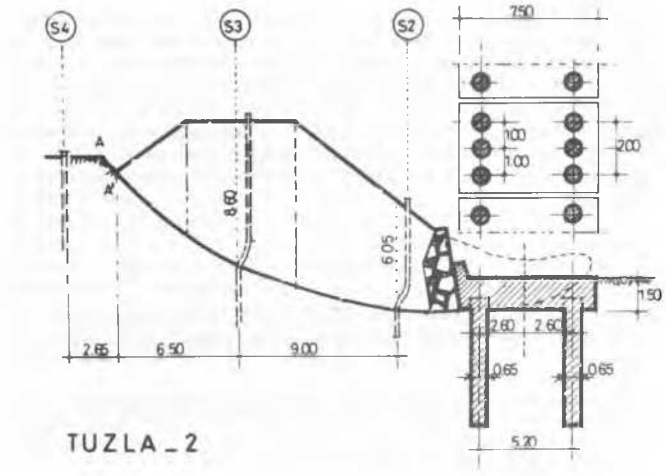

FIG. 2 - *Piled retaining structure at Tuzla.*

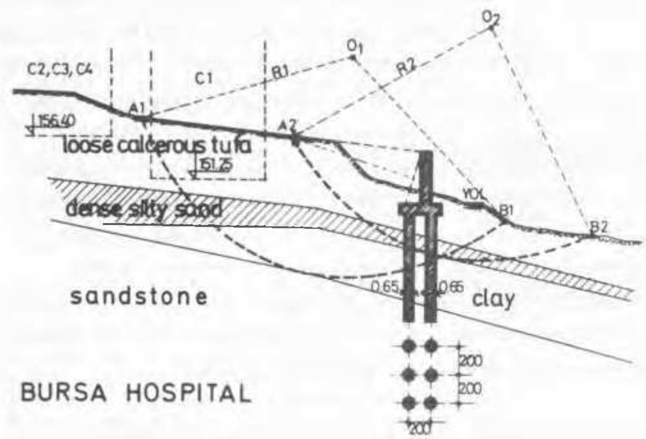

FIG. 3 - *Piled retaining structure at Bursa.*

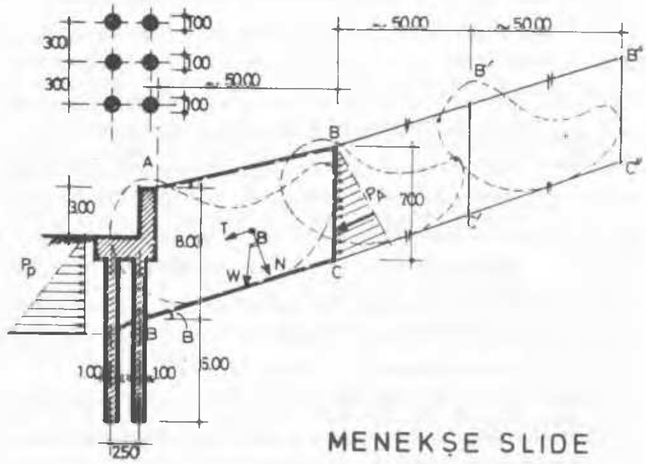

FIG. 4 - *Proposed piled retaining structure as a preventive measure at Menekşe slide near Istanbul airport.*

and frequent changes in the regime of the underground seepage complicated the situation. One important aspect of the slide was the almost constant depth of the affected soil mass. And construction of a retaining structure at the toe of the slope was proposed (Fig. 4).

In a more experimental kind of application piles were used as a permenant retaining structure together with a reinforced concrete top wall in a building excavation (Figs 5, 6). In order to provide space to build an office block for a bank it was necessary to carry out an excavation at the toe of a slope in a thickly populated central part of Istanbul. The soil profile consisted of fill and highly weathered graywacke which is underlain below the excavation level with an unweathered graywacke of good quality. The design assumption was that the rigid cap would provide the necessary rigidity to secure the piles and the soil between the piles behave as a "monolith". The periodic displacement measurements made after the construction had shown the success of the design.

FIG. 5 - View of the retaining wall built as a rigid cap on top of the piles at Salipazari.

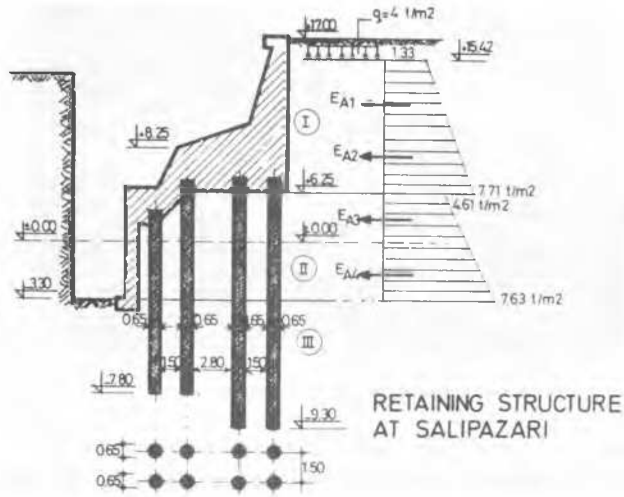

FIG. 6 - Retaining structure at Salipazari.

A similar design was applied at Zeyrek in another thickly populated part of old city in Istanbul (Fig. 7). The soil profile above the ground floor level consisted of heterogeneous fill. Lateral load tests on single piles were made to determine allowable loads. The construction was completed in 1967 and its performance caused no complains.

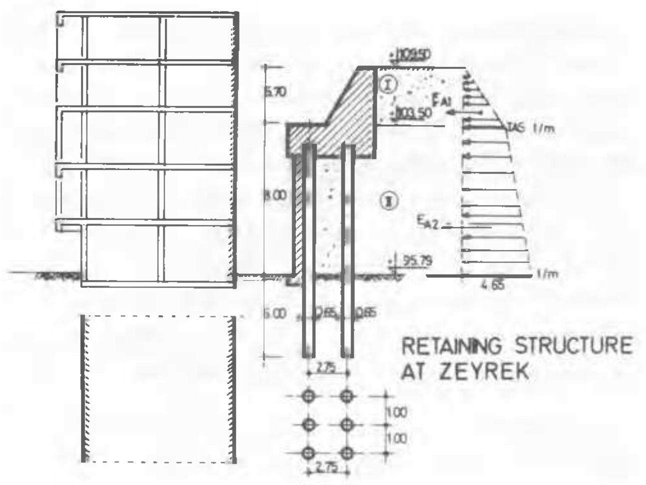

FIG. 7 - Retaining structure at Zeyrek.

As a last example I shall summarize the efforts made to stabilize a talus slide in a remote part of Turkey, Eğirdir. The construction of a hospital complex was started there in 1970 and had had to be interrupted two year later after extensive slides occured and caused heavy damage in some of the incomplete buildings (Fig.8).

FIG. 8 - Eğirdir hospital construction.

The soil profile consists of talus material consisting one meter diameter rock pieces. Ground water level was not encountered in exploratory borings. The slide seemed to have started after a toe excavation leaving an unsupported cut of about 20 metetrs high. In order to arrest the movement a series of measures were attempted. To prevent temporary saturation of the slope a drainage system including deep drains was planned. At critical sections stonewalls on the slope were foreseen to increase the resistance to sliding. And thirdly, groups of piles were planned at the toe of the slope as a last resort. The stabilization project was interrupted after one year of work and only one third of what was planned was achieved. The work has not been resumed until this year. In evaluating the results of the incomplete measures it was interesting to note that piles functioned much more effectively than the other preventive measures (Fig. 9). One of the essential remedies to control the Eğirdir slide is now considered as to install sufficient number of pile groups at the toe and on the slope.

Such examples could possibly be given for other areas of the world with different soil conditions. Pile groups seem to be

FIG. 9 - The slope behind the Eğirdir hospital construction. The piles at the toe of the slope successfully arrested the movements.

conveniently used as a means of support whereever the ground appeared incapable of resisting lateral loads. A design principle may well be to ascertain the strain compatibility for the piles and the soil between them under the influence of service loads. And it is always advantageous to take into account the soil arching behind the pile group. Certainly, we should know more about the soil-pile behaviour for the pile-groups in order to set more refined principles of their design. So in conclusion, I would like to emphasize the need for measurements and data concerning the behaviour of pile groups subjected to lateral loads.

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H.G. Schmidt (Oral discussion)

SETTLEMENT MEASUREMENTS ON 3 PILE GROUPS
Mesures des Tassements de 3 Groupes de Pieux

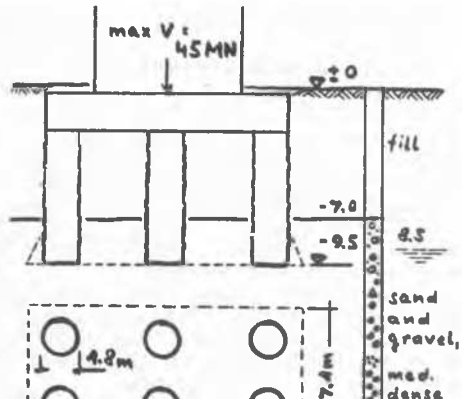
According to the directions given in German Standard DIN 1054, settlement predictions for pile groups involve two components: (1) settlement of single pile, derived from pile load tests, and (2) settlement of the pile group, which is calculated for a fictive spread foundation. For three pile groups, settlements predicted in this way are compared with measured values.

(1) Chimney of Thermal Power Plant at Mannheim, 200 m high (see fig. 1a).
Maximum load = 5.9 MN per pile,
from deadweight: 2.4 MN per pile.
Calculated settlement: 1 cm (from single pile; group action was thought to be negligible).
Measured settlement: 0.5 cm.

(2) Stair case tower of Thermal Power Plant at Mannheim, 100 m high (see fig. 1b).
Maximum load = 7.4 MN per pile,
from deadweight: 2.75 MN per pile.
Calculated settlement: 2.4 cm (1.0 cm from single pile, 1.4 cm from group action).
Measured settlement: 1.0 cm.

In both cases, maximum pile loads result from wind forces, and actual loads can be estimated only.

(3) Pier of a highway bridge at Köln (fig. 1c). In this case, the maximum load occured during the construction procedure and was due to deadweight alone.
Maximum load = 7.5 MN per pile.
Calculated settlement: 3.4 cm (2 cm from single pile, 1.4 cm from group action).
Measured settlement: 3.2 cm.
This agreement of predicted and measured settlements is to be considered as an accident. Prediction of pile group settlements still depends to a large degree on experience, engineering judgement, and good luck.



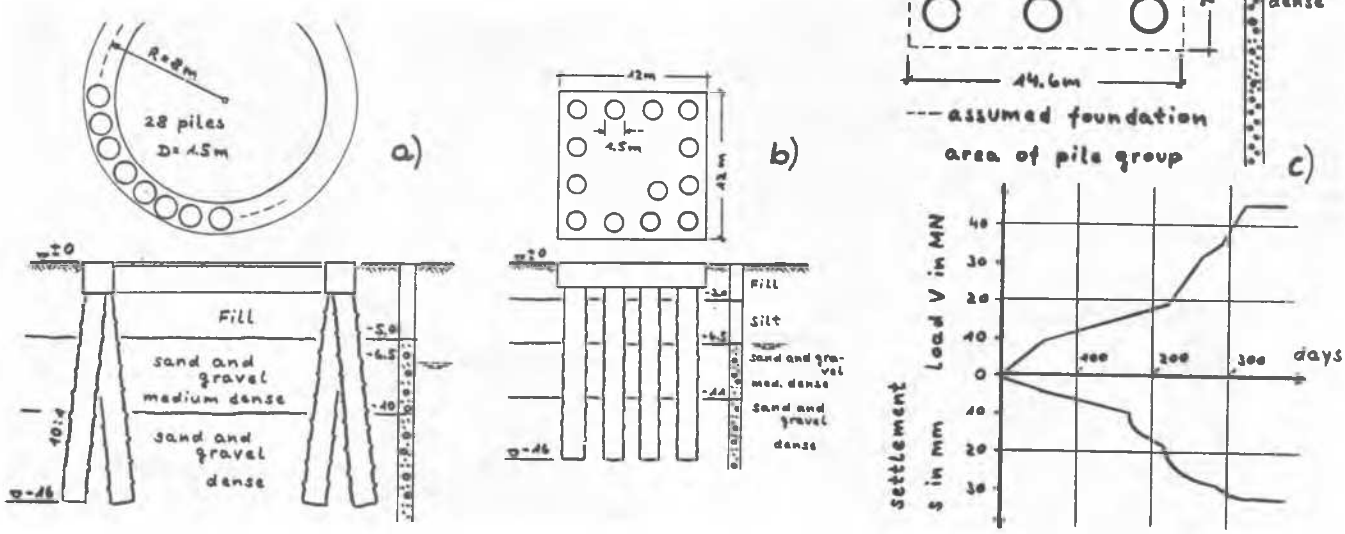


Fig. 1 Pile foundations for a) the chimney, b) the stair case tower of a thermal power plant at Mannheim; c) for the pier of a highway bridge at Köln; F.R.G.

M. Appendino (Written discussion)

Discussion of papers by Möller and Bergdahl on
DYNAMIC PORE PRESSURE DURING PILE DRIVING IN FINE SAND
and by Steenfelt at al. on
INSTRUMENTED MODEL PILES JACKED INTO CLAY

The paper from Steenfelt et al shows tests results from piles jacked into clay. It is interesting to note that a positive pore pressure peak develops shortly after the pile tip overpasses piezometers level. I noted in the field a similar effect with the piezometric probe when penetration was interrupted for dissipation tests. The pore pressure peak may be interpreted either as a time-lag of instrumantation (incomplete saturation or volume deformation of the probe) or as a soil property proving creep effect under undrained loading. Pore pressure increases under undrained load was well documented by Prevost

(1976), Richardson and Whitman (1963) in triaxial tests.

Creep effects may be important also in sand.Möller reports small pore pressure variations (negative--positive) at tips of model driven piles. Very similar behaviour was observed by Appendino (1973-1979) from piezometers installed at pile tips in the field. Back calculation performed assuming undrained failure conditions indicates that a larger pore pressure schuld be obtained if sand properties are taken from standard triaxial tests Appendino (1979). The difference may derive from partial drainage or incorrect soil failure modelling or instrumental peak damping. It may abso be the consequence of using static load tests do determine soil properties. Infact when sand is quickly loaded creep is absent and sand will result less contractive or become dilatant at the same density and confining pressures.

If the observed effect would be proved due to creep, then soil properties to analyse pile driving must be determined with quick testing techniques.

A.A. Bartolomei and T.B. Permyakova (Written discussion)

ABOUT EXCENTRICALLY LOADED FRICTION PILE CLUSTER WORK

Complex experimental investigations of prismatic friction pile clusters actual work are conducted in the Perm Polytechnical Institute. Eccentrically loaded pile cluster work studying, taking into account objective regularity discovery of friction forces distribution along pile cluster lateral surface and compression stresses distribution in eccentrically loaded pile clusters active zone, is one of the investigation directions.

The experimental investigations were conducted using small scale tensometer-pile clusters in laboratory with the control of the basic results followed in the field conditions using actual pile clusters.

The results of friction forces distribution along the pile cluster lateral surface are shown at Fig. 1. That pile cluster, consisting of 9 small scale piles, was loaded with eccentricity e = 0,25a and e = 0,5a (a is one half of the pile grating). Friction forces along two opposite sides of the pile cluster lateral surface are compared: on the side X = a which is the nearest to the point of load application (curves I) and on the sede X =-a which is opposite to the first one (curves II).

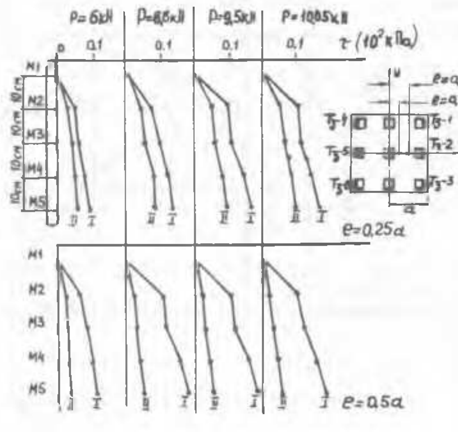


Fig. 1. Friction forces distribution along the pile cluster lateral surface under eccentrically applied loading:
I - tensometer-piles 1,2,3 detector indications
II - tensometer-piles 4,5,6 detector indications

Load eccentric application to the pile cluster causes friction forces redistribution along the pile cluster lateral surface, The load application eccentricity growth from 0 till 0,5a causes friction forces increase by 60-65% on the side X = a in comparison with the centric load application. The load application eccentricity growth on the opposite side X =-a from 0 till 0,25a leads to friction for-

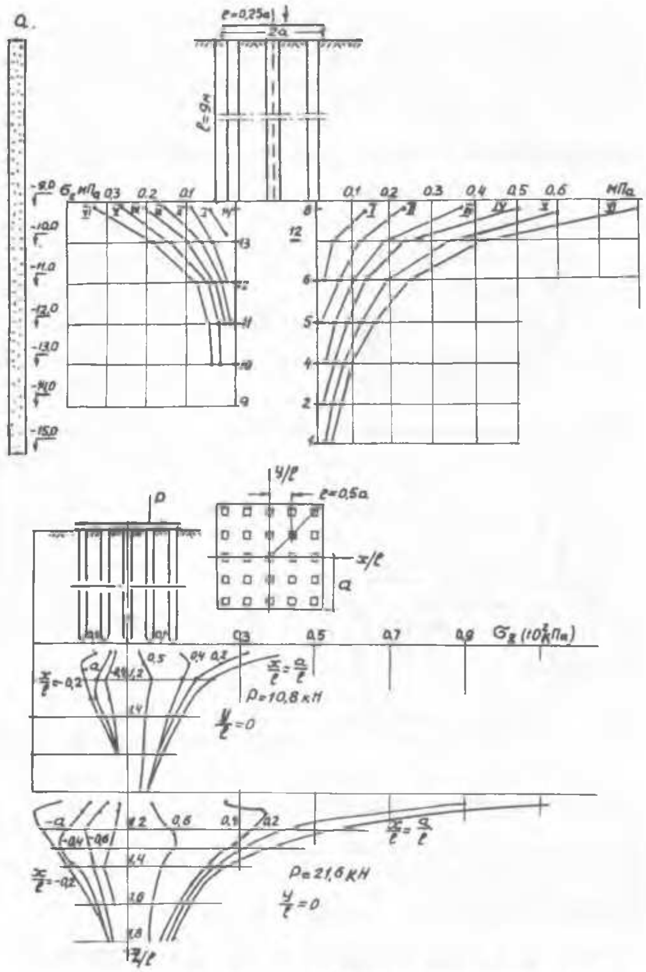


Fig. 2. The results of the actual 6 piles (a) pile cluster active zone stress distribution under loads I-VI - 1500, 2400, 3300, 5100, 6000 kN and small scale pile cluster (b)

ces decrease by 30-35% and then till 0,5a decrease by 65-70%.

Friction forces increase on the side X = a with the eccentricity growth can be explained by the pile inclination as th result of the additional ground repulse which appears due to the eccentric loading. The eccentric loading causes the ground breaking-off from the pile cluster surface on the opposite side X =-a with the appearance of micro-clearance followed which reaches its maximum value at pile top.

Epures of compression stresses at the actual pile cluster base with the eccentrically applied static load growth are shown at Fig. 2. That was pile cluster consisting of 6 piles. Compression stresses characteristic epures in active zone vertical sections of small scale pile cluster are shown at Fig. 2b (piles were placed on different distances from the pile cluster axis.

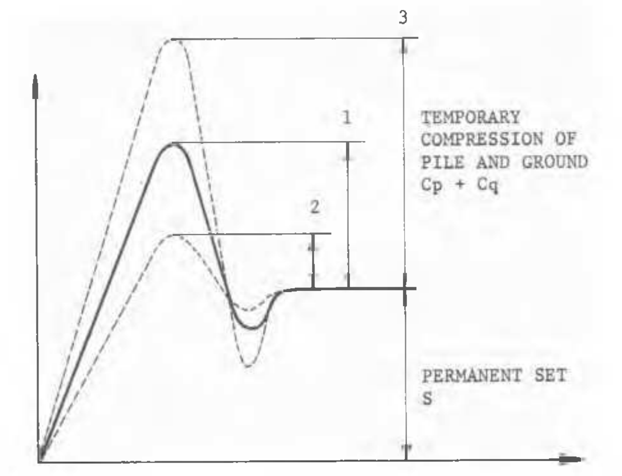
Investigations has shown that in the pile cluster active zone stresses consentration took place during the process of pile cluster eccentric loading in the direction similar the load application point moving. The eccentricity increase from 0 to 0,5a causes stress increase by 2-2,5 times at a level 1,5 m deeper than pile cluster base as compared with the centric loading.

As the result of the eccentric loading the pile cluster active zone depth increase in limits of 10-15% as compared with centric loading and for loads close to ultimate ones can be considered generally equal to the 1,7 - 1,9 length of the pile.

G.E. Bratchell (Written discussion)

CALCULATION OF THE BEARING CAPACITY OF DRIVEN PILES
Calcul de la Force Portante des Pieux Battus

On jobs where it is not intended to test-load driven piles, reliance is sometimes placed on dynamic formulae. Many of these formulae depend on the final set per blow. Indeed, in granular soils, there is a natural feeling that the pile is satisfactory when the driving gets hard and the set per blow becomes small.



A necessary presumption, though, is that the hammer is applying the necessary energy. In the case of Diesel hammers, manufacturer's figures should be used with caution. On one job with which the writer was involved it was discovered that the actual energy was less than half the expected energy and this was discovered only by comparison of the temporary compression and the permanent set for each blow.

DIAGRAM SHOWING THREE PILE DRIVING TRACES HAVING SIMILAR (DESIGN) PERMANENT SETS, S

CURVE 1: DESIGN CONDITION - HAMMER DELIVERING ITS EXPECTED ENERGY

CURVE 2: HAMMER DELIVERING LESS THAN ITS EXPECTED ENERGY (UNSAFE)

CURVE 3: HAMMER DELIVERING MORE THAN ITS EXPECTED ENERGY (SAFE)

G.E. Bratchell (Written discussion)

STRUCTURAL INTEGRITY OF IN-SITU PILES
L'Intégrité des Pieux Moules

It should be noted that test-loading with Kentledge of cast-in-situ piles may check the bearing capacity, but it does not ensure the structural integrity of the shaft. Defects such as missing concrete are a common experience in such piles. In many cases the reinforcement alone will carry the test load but may corrode later on, resulting in unexpected settlement. Some form of check on the structural integrity should be considered.

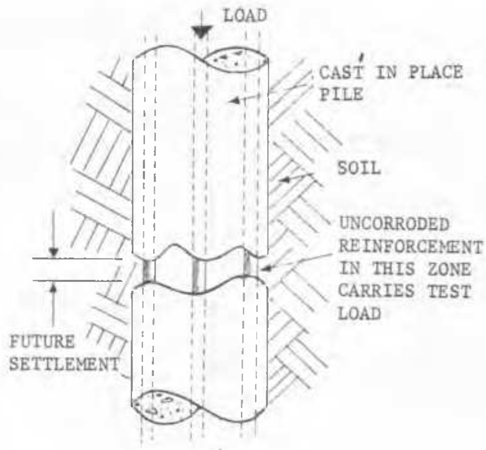
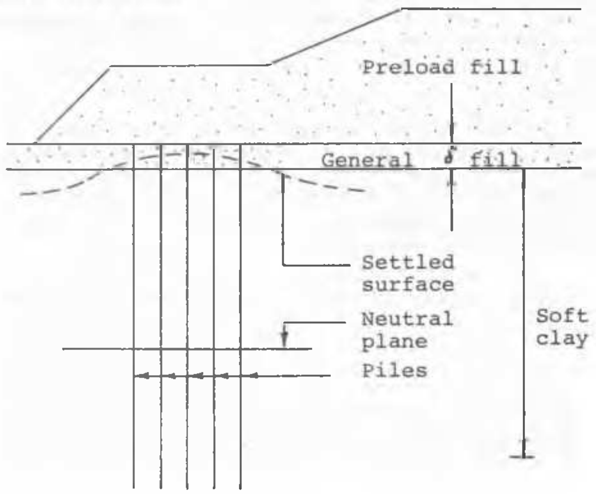


DIAGRAM SHOWING MECHANISM OF POSSIBLE SETTLEMENT OF PILE SUBSEQUENT TO LOAD TEST DUE TO CORROSION OF EXPOSED REINFORCEMENT

K.R. Datye (Written discussion)

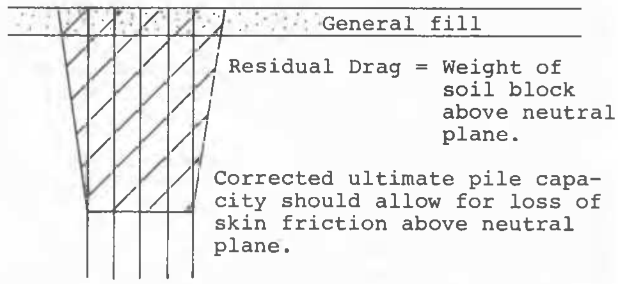
ESTIMATION OF DRAG FORCES IN PILES PLACED IN THE VICINITY OF PRELOAD FILLS

PRELOADING PHASE



This problem has not received adequate attention in published papers on negative skin friction phenomena. In practice, this situation often arises.

PRELOAD REMOVED



$$\text{Allowable pile load} = \frac{\text{Corr. ult. capacity - Residual drag}}{\text{Factor of safety}}$$

It would be evidently unreasonable to consider that the residual drag would equal the negative skin friction due to preload fill during preloading.

The writer would suggest the procedure explained in the following sketches for estimation of the residual drag forces induced in the pile after removal of preload.

A.G. Davis (Written discussion)

MEASUREMENT OF DEFORMATION PROPERTIES OF SOILS AROUND LATERALLY LOADED PILES

The general reporter to this session describes different techniques for the deformation properties of soils around laterally loaded piles. I wish to add a further technique to his list, developed by the C.E.B.T.P. Paris. This consists of an eccentric weight shaker attached to the head of a pile, and which operates under quasi-static conditions at frequencies between 1 Hz and 15 Hz, developing a maximum force of 20 KN.

The lateral velocity of the pile head is measured by a geophone fixed on the pile head, and the pile head response is analysed in terms of its mechanical admittance, as described in paper 8/20 to this session for vertically excited piles.

A typical test result is presented in figure 2, where both the mechanical admittance (velocity/Force) and the lateral pile head stiffness, E'_1 are plotted against excitation frequency.

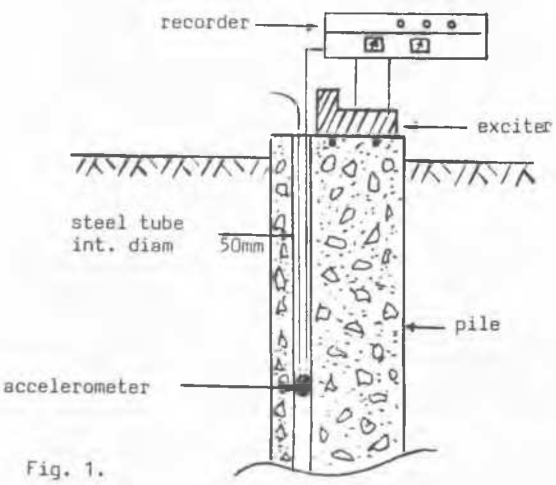


Fig. 1.

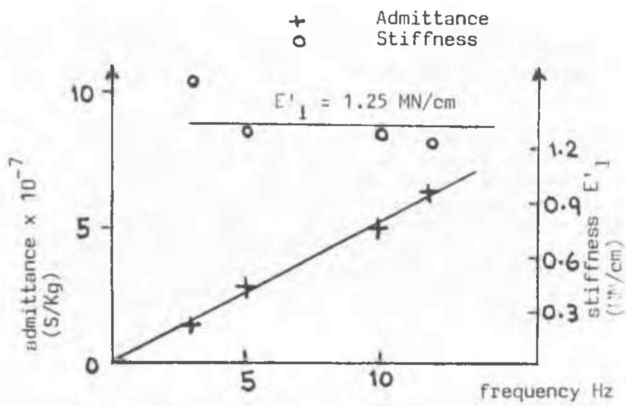


Fig. 2

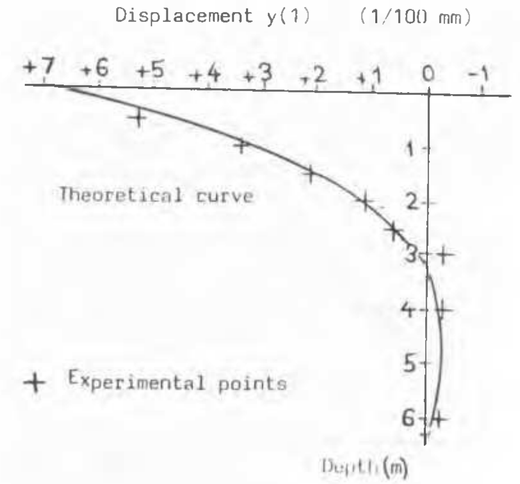


Fig. 3

The pile head stiffness has been correlated with static load tests on over 20 sites with different soils and different pile types. A one to one stiffness correlation ensues, and it has been observed that in nearly all cases the theoretical stiffness deduced from either in situ or laboratory soil tests were very different.

In certain cases it has been possible to measure the deformed pile shape during shaking by means of an accelerometer lowered in a vertical tube precast in the pile.

An example of the results from such a test are shown in figure 3. These tests have confirmed that the deformed pile shape is controlled principally by the upper two metres of soil. Also, the deformed shape and the neutral point position at these load levels fits linear elastic theory very well.

J. Feda (Written discussion)

DESIGN OF LARGE-DIAMETER BORED PILES

As mentioned in the Broms' general report, the weak link of the pile design at present is the evaluation of the strength and deformation properties of the soil and how these properties are affected by the installation method of the piles. One way how to cope with this difficult position is an extensive use of the results of pile loading tests. If they allow to evaluate the specific skin friction q_s and the specific pile point resistance q_o for a standard settlement of 1 cm, a statistical analysis of q_s and q_o /mean value and dispersion/ may be performed for different soil types. For the chosen reliability the results of such an analysis may be directly exploited for the pile design even for settlements greater than 1 cm.

An analysis of more than 200 loading tests of bored piles /different technology/ of diameter $d = 0.6$ to 1.5 m and different effective lengths D was undertaken. The following soil classes were incorporated: cohesionless soils /$I_D = 0.5$, 0.7, 1/, cohesive soils /I_C 0.5, 1/, rocks /A2, A3, A4 - the last one the most weathered/ and weak rocks /A5/. It was proved that a hyperbolic relationship of D/d and q_s /or q_o/ exists so that a linear regression as shown on Fig. 1 takes place. The analysis proved that for D/d > 3 large-diameter bored piles behave like friction piles. The correlation coefficient of the linear regressions was equal to 0.9, with the exception of stiff cohesive soils where it amounted to 0.7 only.

The coefficient of variability of 15% to 20% was found both for q_s and q_o. Further on, the ratio q_o/q_s was for individual soil classes constant /e.g. A2 - 122; $I_D = 0.5$, 0.7, 1 - 4, 4.4, 11.1; $I_C = 0.5$, 1 - 4.3, 10.5/- see e.g. Fig. 3. If compared with the CPT, approximately: for cohesionless soils

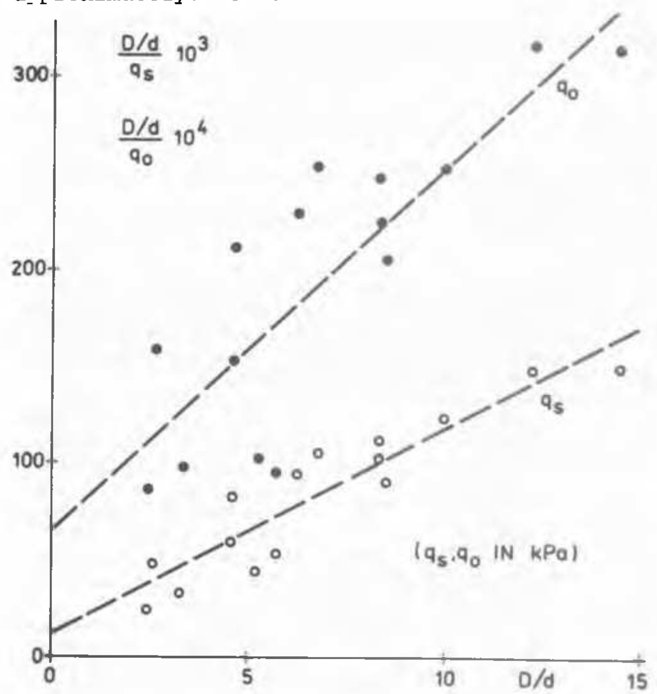


Fig. 1 Linear regression of q_s and q_o for dense cohesionless soils /$I_D = 0.7$/

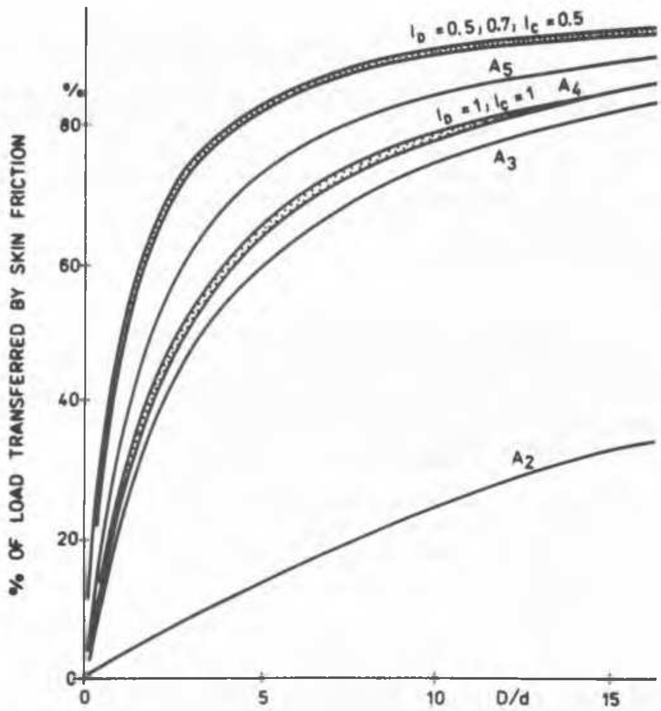


Fig. 2 The effect of the D/d ratio on the pile load transfer

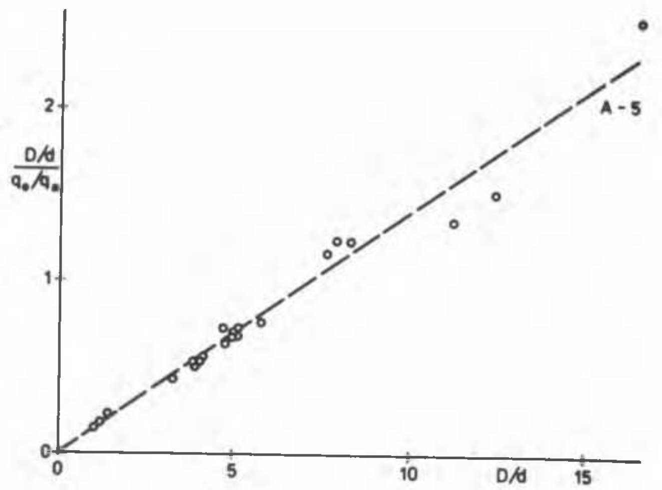


Fig. 3 The q_o/q_s ratio for weak rocks

$$/q_s/q_o/_{pile} \doteq 10\ /q_s/q_o/_{CPT}$$

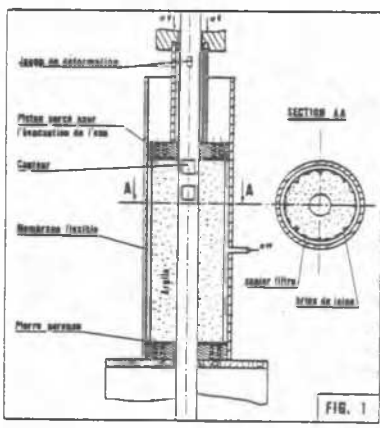
for cohesive soils:

$$/q_s/q_o/_{pile} \doteq 2\ /q_s/q_o/_{CPT}$$

J. Fournier, J.P. Mizikos and P. Ropers (Written disc.)

MESURE DES PARAMETRES NECESSAIRES AU CALCUL DU FROTTEMENT LATERAL D'UN PIEU
Parameters Measurements Required to Calculate the Axial Capacity of Piles

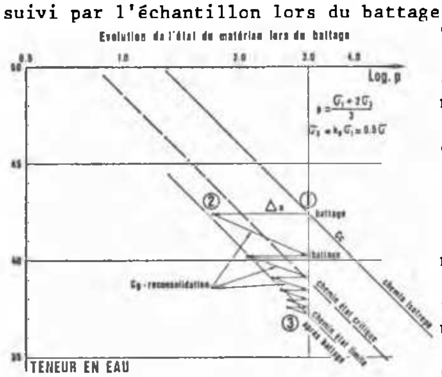
Pour une argile kaolinitique, nous avons mesuré l'évolution du forttement latéral au cours des différentes phases du battage d'un pieu (initial skin friction, skin friction after driving, skin friction during the reconsolidation and after a long time set-up) (fournier (1)).



SCHEMA DE L'APPAREIL

L'ébranlement provoqué par la percussion du marteau sur le pieu et l'évolution de pressions interstitielles sont mesurés par des jauges de déformation et des cellules de pression interstitielle fixées sur le pieu (Fig.1).
Les mesures du frottement latéral comparées aux mesures de pression interstitielle confirment que les variations du frottement latéral pendant le battage peuvent s'expliquer uniquement par l'évolution des contraintes effectives dans le cas des argiles normalement consolidées.

Pour prévoir l'évolution des pressions interstitielles nous avons mis au point un essai triaxial dans lequel des chocs sont donnés sur la tête de l'échantillon. Les variations de pression interstitielle mesurées sont comparables à celles mesurées dans l'essai de simulation du battage et permettent de suivre le chemin de contrainte suivi par l'échantillon lors du battage d'un pieu (Fig.2).



Trois résultats importants sont mis en évidence et permettent de compléter l'article d'esrig et kirby(2)
1 - La variation de contrainte effective moyenne est supérieure à la diminution de contrainte effective nécessaire pour amener le matériau à son état critique. Contrairement à esrig et kirby nous pensons que ce phénomène s'explique par une mauvaise détermination de l'état critique à partir des essais triaxiaux classiques. L'état limite atteint par battage serait alors déterminé plus précisemment soit par un essai triaxial avec chocs soit par un essai triaxial cyclique (one way cyclic loading).

2 - Lors de sa reconsolidation le matériau suit un chemin de pente C_g (indice de gonflement). L'état final du matériau dépend donc du nombre et de la durée des phases de repos pendant le battage (set-up).

3 - Après dissipation de la pression interstitielle le frottement latéral continue d'augmenter. Il peut atteindre dans l'essai de simulation 2 à 3 fois sa valeur initiale après un repos de 3 mois.

Enfin, les résultats obtenus sont utilisés pour vérifier la validité des formules usuelles utilisées pour calculer le frottement latéral. Dans le cas d'un frottement latéral appliqué sur la face extérieure du pieu nous montrons que le rapport $\frac{\tau \text{ mesuré}}{\tau \text{ calculé}}$ est comparable et voisin de α (coefficient d'adhérence) donné par caquot et kérisel pour les formules $\tau = \alpha C_u$, $\tau = \sigma'_N tg \phi'$ (coulomb) et $\tau = \frac{M}{2} P'_f C_{os} \phi'ss$ (esrig et kirby). Cependant les valeurs du rapport $\frac{\tau \text{ mesuré}}{\tau \text{ calculé}}$ sont nettement différentes des pratiques usuelles. (tableau 1).

τ mesuré en KPa	τ calculé			τ mesuré/τ calculé		
	esrig	coulomb	α Cu	esrig	coulomb	α
14	22,7	18	18	0,62	0,78	0,78
18,2	42,3	44,5	34	0,43	0,41	0,53
20	48,9	50,7	34	0,41	0,39	0,59
20,6	57,8	45	42	0,36	0,46	0,49

Tableau 1.

(1) - Fournier J. (1980) Le frottement latéral le long des pieux battus dans les argiles. Thèse de D.I. Ecole Centrale de Paris.

(2) - Esrig M.E. and Kirby R.C. (1979) Advances in general effective stress method for the prediction of axial capacity for drivin piles in clay. OTC 3406.

A. Holeyman (Written discussion)

Comments on: ALLOWABLE STRESS IN CONCRETE PILES DURING DRIVING

During the past 20 years, a large effort has been directed at the prevision of stresses in bodies to be driven into the ground. One major aimof this determination is to guarantee the integrity of the pile while driving, so that the computed or measured stress does not exceed an allowable stress, or a dynamic strength.
Though being equally important, this last characteristic has not yet received the attention it deserves. That is why this discussion gathers different approaches to grasp this concept.

The national regulations give some fixed values of the allowable stress of concrete (e.g. in Sweden, compression cannot exceed 20 MPa and traction 4 MPa) or the allowable height of drop (0.1, 0.3 or 0.6m depending on the circumstances).
This is rather rough treatment of the problem. In our opinion the dynamic strength of the concrete,relevant to the driving operations, should depend on several factors, and amongst them :

- quality of concrete
- % of reinforcement (longitudinal and stirrups)
- geometrical factors of reinforcement and concrete section
- quality of steel
- number of blows
- average duration of load.

Information about the allowable strain in concrete can be obtained during driving measurements of a pile where a break take place. Most of the time the spot where the damage occurs is not the place of measurement and it is necessary to :

1. locate the place of the damage
2. estimate the maximum stresses induced at this location by the previous blow,

which makes this determination rather unreliable. It also does not tell in some cases whether the damage is due to excessive compression or to excessive tension.

The opening of a crack at the location of the strain gauge will show immediately, and should be the best way to determine the dynamic strength of a particular pile. We were lucky enough to record such a case in which the previous blow gave a compression of 1750 str and an elongation of 112 str. (Fig.1, left).
The blow for which the extension of a crack was obvious gave corresponding values of 1400 str compression and far in excess of 2100 str elongation. (Fig.1, right). As the stiffness modulus of the pile was estimated to be 3.400MN/m, this figure led to a dynamic to traction strength of 0.38 MN, which compared about 2/3 of the elastic limit of the reinforcement.
As the modulus of elasticity of the concrete was estimated to be 40.000 MPa, it implied a tension on the order of 4.5 MPa and a compression on the order of 70 MPa.This reminds us that one has to be very careful when measuring and interpret-

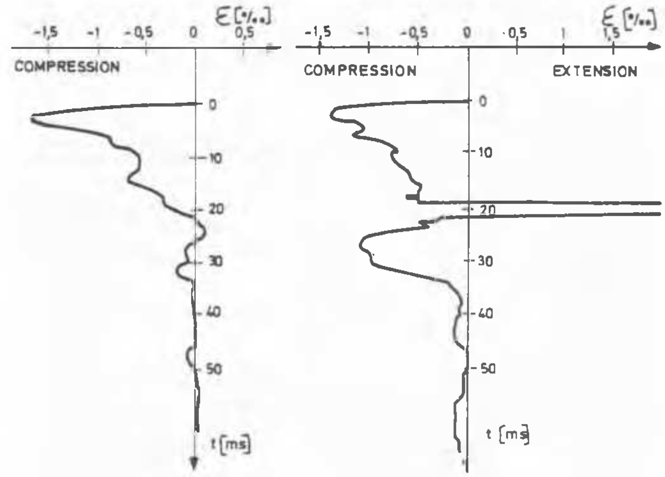


Fig.1 : Deformation under impact.
- left : before cracking
- right: after cracking.

ting the strains in a material which cracks - or which is already cracked.

As a matter of fact, most of the time the concrete is microfissured before the pile is ready for driving : shrinkage and handling forces have already induced microfissures in the concrete. These fissures, as they open, close, increase in number and size have an influence mainly on the number and size have an influence mainly on the non linear tension behaviour of the analized member, and raise some questions:

* what is the speed of the tension wave compared to the compression wave ?

* how can we account for a different behaviour in compression and tension in our wave equation analysis computer codes ?

* how does it affect the measurement of strain ?

Whereas it is clear that microfissures cannot be considered as detrimental to the function of the pile, one must limit the permanent width to which they may open (e.g. 0.2 mm) and establish the corresponding criteria for their detection by acoustic wave reflection.
This definition, which involves a more complex treatment of the behaviour of reinforced concrete in tension, is a perequisite to the objective of the stress determination, i.e. the integrity of the pile.

A. Holeyman (Written discussion)

DISK SPRING CAP
Comments to the discussion by T. Iwanowski

The disk spring cap is presented by Mr T. Iwanowski as an improved way of transmitting the energy of a hammer to a pile to be driven. Owing to its prestressed condition, this new cap develops upon impact a more rectangular pulse shape than a "conventional" helmet. The benefits reported from this process are:
1. Reduction of peak stress
2. typically a doubling of the penetration, supposedly for the same blow. As a way to convince ourselves of the usefulness of such a device, we have followed a straightforward analytical approach,namely the wave equation method as developped by Smith (1960).

The pile chosen for this analysis had the following features :
- prismatic reinforced precast concrete pile
- length : 24 m
- cross section : 0.08 m^2
- linear mass : 200 kg/m
- composite modulus : 42500 MN/m^2

The resistance of the soil can be described as follows :
- no skin friction
- resistance of the base : elastic-plastic with values of the rupture chosen at
  R = .0.04/0.40/1.20/2.40 and 4.00 MN
  Quake : Q = 2.54 mm
  Damping: J = 0.5 s/m

The hammer had a mass of 3,600 kg and the drop, with an efficiency of 0.8, was 0.3/0.6/1.2 and 1.8 m.
The conventional helmet we use has the following average properties, measured in the laboratory, after normal wear :
stiffness : k = 400 MN/m
heigth : h = 0.3 m
mass : M = 350 kg

The prestressed helmet we modelized in our computer code had the following features :
stiffness : k = 4000 MN/m until 0.8 MN;
from then on k = 55 MN/m
height : h = 0.8 m
mass : M = 750 kg

The coefficient of restitution was chosen for simplicity for both materials equal to 1, as its influence is nominal, as evidenced by Desai and Christian (1977). Figure 1 compares the loading curves of the two pieces of equipment considered. The parametric study which has been conducted shows that:

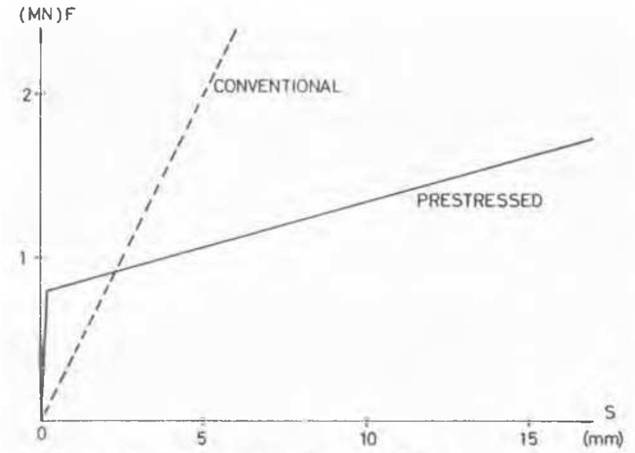


Fig.1: Loading curves of helmets

- the peak stress induced in the pile with the prestressed helmet is varying between 70% (drop = 1.8m) and 90% (drop = 0.3m) of the stress developped by the conventional helmet
- the set, obtained by the deduction of the quake from the maximum toe penetration, is smaller with the prestressed helmet for values of the resistance at the base of 2.4 and 4MN, is about equal for the base resistances of 1.2MN and is larger for base resistance equal to 0.4 and 0.04MN. (about 20%)

Fig.2 shows,as a function of time the force diagrams reaching the base with both pieces of equipment in the case of a drop of 1.2 and a base resistance of 1.2MN. One notices indeed that the shape of the pulse is more rectangular and peak lower with the prestressed helmet. But this does not affect the penetration significantly since the corresponding sets were found equal to 13mm fot the prestressed helmet versus 12mm for the conventional.
This discrepancy with the second benefit mentioned by Mr Iwanowski cannot be explained by the difference in coefficients of restitution since

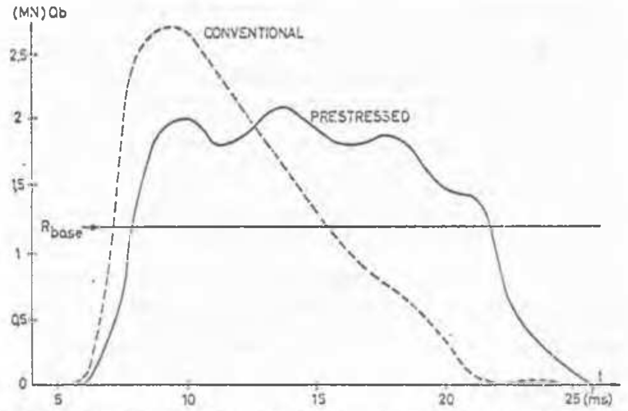


Fig.2: Force waves reaching the base of the pile

the sets are fairly large, nor by the lack of friction. As the world-wide accepted scheme of the equation should not be questioned for elastic bodies, one wonders about its adequancy in describing the behaviour of non linear elements as a prestressed helmet.

In our view, the doubling of the set has to be understood with respect to the allowable stress level in the pile. Indeed, the same peak stress is induced with a drop of 0.97m on a conventional helmet and with a drop of 1.8m on a prestressed helmet. Fot those two blows the respective sets are 9.5mm and 18.5mm, which is about double of the penetration, with the same safety as to the structural integrity of the pile.

References.

1. E.Smith (1960) :"Pile. Driving Analysis using the Wave Equation". ASCE. Journal of the Soils Mechanics and Foundation division, august 1960.
2. C.Desai and J.Christian : (1977). Numerical Methods in Geotechnical Engineering : Mc Graw Hill, New York.

T. Iwanowski and B. Larsson (Written discussion)

FIELD PILE ANALYSIS BASED ON STANDARD INSTRUMENTS

A modern application of the minicomputer significantly facilitates field analysis of pile behaviour and soil resistance during pile driving. Principles of the analysis, derived from the onedimensional wave propagation theory are widely accepted as a useful tool for modern pile construction control. The hitherto used systems, based on these principles, are bounded by limited performance of analog devices applied in the field or by nonportability of large computers. Developed by Piling Development in cooperation with Uppsala University the PiD Piling Analysis System avoided these disadvantages. Due to application of a digital storage oscilloscope with floppy disc memory, an easily portable digital minicomputer and digital plotter, the system can fully operate in the field in relatively rough environment, performing all operations in very short time. The system consists of: a pair of strain transducers and accelerometers, attached during the measurement to the pile sides close to the pile head; connection box equipped with calibration circuits (including shunt resistance calibration of the strain bridge); PiD Piling Pre-Analyzer (including preamplifier and integration circuits and digital display of maximum force and velocity values for each hammer blow); Nicolet digital storage oscilloscope with floppy disc memory connected by the interface to HP-85 minicomputer and digital plotter. The measured signals proportional to the force and particle velocity of the pile are stored in the oscilloscope memory during pile driving, and automatically sent to HP-85 minicomputer. The HP-85, programmed in Basic and Assembler performs all of computation and the following diagrams are automatically plotted within 2-4 minutes:

- force vs. time
- velocity/particle velocity/vs. time

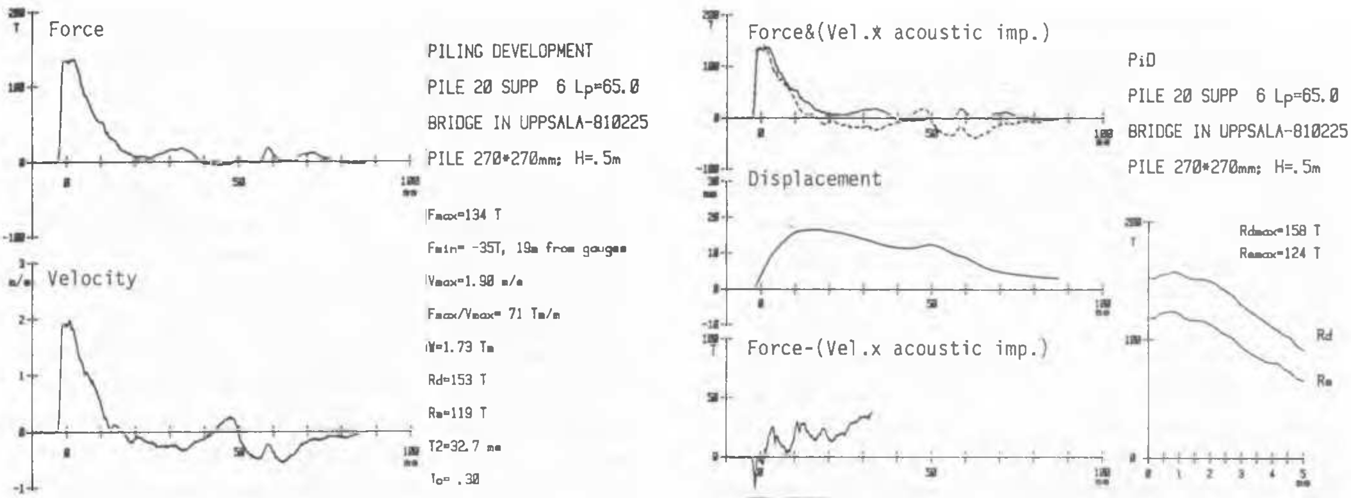


Fig. 1 Examples of pile documentation (page 1 and 2) obtained in field during measurements in Uppsala by means of PiD Piling Analysis System.

- force and (velocity x acoustic pile impedance) vs. time
- difference between force and (velocity x acoustic pile impedance) vs. time
- pile head displacement vs. time
- dynamic and static pile bearing capacities vs. time/in a case of a large elasticity under the pile tip causing underestimated values of pile capacity/.

Besides these curves, the following data are printed on the diagram chart:
- maximum force of the initial impact wave
- maximum tensile force and its location along the pile
- Maximum velocity/particle velocity/ of the initial impact wave
- actual acoustic pile impedance
- energy transferred to the pile
- dynamic and static pile bearing capacities
- time for wave reflection.

These diagrams and figures give reasonable data not only for prediction of total pile bearing capacity, but they can also be used for pile integrity check, show distribution of skin friction and influence of pile tip elasticity. There is additionally printed all information describing pile length, pile location, the date of measurement etc., so within 2-4 minutes the full pile documentation is ready. Also the floppy discs with recorded measured signals can be stored for possible further analysis. The HP-85, after connecting through telephone modem can also be used as terminal for communication with a host computer to perform more sophisticated wave equation analysis. Such results can be sent back and be available in the field in relatively short time.
The very important advantage of the PiD Piling Analysis System is its easiness of operation. The system doesn't require any computer or programming knowledge from the operator, and all operations are instructed through the minicomputer screen.
Thus, due to application of new computer technologies, engineers have obtained a new efficient tool for field control fo pile structures as well as of performance of piling equipment. It is to be noticed that this is obtained by using a standard digital oscilloscope and a standard minicomputer in combination with a specially designed Pre-Analyzer.

Y. Nishida (Written discussion)

Mr.Kilker el (Paper 7/20) presented an intrested information of the influence of effective stress on SPT-N Values. N-Values is dependent on meny factors. Then the writer would like to present the following relationship, from an analogous study to the expansion of semi-spherical cavity in soils around a pile base, for the coefficient of driving efficiency e_f, the angle of friction ϕ, Poissons ratio ν , Youngs modulus F, the overburden pressure p_o of soils:

$$N = \frac{30}{e_f 243 \dfrac{(3-\sin\phi)}{3p_o(1+\sin\phi)} \left\{\dfrac{E(3-\sin\phi)}{12p_o(1+\nu)}\right\}^{\frac{-2\sin\phi}{1+\sin\phi}} -0.25}$$

E is dependent on the value of p_o and it is found that $E \propto p_o$ or $E \propto \sqrt{p_o}$ by the experimental data. Then it may be concluded that

$$N \propto 1/(A + B\, p_o^{-\alpha})$$

where A and B are some constants and $\alpha = 1/3 \sim 1.0$ for practical soils. The above expression may give an estimation of influence of effective stress on N-Values quantitativly.

The writer would like to emphasises that the coefficient of lateral earth pressure against a pile shaft, K, is much dependent on the displaced soil volume due to the lateral thrust by the pile. By studying a cylindrical expansion of soil around a pile shaft the writer can present the following relationship between the K and the coefficient of earth pressure atrest.

$$K_o \leqq K \leqq K_o + \left\{\frac{1-\sin\phi}{2(\nu+\sin\phi)} - K_o\right\}\left\{1 - \frac{1+\sin\phi+2}{1+\nu}\left(\frac{R}{a}\right)^{\alpha}\right\}$$

where $\alpha \equiv 2(\nu+\sin\phi)/(1+\sin\phi+2\nu)$, ν is Poissons ratio and ϕ is the angle of friction of sands. a is the radius of a pile and R is the radius of compacted zone of soils by a pile depending on Youngs modulus of soils. In practice R/a has the value of 2 $\sim$ 6 for sands.

The above relationship will give that K is 1.0$\sim$1.5 for a driven pile while less value for a bored pile than K_o. It may be suggestive to reports by Mr.Klos el(8/34), by Mr.Lindqvist el (8/36), by Mr.Meyerhof el(8/39), by Mr.Mohan el (8/41), by Mr.Trenter el(8/59), by Mr.Velloso el(8/60) and by Mr.Balasubramaniam el(8/4).

When the pile formula (dynamic formula) is applied to predict the bearing capacity of a pile, it is necessary to assess the elastic deformation of soils per blow in advance. According to the writer's study it can be concluded that the elastic compression of soils under a pile base is(0.1$\sim$0.2) times pile radius for sands and (0.5$\sim$0.7) times pile radius for normally consolidated clays when the pile penetrates in to the ground by a hammer driving. This numerical data may be suggestive to reports by Mr.Moe el(8/40) and by Mr.Lindqvist(8/36).

Mr.Lindqvist applied Boussinesq equation to calculations of the displacement due to skin friction force. However the writer believes that Mindlin's equation is better to-do in this case as well known.

Mr.Steenfelt el(Paper 8/56) presented the very instructive data. Then the writer would like to ask the influence of the ratio between the pile diameter and the chamber diameter on the generated pore pressure, since the ratio seems to be 10 in this experiment. According to the writer's study on the excess pore pressure in group piles, Δ u in the failured zone may be:

$$\Delta u = c_u \left[\frac{1+}{3} \left\{ \frac{2(1+\ )}{1-\nu} \frac{R^2}{b^2} + 4 \log\left(\frac{R}{r}\right) \right\} + (A-\frac{1}{3}) \sqrt{3 + (1-2\nu)\left\{ \frac{(1+\nu)R^2}{(1-\nu)b^2} + 2 \log\left(\frac{R}{r}\right) \right\}^2} \right]$$

where c_u is the undrained shear strength, ν is Poissons ratio and A is the coefficient of pore pressure. R id the radius of failured zone and b is the radius of chamber in this case. r is the radial distance from the pile axis, ($r \leqq R \leqq b$). R is dependent on both b and Youngs modulus of clays and the above expression is limited within the zone $r \leqq R$. The Author's eq.(2) is effective only for $\nu \fallingdotseq 0.5$ and no vertical deformation (plane strain condition). If the vertical deformation is allowed

$$d\Delta u / d(\log r) = -(4/3)c_u$$

even if A is 1/3.

M. Popescu and V. Laber (Written discussion)

THE YIELD FORCE ON STABILIZING PILES IN A ROW

Piles were widely used in recent years as a landslide control work. The lateral force acting on piles used for stabilizing moving slopes is the smallest of the following two forces : (1) the earth thurst necessary to stop the slope movement, (2) the lateral soil-pile interaction yield force. The first force can be calculated by means of conventional slope stability equilibrium methods.

The problem of the yield pressure has been analysed by Ito and Matsui (1975) considering the interval between piles, in order to represent the complicated mechanism of the interaction between piles and plastically deforming soil wich is obliged to squeeze between the piles in a row (fig.1). De Beer and Carpentier (1977) made similar calculations based on modified assumption regarding the stress state arround piles. Comparing the observed lateral forces on the stabilizing piles against landslides with the values calculated according with the theory of the above mentioned authors, a good agreement of the order of magnitude was found.The values calculated by IM formula are always greater than the values calculated by DBC formula, the differences being the largest for the case c=0.

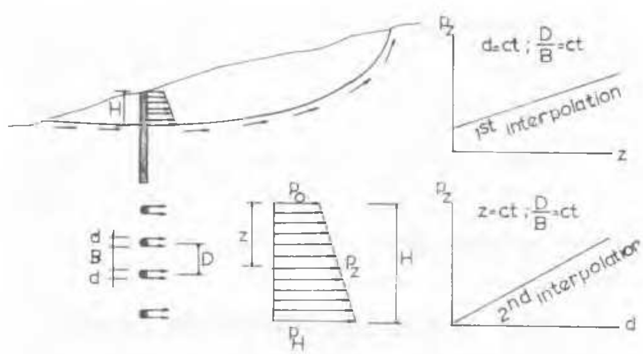


Fig.1

The theoretical equations of the lateral force p acting on pile per unit thickness of soil layer are rather complicated. Consequently it was considered appropiate to prepare a set of plots giving the variation of p with soil strength parameters c and tgØ, as presented in fig.2. These plots allow rapid determination of p_o, at z=0, and p_{10}, at z=10 m, z being measured from the slope surface. As the lateral force increase linearly as z increases, the values p_H are obtained by linear interpolation between p_o and p_{10}, when lateral deformation occurs in a soil layer of thickness H≠10 m.

The lateral force increases as the ratio B/D (clear interval between piles/center-to-center interval) decreases and it increases rapidly as B/D decreases still more, in the case of a constant diameter of pile, d. As the usual interval D= (3-5)d, a medium ratio B/D= 0.7 was considered in fig.2. The lateral force p increases as d increases and this relation is linear when B/D is constant. Consequently a second linear interpolation is necessary to obtain the p value for a pile diameter d≠1 m or 3 m, as represented in fig.1.

It must be emphasized that an overestimation of the lateral force acting the pile is a safe assumption for the pile stability, but an unsafe assumption for the overall slope stability, and vice-versa(fig.1). On this line it is recommended to use the IM values for pile stability analysis and DBC values for slope stability analysis.

Also it must be remembered that the placement of piles before the occurence of a slide is an economical measure. If the slide has occured, the soil is disturbed and its shearing resistance parameters are reduced because of remolding, and more piles are needed than when they are placed before the slide (De Beer, 1977).

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De Beer, E., Carpentier, R. (1977) : "Discussion on a paper by Ito and Matsui", Soils and Found., no. 1.

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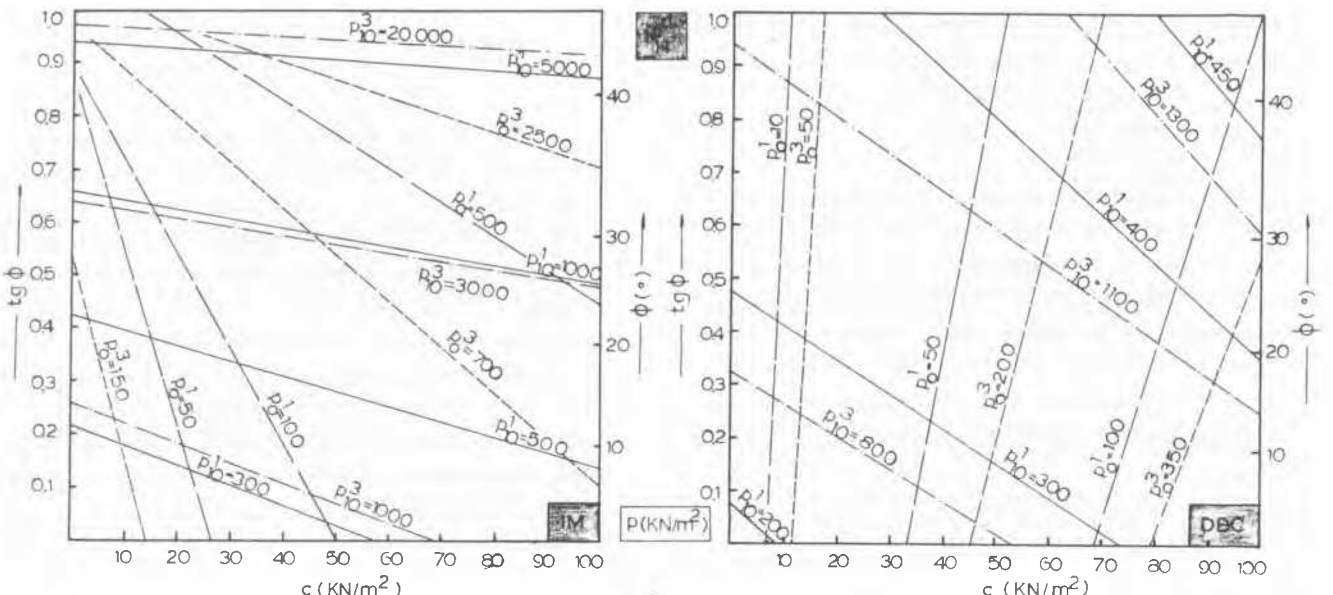

Fig.2

G. Ramaswamy and N. K. Jain (Written discussion)

On A SIMPLIFIED ANALYSIS OF PILES WITH LATERAL LOADS by C.S. Oteo and J. Valerio, Vol.2, p 795.

Authors have suggested the use of 'equivalent cantilever method' to determine the deflection at the pile head and the maximum moment for partially embedded piles. A similar approach had been suggested by Davisson and Robinson(1965). The present analysis (by the authors) is based on elastic behaviour of soil whereas the analysis by Davisson and Robinson(1965) is based on modulus of subgrade reaction theory.

The writers have obtained a rigorous solution for deflection, moment etc. based on modulus of subgrade reaction theory for partially embedded piles subjected to vertical and lateral loads. Based on the results of the analysis the writers have the following points to make.

1. It is true that the maximum moment in the pile is not the fixed end moment of the equivalent cantilever and needs to be corrected using a reduction factor(m) as suggested by the authors. The theoratical values of m presented by the authors, closely agree with those obtained by the writers. However, in the opinion of the writers, a correction to the theoratical values of m based on experimental values is not warranted as the moment measurements are through indirect means using mostly strain gauges and these are not generally so accurate as to treat them as actual values.

2. Authors have made no mention regarding the position of maximum moment. The position of maximum moment below the ground level is not equal to the depth of fixity. The position of maximum moment depends on the unsupported length of the pile. The results of the analysis by the writers is presented in Table 1.

3. The axial load increases the deflection of the pile head and the maximum moment significantly and need to be taken into account in the flexural analysis of partially embedded piles.

Table 1 - Position of Maximum Moment and Depth of Fixity.

Unsupported length $Z_u = L_u/R$	Depth of Fixity of equivalent cantilever $Z_f = L_f/R$		Position of Maximum moment below ground level $Z_m = L_m/R$	
	Piles in Clay	Piles in Sand	Piles in Clay	Piles in Sand
1.0	1.46	1.85	0.54	0.96
2.0	1.41	1.82	0.36	0.84
3.0	1.38	1.79	0.26	0.66
4.0	1.35	1.78	0.21	0.58

where

L_u = dimensional unsupported length

L_f = dimensional depth of fixity

L_m = distance below G.L.

R = relative stiffness factor

For clay - $R = \sqrt[4]{\dfrac{E_pI_p}{K_h}}$

K_h = modulus of horizontal subgrade reaction

For sand $R = \sqrt[5]{\dfrac{E_pI_p}{n_h}}$

n_h = constant of horizontal subgrade reaction

4. Results presented in Fig.4 by the authors are for free pile head condition. In pile groups the pile head is likely to be fixed to some degree or other. Fixity of pile head would significantly affect the pile deflection and moment. In view of this the results presented in Fig.4 have limited application.

REFERENCES

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M. Roy (Written disscussion)

The comments on the methods to use to calculate the shaft resistance of piles in clay have been made during the panel discussion and it was clear that the total stress approach using the undrained shear strength is still extensively used. I would like, in this discussion, to emphasize on the basis of experimental evidences that the effective stress controls the shaft bearing capacity during the driving, the consolidation and thereafter.

Full scale investigations with steel piles having an outside diameter of 22 cm were carried out on the St.Alban test site to study the behaviour of friction piles driven in soft sensitive clay of marine origin. The St.Alban clay belongs to Champlain clay deposits which have been formed during the northerly recession of the Wisconsin ice sheet between 12000 and 8000 years before present. The clay, which has been the objects of many investigations has been described in many papers (La Rochelle et al 1974 and Roy et al 1981) and we refer the reader for more details on its main characteristics.

In our program piles were driven and loaded to failure at different time intervals to investigate the change in bearing capacity with time. The pile was loaded to failure at time intervals of 14, 91, 189, 472, 790 hours and two years. Pile loads were applied in increments of 6,67 kN maintained for 15 minutes up to failure. During the test, the pore pressure response of the soil around the pile was monitored by Geonor piezometers installed at 31 cm from the pile axis and to the depth of 3, 4,6 and 6,1 meters.

The average unit skin friction τ_{fp} was computed at different depths during driving. These results (Roy et al, 1981) shown that the unit skin friction decreases rapidly as the pile penetrates. Between shallow depth and 7,6 m depth, τ_{fp} decreases from about 15 kPa to 2 kPa respectively.

The results obtained thereafter are presented in figure 1 as the ratio $\Delta u(t)/\Delta u(o)$ and $Q_s(t)/Q_s$ (t = 2 years) in function of time after driving. It is shown that the shaft bearing capacity increases proportionally to the dissipation of the induced pore pressures during the consolidation period. Fifty (50%) and eighty (80%) percent of the shaft bearing capacity are obtained after 60 and 200 hours of consolidation respectively, while full dissipation of induced pore pressure has been observed after a period of time larger than 700 hours. These results emphasize the fact that drained conditions should govern the bearing capacity of the friction pile in clay. Consequently, the bearing capacity measured in pile tests is controlled by effective stress and its evaluation should be done with effective parameters corresponding to the properties of soil surrounding immediately the pile. Then, greater emphasis should presently be put to evaluate these parameters.

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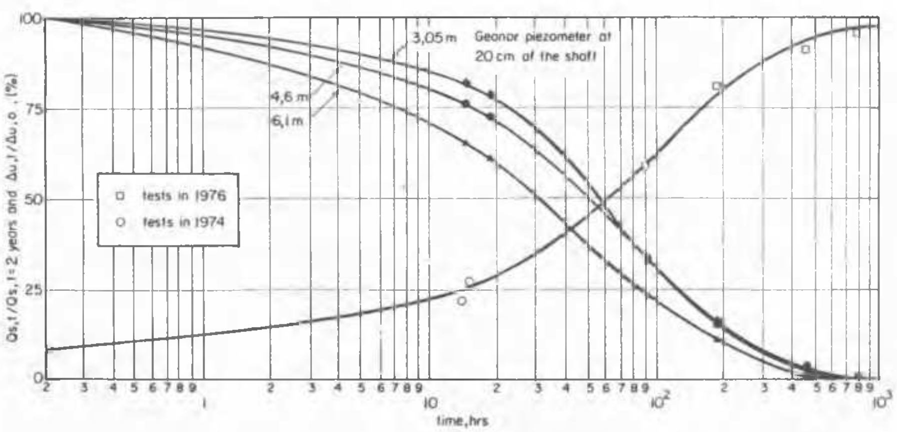

Fig. 1. Increase of the shaft bearing capacity during consolidation

J.G. Sieffert (Written discussion)

ENERGIE TRANSMISE AU SOL AU COURS D'UN VIBROFONCAGE

Dans le cadre du fonçage dynamique des pieux et des palplanches, nous nous sommes intéressés plus particulièrement au vibrofonçage. L'originalité de la méthode développée en laboratoire provient de la gamme de fréquence d'excitation utilisée. Un excitateur électromagnétique de notre conception permet d'appliquer en tête de pieu ou de palplanche une force sinusoïdale dont la fréquence peut varier entre 1 500 et 3 000 Hz, alors que les systèmes classiques sont limités à quelques dizaines de Hertz. L'utilisation de fréquences élevées se traduit par des déformations du pieu - et par conséquent par des déplacements alternatifs du sol en contact avec le pieu - de très faibles amplitudes. On conçoit aisément l'intérêt de la méthode, en particulier pour les travaux réalisés au voisinage immédiat d'ouvrages existants.

Une première série d'essais réalisés en laboratoire dans du sable sec avec un pieu de 3 cm de diamètre et de 3 m de longueur a permis de vérifier qu'il est possible d'obtenir un fonçage efficace à ces fréquences, à condition toutefois d'utiliser une des fréquences de résonance du système excitateur-pieu.

Nous avons ensuite développé une théorie en supposant que le frottement latéral sol-pieu ou sol-palplanche est du type sec. Parallèlement, de nouveaux essais ont été réalisés avec une instrumentation plus complète permettant le calcul des différentes puissances et énergies mises en jeu. Nous nous proposons de présenter ici quelques résultats obtenus avec une palplanche.

DISPOSITIF EXPERIMENTAL

Excitateur :	masse : 8,5 kg	raideur : 630 MN/m
Palplanche :	masse : 2,3 kg	longueur : 2,00 m
	largeur : 15,0 cm	épaisseur : 1 mm
Sol :	sable sec	densité : 1,6

Fréquence d'excitation : 2 678 Hz.

La mesure de la force appliquée par l'excitateur à la palplanche, de l'accélération de la section supérieure de la palplanche et de l'enfoncement en fonction du temps permet de déterminer l'énergie E_f fournie par l'excitateur, l'énergie E_i dissipée par amortissement interne dans la palplanche et par différence, l'énergie E_s transmise au sol.

RESULTATS

La figure 1 présente l'évolution de l'énergie (E_f) fournie par l'excitateur et de l'énergie (E_s) transmise au sol en fonction de la fiche. Les courbes représentatives sont bien évidemment croissantes et font apparaître une asymptote verticale correspondant au refus. Ce résultat est logique puisqu'au refus l'énergie sert uniquement à entretenir les oscillations dans la palplanche.

La figure 2 reprend les mêmes résultats en faisant apparaître plus clairement la répartition entre l'énergie transmise au sol et l'énergie dissipée par amortissement interne de la palplanche. On remarquera que la part de l'énergie transmise au sol augmente avec la fiche pour atteindre environ 75 % de l'énergie fournie au voisinage du refus.

CONCLUSION

L'intérêt de cette étude est d'avoir pu aboutir à l'estimation de la répartition des différentes énergies mises en jeu. Un essai d'enfoncement statique a été réalisé à l'aide d'un vérin avec la même palplanche et dans le même sol. La figure 1 montre que le bilan énergétique est favorable au vibrofonçage, sauf au voisinage du refus. Il s'agit là d'un argument important en faveur du vibrofonçage à fréquence élevée. Une étude est actuellement en cours pour effectuer une comparaison analogue dans le cas du battage.

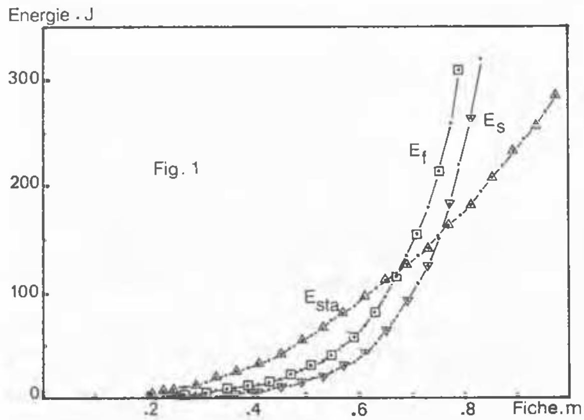

Fig. 1

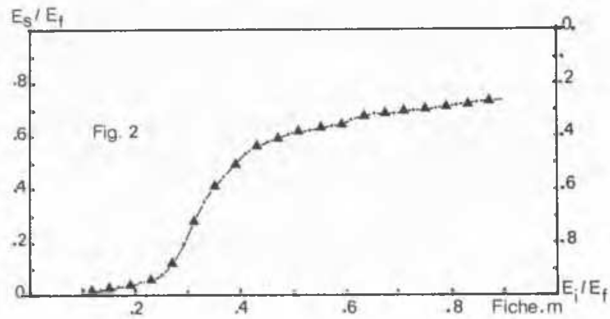

Fig. 2

M. Wallays (Written discussion)

ULTIMATE BEARING CAPACITY OF AXIALLY LOADED DRIVEN AND BORED PILES - CONE PENETRATION TESTS (CPT)

The cone resistances given by the cone penetration test can be considered as the ultimate bearing capacities given by successive loading tests on the base of a low diameter pile. Because of the diameter difference, the pile with a larger diameter must be driven deeper than the penetrometer, in order to afford the same base resistance in a soil with uniform characteristics. This can be observed experimentally (e.g. fig 2 to 4), and is due to the fact that the base resistance depends, for a main part, on the deve-

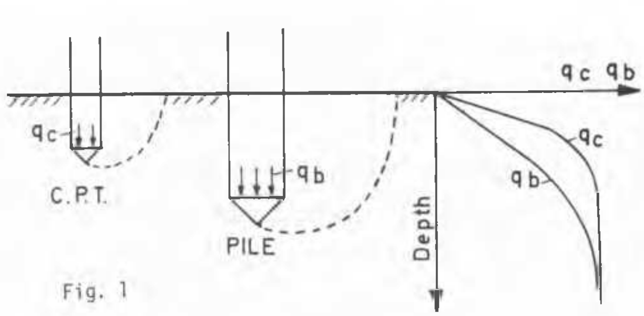

Fig. 1

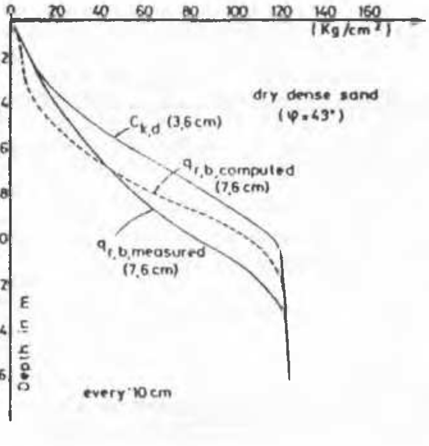

Fig. 2

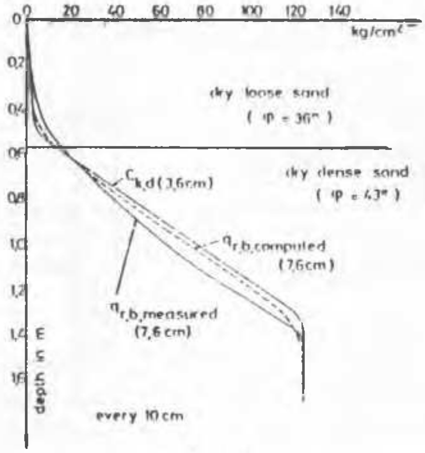

Fig. 3

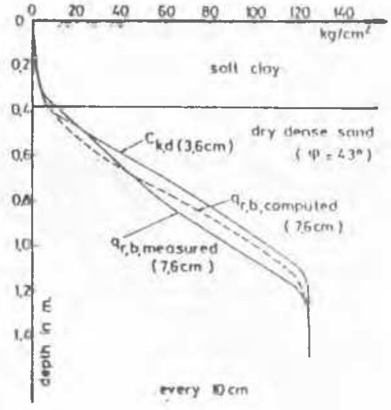

Fig. 4

lopment in the competent layer of the slip surface compared to the pile diameter (fig.1). Hence, when it is wanted to assess the diagram of the base resistance versus diameter of a large penetrometer with the same diameter as the pile, the diameter ratio has to be taken into account. Different authors, among them Begeman, De Beer, and Meyerhof have proposed methods for assessing this scale effect. In his 1971-1972 paper, De Beer produces assessed penetration diagrams from series of C.P.T. tests available at this time and performed at locations, where tests were afterwards carried out with larger penetrometers or piles, and he compared them with the experimental data obtained with the larger diameter piles. The results of the tests, carried out among others by Kerisel and his collaborators, Plantema, Pieux Franki and on different types of piles, were used in De Beer's work, in order to check its validity.

In 1977, at the Tokyo Conference, Meyerhof published, among others, diagrams versus depth of the point resistance measured, in a layered soil, with a 36 mm penetrometer and an instrumented steel pile 76 mm diameter, the upper stratum being formed by loose sand or soft clay and the lower stratum by dense sand. These comparative tests were also carried out in an homogeneous dense sand. Meyerhof's paper is devoted to the justification of the method he proposes in case of layered soils for assessment of the scale effect. Here, the experimental data published by Meyerhof are used, in order to check once more the validity of De Beer's method. In fig 2, 3 and 4 corresponding to the three considered cases, the full lines give versus depth the measured cone and base resistances and the dotted line the forecast latter. As a whole, the correlation between the calculated and the measured base resistances is satisfying . Nevertheless, some slight divergences arise in the critical thickness of the dense layer : in the upper part of this thickness, the forecast values of the base resistance are somewhat too low, and in the lower part, they are somewhat too large. This conclusion is rather unusual , because De Beer's method is arranged in order that, in the majority of the cases, the assessed base resistance does not be larger than the experimental one.

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Meyerhof G G and Valsangkar A J, (1977), Bearing capacity of piles in layered soils. Proc 9th ICSMFE,(1), 645-650, Tokyo.

M. Wallays (Written discussion)

HEAVE OF DRIVEN CAST-IN-PLACE PILES

The way to eliminate the lifting of the shaft from the bulb is the incorporation of the reinforcement into it, as it is usually realize for tensile Franki piles : a window of 2 - 3 diameters without stirrup is foreseen at the bottom of the reinforcing cage. This window is made indistortable at the upper and lower side This can be obtained with strong transverse reinforcements e.g. 3 or more adjacent stirrups (fig.1).
The reinforcing cage is placed from the beginning of the bulb construction or when a part of the bulb is already carried out. By hammering new quantities of zero slump concrete, the bars of the window are deformed, so that

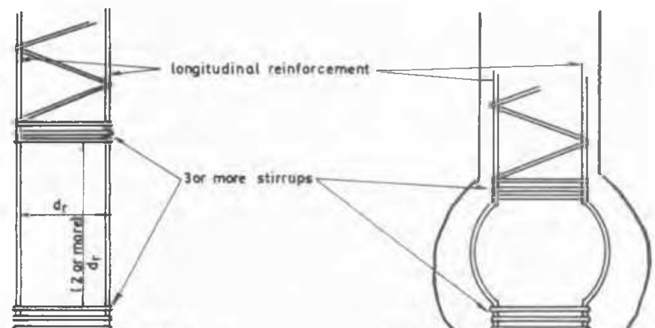


FIG.1 FIG.2

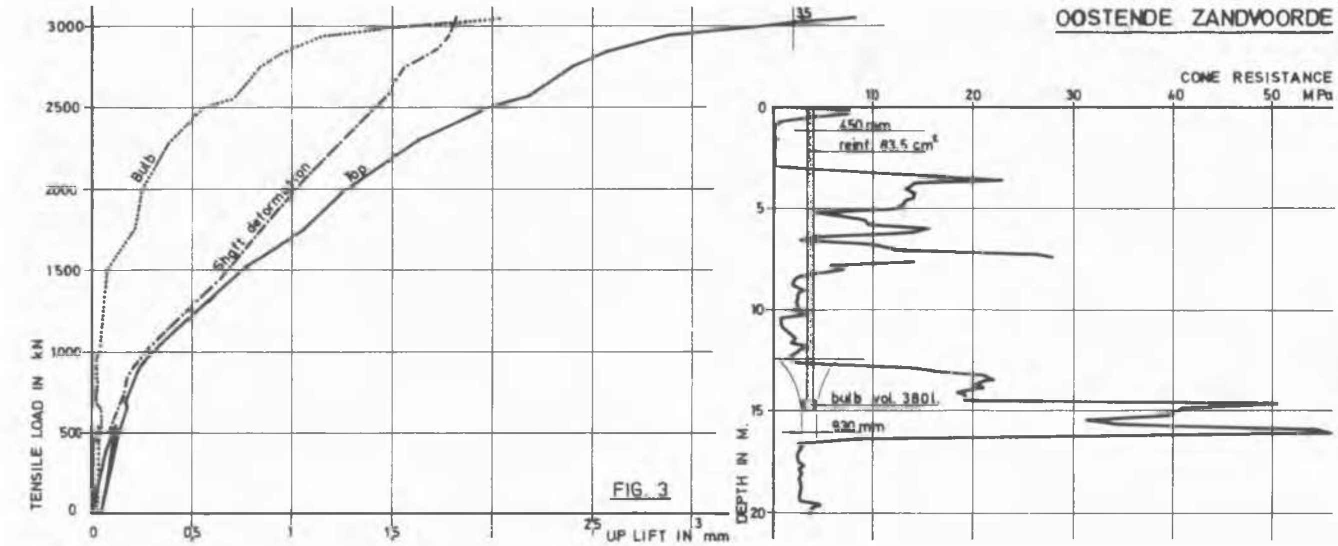


FIG. 3

the bulb is reinforced with curved bars (fig.2). Tensile loading tests have shown that such reinforced Franki bulbs can resist tensile forces larger than 250-300 tonnes (fig.3).

When the shaft is reinforced on its whole length and the reinforcement is incorporated into the bulb, the risk of separating or rupturing the shaft of previous driven piles is eliminated.

When,for the shaft,10 - 15 cm slump concrete is used instead of zero slump concrete,driven cast-in-situ piles being within 24 hours not installed closer than 12 shaft diameter and 10 bulb diameter from adjacent piles is usually recommended.This is for plastic shaft concrete a more severe recommendation than the minimum 9 shaft spacing proposed by Clark for shaft dry concrete. Nevertheless, when the longitudinal reinforcement equals at least 1 percent of the area of the shaft section and is incorporated into the bulb, the minimum spacing is accepted within 24 hours to be reduced to 6 shaft diameter and 5 bulb diameter. The introduction of the 5 bulb diameter into the recommendation makes that for relatively heavily reinforced piles, whose shafts are constructed with plastic concrete,this recommendation is approaching Clark's one for piles whose shaft is constructed with zero slump concrete.
An other way to eliminate the risk of rupturing the shaft and of lifting the base from the shaft is to drive first a series of driving tubes and to concrete afterwards the piles. This solution is mainly suitable for small groups of piles.

The described precautions, to be taken in order to eliminate the risk of rupturing the shaft, concern all kind of driven cast-in-situ piles, with ot without bulb. One of the advantages of the bulb, when it is bonded to the shaft, is the elimination of the danger that the piles are lifted some millimetres or centimetres out of the soil.

Reference :

Clark J,Harris M & Townsend D,(1981)
Heave of compacted expanded base concrete piles. Proc 10th ICSMFE,(2),667-672,Stockholm.

R.L. Wei (Written discussion)

EFFECT OF SURCHARGE ON PILE FOUNDATION

Due to some non-technical reason, a lock had to be constructed on an inadequate foundation. One end of the lock was founded on base rock, while the other end was underlain by a thick soft silty clay stratum (Fig.2). As soon as the lock had been constructed, the excavated soil was backfilled up to the original ground surface and then a 10m high embankment had to be constructed adjacent to the side pier (pier #6).
It was considered that the lateral deformation of the soft clay induced by the embankment load might cause deflection, crack even break of the cast-in-situ concrete piles under side pier. Nevertheless, owing to the rather good permeability of the silty clay and the interbedded fine sand, it seemed reasonable to take the advantage of consolidation of silty clay to reduce its lateral deformation. Hence construction by stages was suggested. In the first stage, an embankment of lowered and reduced section would be constructed, and the construction rate should be controlled according to the field observations. In order to monitor the deflection of pile as one of the means of controlling, a pair of inclinometers was buried in one of the piles under the side pier at depth of 2.8m below the top of pile. When the height of the embankment had been raised to about 6m, it was observed that the lock settled unevenly, and cracks appeared in some structural members of the lock. The variation of the inclinometer readings was very sensitive to the loading rate (Fig.I), for instance, as the loading rate increased, the variation of the inclinometer readings might be speeded up to 2½ times as much as before.
In view of the development of pile deflection

and taking account of the cracks appeared in the lock, it was decided to slow down the construction rate. The placement of soil within a distance of 12.5m from the side pier was ceased, and beyond this range the construction of embankment was continued but with further reduced section. Since then the variation of the inclinometer readings slowed down remarkably.

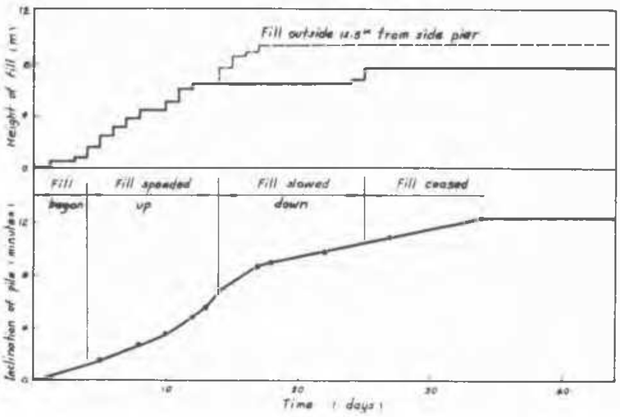


Fig.1 Development of deflection of pile with loading rate

The deflection of pile was induced due partly to the lateral thrust of the deformed soil mass and partly to the differential settlement of pier. According to some preliminary prediction there was a negative moment acting on the top of pile, so it is concluded that the effect of the lateral thrust was predominant. Also it

Table 1 Development of deflection of pile and inclination of pier

Date	18/5	24/5	1/6	30/6
Deflection of pile at depth of 2.8m (rad.)	0.0008	0.0019	0.0029	0.0037
Inclination of pier induced by differential settlement (rad.)	0.0005	0.0009	0.0017	0.0025

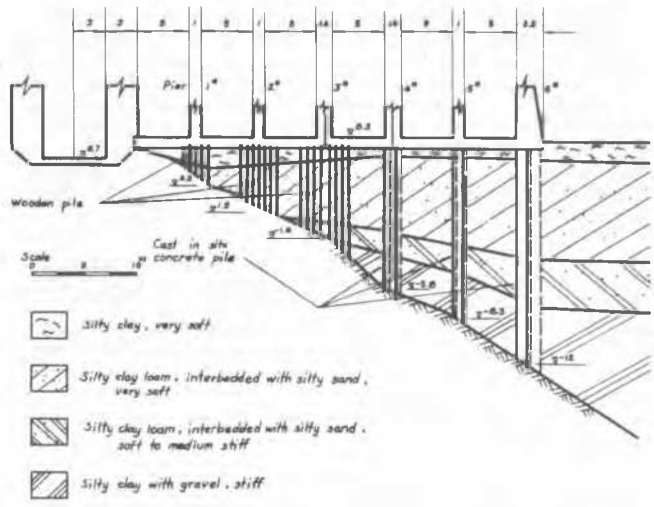


Fig. 2 Typical cross-section of pile foundation of the lock

was found to to our astonishment that long before the deflection of pile developed to 0.0008 rad., the plain concrete of the lower part of cast-in-situ pile might have been broken. Therefore it was decided to postpone the construction of embankment until next year. One year later on the embankment had been accomplished with normal construction rate without serious problem other than some further development of differential settlement and cracks in the lock. In summary it must be emphasized that the loading rate must be strictly controlled in order to minimize the undesirable effect when the ground near the pile foundation is surcharged in large area, and the loading rate should be kept even much lower than that adopted to guarantee the stability of the foundation. This could be seen from the fact that the ground surface near the embankment toe did not heave at all and the maximum value of coefficient of pore pressure $\bar{B}$ ($=\Delta u/\gamma h$) in the ground was also less than 0.25 as the lower part of the pile was predicted to have been broken.

G.G. Meyerhof, Co-Chairman

CONCLUSION OF ORAL DISCUSSION

This afternoon, we have listened to some interesting discussions on the bearing capacity of single piles and the behaviour of pile groups under vertical and horizontal loading. I should now like to make a few summary remarks which also include some observations on what you heard this morning from our General Reporter.

Dealing first with friction piles in clay, we have two methods of estimating the ultimate shaft resistance, which may be typified as the alpha (total stress) and the beta (effective stress) methods. We know that the more rational method of these two is the effective stress method using a beta value. Some of the scatter which has been observed in the beta values is due to two factors both of which I have discussed in my Terzaghi Lecture (ASCE, 1976). First, the beta value decreases with the length of piles due to progressive soil failure. For short piles it is typically roughly constant up to a depth of about 20 metres. However, for long piles exceeding about 100 metres, as for offshore structures, we can use only approximately one-half of this value, because of the decrease of the beta value with the length of the pile. This decrease can be explained by progressive failure of the clay along the pile shaft due to the change from the peak to the residual effective friction angle of the clay.

Secondly, the beta values increase with the over-consolidation ratio of the clay, roughly as the square root of this ratio. But it also increases with the pile taper and it depends, of course, also on the method of installing the piles, which is extremely important in fixing a relevant beta value for a given situation. If the beta value depends on the length of pile, overconsolidation ratio of the clay and other factors, as has been well documented, it is obvious that the alpha value in the total stress method also depends on all these factors, because the pile does not know what theory we have been using in estimating its ultimate shaft resistance! The decrease with length of pile had not previously been considered in the alpha method, but it should be included as one of the variants. The alpha value also increases with over-consolidation ratio of the clay and pile taper, just as the beta value.

For this reason the comparison made by some discussors of the total and effective stress methods in stiff clays was interesting and these kind of comparisons are necessary in different types of clay and in different parts of the world. Such comparisons show that we cannot use a given method of analysis or given soil parameters or constants if you like to call them, and transfer the results from one region to another without checking the method in a given case.

The separation of point and shaft resistances of piles, which was brought up by another discussor is important in this connection, because in clay the point resistance is generally based on the undrained shear strength, while the skin friction should be preferably based on the effective stress or beta method or, if experience is lacking, on the total stress or alpha method.

So far as estimates of the lateral resistance of pile, load tests are probably the most useful, since it is difficult to estimate from soil tests the horizontal modulus, which is required. Nowadays we prefer to use the stiffness modulus of the soil rather than the modulus of pile or soil reaction, and corresponding theoretical solutions of the lateral resistance and deflections are available for homogeneous non-homogeneous soils. If the soil is non-uniform or anisotropic, it is obvious that the horizontal soil modulus governs the lateral pile resistance, which in normally-consolidated soils is much smaller than the vertical modulus, while the opposite holds in overconsolidated soils.

I should now like to make some brief remarks on the ultimate bearing capacity of piles in sand. Here we should remember that the early correlations were made for driven piles. Thus, taking the point resistance of piles in sand equal to the cone resistance obtained from static cone penetration tests will only apply to driven piles and moreover only to piles of relatively small diameter, especially in dense sands due to the scale effects.

When we are dealing with piles of large diameter, however, in sand of a given cone resistance the point resistance of the pile decreases with greater base diameter, as shown by many recent investigations. They have shown for instance, that in dense sands the point resistance of a driven or jacked pile of large diameter, say greater than 2 metres, is only about one-third of that of a small pile of conventional dimensions, say less than 50 centimetres. In loose sand there is no such scale effect, as was shown in one of the papers submitted to this Session. Further for bored piles in dense sand we have a similar scale effect, and this has been well investigated on the Continent. However, the point resistance of a bored pile is much smaller than that of a similar driven pile and we can use only about one-third of the static cone resistance, as was also mentioned by some of the discussors.

On the other hand, the skin friction obtained from skin friction measurements on piles in sand is independent of the pile diameter, but again for bored piles it is typically only one-half of that of similar driven piles. Therefore, an allowance must be made for the method of pile installation, which has always been emphasized by Terzaghi, as well as for scale effects, especially in dense sands.

The same limitations mentioned above apply also to the use of standard penetration test N values in sand. For point resistance the original value which I gave in 1956 of 4N (kg/sq. cm) or 0.4N (if you like the Mega Pascals of today) applies only to driven piles of conventional size and if they penetrate at least 10 pile diameters into the sand bearing stratum. For large pile diameters and at shorter penetration depths smaller values of bearing capacity have to be taken, and similarly for bored piles we can only allow one-third of corresponding driven pile capacities. In other words, whether we use the standard penetration test or the static cone penetration test (since the pile does not know what methods we are using) similar reductions apply in both cases. The difference between the ultimate capacities of large and small diameter piles which was mentioned by one of the discussors this afternoon, in dense sand, is due to the scale effect which is a function of the compressibility, crushing, arching of the sand and other factors.

Finally, one of the discussors gave some interesting information on the partial safety factors of point resistance and skin friction of piles in sands and clays, and when we assign safety factors for use in practice such investigations are of great interest.

In concluding, I should like to thank all the speakers for their interesting contributions to the discussion of this Session.

J.Trofimenkov, Chairman

CLOSING OF SESSION

Now we have come to the end of this Session 8 on pile foundations.

I think that it was a very fruitful discussion today. We are all sorry that we could not hear all the speakers, but we have had a large number of interesting contributions, and I shall not even try to summarize them.

From the papers presented to this Session from the General Report and discussions that we have had it is clear what a great progress has been made since the last International Conference in our understanding of the interaction between pile and soil.

Our today's discussions have given us a lot of information which may be used in our practical work. At the same time we understand that many ideas and test results will serve for profession as a basis in future theoretical and experimental work. And these are very important and far reaching results of our Session.

There is a practical evidence that pile foundations are effective not only in sandy soils but in clay soils as well as even under very large loads. Under these circumstances, effective stress methods for prediction of axial shaft capacity for driven piles in clay have become very important. These methods require consideration of the state of stress and soil properties prior to pile installation, the changes in stress and soil properties as a result of pile driving, subsequent reconsolidation of the soil after pile driving, and finally, pile loading to failure. The present state of soil mechanics gives sound background for such an approach. The assessment of this approach by laboratory model tests we have heard today.

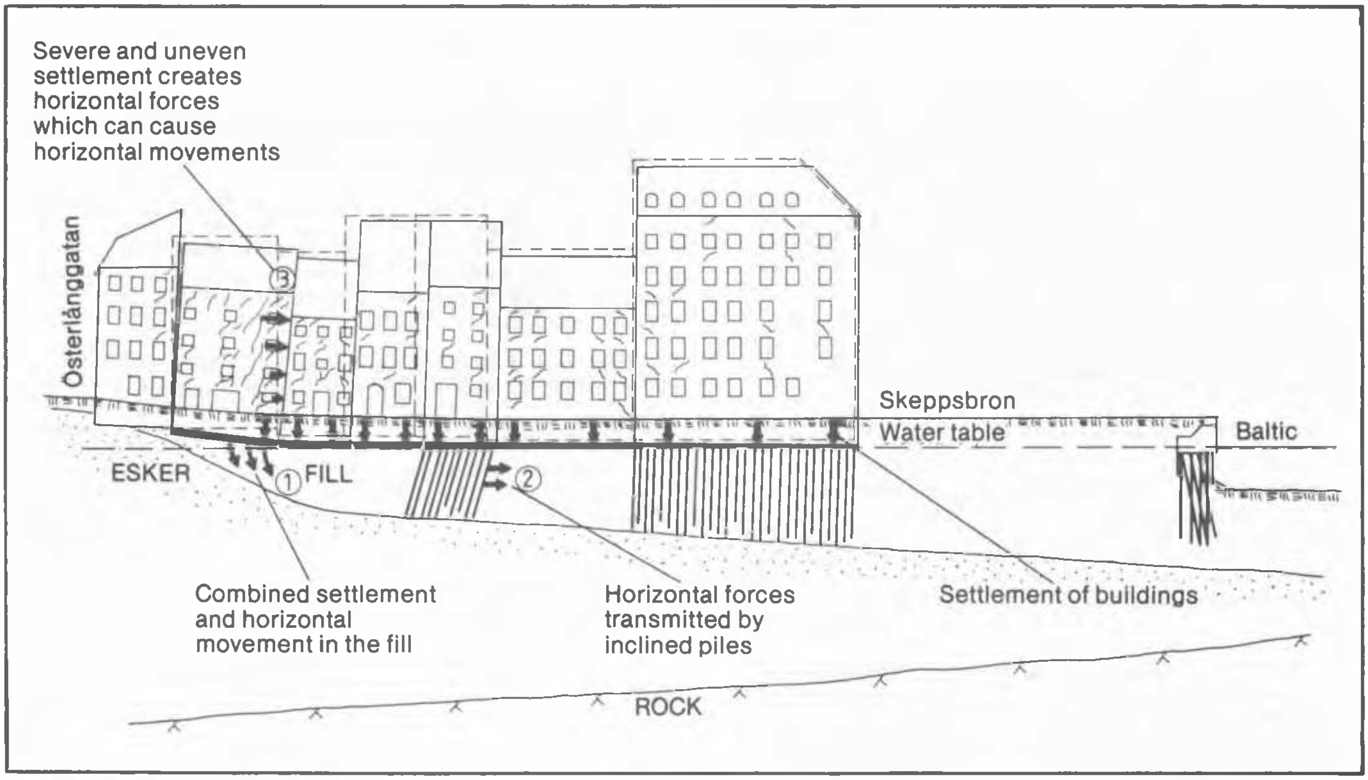
With this approach we may have a considerable advance in understanding of pile behaviour, including time effect, and hence in the use of pile bearing capacity. We believe that this way is a very promising one.

It is a pity that we do not have much data on behaviour of pile groups. And I think it is another direction in which we should work in the coming four years before the next International Conference.

On behalf of all members of the Session, I would like to thank our panel members who have worked hard to prepare for this discussion and to thank those who presented contributions to the discussions. I must apologize to those members who wished to speak but for whom we could not find time.

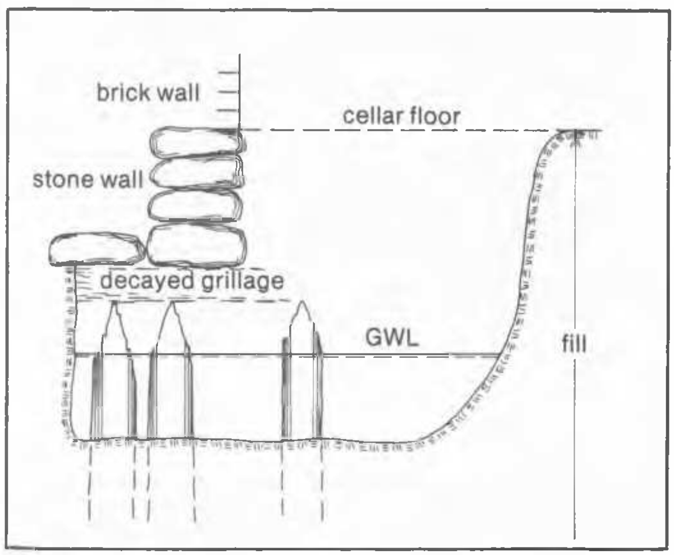
I express our particular thanks to Prof. Broms and Prof. Weinhold, General Reporter and Co-Reporter.

The Session is closed.



Principles for the creation of horizontal displacements of soil and buildings in the Old Town of Stockholm. (Technical Visit A)

Severe settlement damage in a courtyard building in the Old Town of Stockholm.(Technical Visit A)



Section through a test pit in a cellar in the Old Town of Stockholm. Timber grillage and pile tops decayed above ground water surface.(Technical Visit A)

Figures on this page are taken from the Publication "Combatting Subsidence in the Old Town of Stockholm" by Håkan Bohm and Ulf Stjerngren. Issued by the Swedish Council for Building Research and presented to the Delegates of X.ICSMFE.