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MAIN SESSION 3
3-e SESSION PLENIERE
ТРЕТЬЕ ПЛЕНАРНОЕ ЗАСЕДАНИЕ

MAIN SESSION III

DEEP FOUNDATIONS, INCLUDING PILE FOUNDATIONS (Design and New Methods of Construction)

Chairman: Prof. A.Kezdi (Hungary); General Reporter: Prof. W.L.Zeevaert (Mexico)

Members of the Discussion:
M.J.Tomlinson (U.K.), A.F.van Weele, The Netherlands,
E.Horvat, (The Netherlands), A.S.Vesic (USA),
L.G.Mariupolsky (USSR), Ch.Veder (Austria)
B.Černak C.E. (Czechoslovakia), W.Wittke (GFR),
Sh.Prakash (India), P.Klestchev (USSR),
Th.Tassios (Greece), D.Mohan (India),
A.Bartolomey (USSR)

Chairman Prof.A.Kezdi (Hungary)

In introducing the session dealing with deep foundations including piles, I would like to emphasize a few important facts which will determine, in my opinion, the future fate of this age-old foundation method.

Here are first the lessons we learned in the past decades and which seem today well-established among the members of the profession. We know today, that, in most cases, there is no ultimate load for piles, except for the cases where the pile itself fails. The process of failure is almost always a progressive one and no definite load value can be given as an upper limit. Then, it is with deep satisfaction, that I did not see any paper amongst the conference contributions dealing with any "refinement" or development of dynamic pile bearing formulae. Finally, the influence of the method of installation on the bearing capacity of a pile is being recognized more and more.

The conclusion we have to draw from these facts clearly shows that only an investigation of the behaviour of the whole pile foundation will further our knowledge in this field, an investigation of the method of construction, of the response of the soil both during installation and loading, both on short term and for long term, interaction between piles themselves in a group on the one hand and piles and structure on the other. Much effort was spent on the investigation of the ultimate load of an individual pile-which as I said actually does not exist-and apparently minor details, which, however, are in many instances of decisive importance, were neglected. To apply an old German saying here, it's been said that's easy to become father but it's difficult to be a father-so it's easy to become a pile but difficult to be a pile.

According to my belief, time has come for the investigation of the above mentioned complex behaviour. This will be possible, if we put together the information which is consolidated in theories, in construction practice and through the observation of existing structures.

However, there are requirements which have to be fulfilled before we start this noble work.

First, we have to improve the methods of construction in order to make them safe enough to ensure the anticipated conditions. And here I wish to mention that many failures of piled foundations were due to improper or careless construction work. But even if it does not come to be a failure, this way of handling the problems makes the application of any theory meaningless, because then, nobody knows, in what measure the inherent assumptions of the theory are fulfilled. In my opinion, the fear from an improper construction puts the factor of safety rather high causing unnecessarily high costs.

Second, we have to find out the changes an installment of a pile or of any other element of a deep foundation will cause in the soil and how this will influence its behaviour.

Third, a better understanding of group behaviour has to be sought. We still miss, for example, a suitable definition of pile groups and our knowledge regarding pile behaviour is rather limited.

Finally, the rational and prudent observation and analysis of existing structures on piles will furnish the basis on which the assumptions for the new theoretical investigation can be established.

In my opinion, time is ripe now to accomplish these tasks properly, using the joint efforts of the scientists and civil engineers in many countries.

Two final remarks will further enhance the importance of this work.

Proceeding in the way I proposed, a great number of piles which are unnecessary today, can be omitted which will result in a big economic gain. These piles are driven today partly for a rather high factor of safety which is used to balance our limited knowledge, and partly for other reasons.

Then, I would like to point out-and perhaps, this is a point I should have mentioned at the beginning-the field of the deep foundations is today a rapidly changing world. Old methods are dropped and new ones developed. It is sufficient to mention here the slurry trench walls. If we do not provide the necessary tools for their rational design, then hidden resources cannot be exhausted and they cannot be used economically.

With these ideas in mind, I wish the session a successful work.

Chairman A.Kezdi (Hungary)

Now I wish to invite Prof.Zeevaert, the General Reporter of our Session, to give a brief Summary of his State-of-the-Art Report on Deep Foundations including Pile Foundations.

Prof. W.L.Zeevaert, General Reporter (Mexico)

Ladies and Gentlemen,Members of the Board, old times Friends and Colleagues:

It is an honor to act once more as your General Reporter on Session 3.I wish to acknowledge my appreciation for this distinction to the Organizing Committee.

Many valuable contributions from various countries have been reported to this Conference on the behavior of piles,piers and pile foundations; however, no papers on deep compensated and compensated friction pile foundations were submitted.

The following is meant to be a brief summary report of conclusions and recommendations,based on findings given in the papers presented to Session 3.

I found in the contents of the papers presented, an advancement in the understanding of the behavior of deep foundations since the last International Conference. The engineering profession appears to have been investigating much, on problems that are still the concern of the foundation engineer, mostly under field conditions.

The material presented deals with the following subjects: skin friction and point bearing piles, pile tests, static and dynamic behavior of piles subjected to lateral loads, large diameter special piles and caissons, anchored plates and piles, pile driving,penetration tests, earth pressure on rigid foundation walls,and the settlement of buildings on pile foundations. The subjects treated in the papers may be found briefly explained in my General Report.

Field investigations on natural size piles and piers are expensive. Therefore,they are not carried out as often as desirable.However, when performed,the reward is always profitable because of the knowledge acquired in learning more on the mechanics of behavior of these foundations,and the ability to diagnose the problems better is always translated in economy.Therefore,proper investigations to determine the soil mechanical properties, in conjunction with well planned programs in the field, should be continued and highly encouraged.

Difficult subsoil conditions may be,in many cases,the only sites available for construction because of the increasing population and industry in large cities. In addition to a poor soil condition, the available zones may be subjected to environmental forces such as: ice,high wind,ground surface subsidence and earthquake strong ground motions. The influence of one or several of these forces cannot be overlooked when designing deep foundations. Under such circumstances, the foundation engineer is compelled to learn on the deformational behavior of deep foundations by the methods he may have at his disposal, like performing model tests, field pile tests,theoretical investigations using the mechanical properties of the soil, and last but not least,with the experience gained in observing the behavior of constructed foundations,that have been subjected to these environmental forces throughout several years of service. The observations should be compatible with each other,to be able to reach safe and practical conclusions from the engineering point of view, leading the foundation engineers to the ability of designing deep foundations with the best precision and economy.

The use of pile and pier foundations is increasing considerably, because heavy building foundations may prove economic when a good bearing stratum is not very deep seated. The problem of negative skin friction induced by ground surface subsidence,and the horizontal drifting forces set by strong ground motions, are two of the most important environmental forces that cannot be overlooked.

A deep pile or pier foundation is selected first,from the point of view of stability by its bearing capacity, given by the soil from the shear strength parameters or from penetration test devices; and second, the behavior of the foundation should be analyzed for total and differential vertical and horizontal displacements, and rotations, using the stress-strain-time properties of the subsoil materials obtained in the laboratory. The tilt produced either by eccentric loadings, differential soil properties or overturning moments produced by wind or earthquake forces, should be carefully investigated by the foundation engineer in every case,but most important in cases where difficult subsoil conditions are encountered. A forecast of the displacements and rotations to be expected in the foundation, should be performed for comparison with the limiting permisible or allowable codified values. The forecast shall be made with reasonable accuracy.

When reviewing the contributions presented to this Conference, I have kept in mind these two aspects of the problem in designing deep foundations, mainly: the stability given by the deformational mechanical behavior of the soil sediments.

The stability problem is that of a foundation breaking into the ground,it should be first fulfilled; and second,one should investigate the total and differential settlements produced by the mass forces of the building, and environmental forces encountered at the site or area under investigation. Proper stability is assessed for bearing capacity using adequate factors of safety,however in many cases, this practice is not sufficient if the vertical and horizontal displacements and rotations of the foundation cannot be approximately forecasted,to comply with the general demands or specifications of the project in question.

The engineering profession up to the present, in its research work, has performed extensive laboratory investigations in small models under ideal conditions, pertaining single and small pile groups. When the question arises of the design of a large foundation, in many cases extrapolation is performed. One has to keep in mind, however,that small and medium size scale tests, may be not representative of the environmental forces involved affecting the prototype, mainly when the time element and mass forces are present. Nevertheless the laboratory investigations performed in model piles, pile groups or pier like structures are important, since the academic work of this type is instructive to discover under ideal conditions the trend of behavior to be expected in the field, and be able to have the basis to set or calibrate theories and laboratory findings with field behavior. Unfortunately,there is no similitude law that may take into account all the forces involved in deep pile foundations, permitting prototype correlations. Therefore,the different aspects of behavior have to be worked out separately,and then from the knowledge gained in their understanding, the foundation engineer has to use approximate theories to set a background on which he can support his calculations.

Special reported tests on pile groups capped with a rigid foundation have indicated that pile loads are higher at the edge piles than at the centèr piles of the group. This fact is sustained also by theoretical investigations.

The investigations performed on skin friction in piles have yielded interesting conclusions. It has been found that negative skin friction may be estimated with reasonable accuracy for single pile tests. I think,however,that the main concern of the foundation engineer is to translate these results to large pile groups,since the diameter,point shape and geometrical layout of the piles, may invalidate the results of a single pile test, if applied to a large group.

The importance to reduce the negative skin friction forces to gain economy in deep pile foundation design ahs been investigated, and reported in several papers presented. This phenomenon is present when point bearing piles are driven through consolidating soft soil strata. The use of coatings with bitumen or bentonite to reduce the negative skin friction has been performed by different investigators. Nevertheless, no established quantitative recommendations can be given in the present,because of different methods employed. It appears that the reduction in negative skin friction when using coatings, is a matter of specific investigations and procedures, according to the individual experience gained in natural size pile tests, from observations performed in different countries. Laboratory investigations with coating materials and application techniques, appears to be necessary,to establish definite correlations with field tests to determine, in advance,using laboratory tests, the skin friction angle between the pile coating and soil. Learning on these mechanical properties a forecast at low cost can be made in usual foundation design practice. Also, from field experience, it is necessary to learn to what extent is the coating preserved during pile driving and afterwards, that is to say, if this beneficial effect is not reduced or lost during the life of the foundation.

I found general agreement on the phenomenon of heave produced when driving pile groups. The soil mass heaves corresponding to a fraction of the volume displaced by the piles, and high excess pore water pressures are induced in the impervious or semi-impervious soil mass within the piles. The excess pore water pressures, dissipate rapidly and the ground surface has the tendency to regain its initial elevation. The result is that the piles in the group work under negative skin friction forces immediately after pile driving. Observations reported have shown also that heave at the bottom of deep excavations is reduced, if the piles are driven before the excavation is performed.

The phenomenological observations just mentioned, are important to consider in current foundation design practice. The ratio of the volume per unit surface displaced by the pile group to the surface heave, should be recorded for different subsoil conditions, and correlated with the average permeability of the soil mass.

From the papers reporting investigations in model pile groups and single piles subjected to lateral loadings, conclusions are reached in the direction that the horizontal displacements of these piles may be calculated satisfactorily using theoretical methods of subgrade reaction, when the foundation moduli are properly assessed. It has been also proposed to assume a soil modulus varying linearly with depth. I think, however, that the accuracy obtained in these cases depends greatly on the experience of the foundation engineer to select the proper foundation moduli along the depth of the pile for the different strata encountered. When the problem is critical, probably the best method, suggested by one of the authors, is to perform natural size pile tests with at least three piles with different embedment lengths. From the pile deflection configuration observed in the tests, the real average foundation moduli for the different strata where the piles are embedded may be calculated.

Concerning soil interaction problems, I find necessary to clarify certain discrepancies I found in the papers concerning the use of the horizontal "coefficient of subgrade reaction", in contrast with the horizontal soil foundation modulus which is a function of the stress distribution along the pile shaft and of the end conditions. This concept may be clarified for future use in the literature and practical applications in foundation design, as already treated in Session 2.

Dynamic tests on actual size piles have been reported to learn on the behavior of these elements when subjected to vibration and alternating loads at their heads. Correlations have been proposed, and theories developed to learn on the modes of vibration of the piles when subjected to this type of dynamic loading. In connection with dynamic or kinematic loads on piles in foundation design, it is important to consider the environmental forces producing shear forces in these elements, like those induced by high wind and mass forces by earthquake strong ground motions, the latter inducing drift in the total length of the piles or piers, as the mass displaces horizontally by the seismic shear waves. To foresee in the foundation design the influence of these environmental forces, in the present state of our knowledge, the best procedure may be, according to my experience in those problems, to find theoretical solutions based on rational methods, using the mechanical properties of the subsoil materials. This of course, calls for precise laboratory testing on good undisturbed samples. In dynamic problems, the determination of the shear modulus of elasticity is important, since this is the only way to calculate approximately the response of the soil mass to strong ground motions, and by the same token, the response of pile or pier elements to dynamic loadings.

Great interest was given to problems concerning the ultimate loads of anchored piles and plates from model testing. Nevertheless, more information is necessary before reaching to final conclusions on the load displacement characteristics of anchors, used in different subsoil conditions.

Concerning the investigation reported on point bearing resistance, it appears that the general trend in the present state of our knowledge, is to take into account the compressibility of the soil, and reduce accordingly the bearing capacity factors appearing in our orthodox formulas. This has been done for the time being, relating the angle of internal friction with the relative density of the soil affected by the stresses induced under the pile points. When testing piles, it is generally agreed that a small vertical displacement mobilizes the skin friction along the pile shaft, in contrast with the vertical displacement necessary to mobilize the point bearing resistance. Nevertheless, my personal opinion in this matter, is that this phenomenon depends highly on the compressibility and shear strength of the soil of the bearing stratum, in contrast with the rigidity of the soil mass producing skin friction. The case may be that positive skin friction may not develop before certain point penetration has taken place.

I found very interesting the observations reported by our Russian colleagues, concerning the standard cone penetration tests, in connection with the necessity of calibrating the cone test device by means of correcting factors for different soil types, thus obtaining more accurate results when assigning point bearing capacity to different diameter size piles, and also, on the lateral skin friction capacity to be carried by natural size pile foundations.

The use of pre-bored piles or piers using the slurry displacement method has been reported. The point resistance is greatly reduced because of soft material trapped at the bottom of the hole. The procedures reported in two papers dealing with grouting under high pressure to enlarge the base and compact the soil at the bearing stratum, thus increasing the point bearing capacity and reducing vertical displacements, may provide a solution to this problem.

In connection with the standard cone penetration test to perform settlement forecasts, the conclusions appear not to support a reliable empirical correlation to ascertain reasonable accuracy. Therefore, in such cases, it appears that the only means to forecast more accurately settlements and tilting of structures under eccentric loadings, is to learn on the compressibility and consolidation properties of the subsoil strata affected by the stresses induced in the soil mass, where the piles transfer their skin friction and point loads.

Concerning settlement reports on single pile tests, I will stress the fact, that the results obtained in one pile will be only useful to learn on the behavior of the pile-soil interaction of that individual pile. Under the same working load condition, however, one pile in a group of piles does not necessarily comply with the behavior of an isolated pile, as performed in the natural size test pile condition. Moreover, soil properties under different subsoil conditions may invalidate the practice of using the results of a pile test at other sites.

Most interesting I find, the reports on the use of thixotropic jackets to install large caissons reducing side friction during installation and achieving waterproofing. I wish to invite kindly our Russian colleagues, to illustrate in more detail the procedures they have used to achieve this interesting heavy foundation.

Concerning papers on pile driving equipment, which has been always the concern of the construction foundation engineer, to obtain more efficiency and save time during driving, the reports indicate that single and vibratory hammers are still useful and efficient. Nevertheless, in sand and gravelly soils, jetting may be necessary to speed up the work, and be able to reach the necessary pile depth. In this connection, the reports show that the resonant pile driving hammers are specially useful for certain limiting values of grain size in cohesionless soils.

Finally, concerning deep compensated foundations, I wish to stress the point that there are many places where deep soft and compressible soil deposits are encountered. The firm ground is deep seated and the water table may be found close to the ground surface. In such cases and depending on the slenderness of the building, it may be indicated from the economy point of view, to design a compensated foundation. This solution is specially suitable in areas where ground surface subsidence is taking place. The use of deep pile foundations using point bearing piles, will introduce undesirable problems. One of them, is that long piles cannot be driven straight. Furthermore, and most important, in cases of ground surface subsidence, the buildings will emerge strongly from the ground surface, because of the relative vertical displacement between soil and piles. Furthermore, very large negative skin friction forces and generated at the shaft of these elements, increasing considerably their cost because of larger necessary safe point bearing resistance. In such cases, I have used for many years with satisfactory results, a compensated friction pile foundation, allowing the friction piles to work under their ultimate load capacity, and permitting the piles to penetrate the soft soil mass, as the ground surface subsidence takes place. The structure of the foundation is designed as a rigid mat, permitting the foundation slab to take subgrade reactions corresponding to a fraction of the total load of the building. The balance of the load of the building is taken by the friction piles under full capacity, thus obtaining a stable elasto-plastic design. The stress conditions induced in the soil mass should comply with the permisible vertical displacements. With the use of this type of foundation, one can reduce total and differential settlements and produce an economical design. Moreover, on a long time basis, the foundation will follow the ground surface subsidence, and the phenomenon of the building emerging from the ground surface is eliminated.

Finally, I wish to make the announcement that because point 4 of accepted topics for contributions has been treated with amplitude in Session 2, it has been substituted by "Optimum Methods of Construction of Open Caissons and Trench Walls in Thyxotropic Clay Jackets". I thank you very much for your kind attention.

Chairman Prof. Arpad Kezdi

Thank you very much Prof. Zeevaert for your interesting summary of the State-of-the-Art paper. Now I wish to invite the delegates to take part in the discussion wish to call upon Prof. R. Peck to make some comments before the beginning of the discussion.

Prof. Ralph B. Peck (USA)

In past Conferences, most of the papers on deep foundations have been concerned with theories of deep foundations, methods of computing bearing capacity and settlement, types of piles and their behavior as individuals under test loadings, and occasional comparisons of computed and observed settlements. All these subjects have an important place in our profession. Yet, there are other aspects that demand our attention as foundation engineers, not so readily subject to calculation but vital to the success of our installations.

Among these are techniques of construction to assure that the foundation units will actually have integrity and will perform as we anticipate. Drilled-in piers, driven piles, and cast-in-place piles are all subject to defects due to faulty construction, often aggravated by the choice of a type of foundation poorly suited to the subsoil and groundwater conditions. I believe it is safe to say that most of the foundation failures in the last ten years-and there have been many-have not been caused by erroneous estimates of bearing capacity or settlement, but by defects in the foundation units themselves. There are all too many examples of drilled piers and cast-in-place piles with discontinuous concrete, of steel H-piles deflected by boulders and other obstructions to positions and shapes in which they are incapable of supporting the design loads, and of other similar faults.

Most of the construction defects of drilled piers have their origin in the presence of water. Various methods have been devised to cope with the water; most of them are simple in principle but depend acutely on details of procedure and execution. Most published descriptions of successful methods have been lacking in details. I am pleased to see that several of the Conference papers contain detailed descriptions of successful methods of installing deep foundations. This trend should be encouraged. These so called practical matters should not be disdained because they are not scientific. Unless they are properly considered and dealt with, the endproducts of our profession, the foundations themselves, will be unsatisfactory and our scientific advances will be of no avail.

Theoretical considerations and fundamental knowledge of soil behavior also can suggest improvements in construction. It has long been realized, for example, that larger settlements are required to develop the base resistance than the shaft friction on drilled piers. The ingenious pressure-grouting procedure for prestressing the sand at the base of such piers, described in two papers presented for this session, is a striking example of a construction detail designed to take advantage of fundamental behavior.

Our profession has been a leader in publishing information about failures for the good of all our colleagues. This willingness to publicize our errors has permitted us to improve practice. It is not enough, however, to describe what went wrong. We need more information, such as that contained in several papers to this Conference, about procedures devised specifically and successfully to prevent such errors.

Nor can we separate from our practice the training and supervision of workmen and inspectors. Better, more refined, or more rugged machines are not enough. When we install a foundation by a machine working at a depth below which we can actually see what is being done, the smallest misjudgment on the part of the operator may have the most serious consequences. Hence, these matters, too, deserve reporting and discussion in our literature.

Chairman Prof.A.Kezdi

Thank you very much Prof.Peck for your comments. Now we have an interval after which I would like to call upon Prof.Tomlinson from the United Kingdom to make his discussion

Prof. M.J.Tomlinson (United Kingdom)

The general reporter has suggested that the subject of coatings to reduce negative skin friction should be discussed. I would like to mention the more important matter of coatings of soil on the shafts of driven piles which reduce the positive skin friction. That is the skin friction which acts in support of the pile. In most practical cases of piling for structural foundations we are concerned with piles driven through an overburden of weak compressible soil to obtain their required carrying capacity in skin friction and end-bearing in the underlying stratum of dense or stiff soil.

When piles are driven through an overburden of soft clay a coating of this material is picked up by the pile shaft and is carried down into the lower bearing stratum. Because the soft clay coating is weaker than the soil around the pile in the lower stratum the available skin friction is reduced in the same way as a soft bitumen coating reduces the negative skin friction. As the pile penetrates more deeply into the bearing stratum,the soft clay skin is rubbed off. However,if the bearing stratum is a stiff clay a coating of this material is formed on the pile. This is beneficial to skin friction since the stiff clay skin is drier and more compact than the surrounding clay, and it also increases the effective diameter or width of the pile.

Fig.(x) shows the effect of driving a 432mm steel tube through a soft clay into a stratum of gravelly sand. It will be seen that a coating of soft clay is adhering to the middle part of the pile shaft but this coating had rubbed off where it had penetrated the gravelly sand. Comparison of the length of the cleaned-off portion of the tube with the measured penetration into the gravelly sand showed that the soft clay skin had been carried down some four or five shaft diameters into the sand stratum.

Fig (Y) shows a coating of soft clay about 5 mm thick which had been carried down adhering to the trough of a steel sheet pile. This skin had been carried down 3 metres or more into a stiff boulder clay.

The beneficial effect of a stiff clay coating on a pile shaft has been mentioned. A skin of sand carried down by a pile into a firm or stiff clay can also increase the unit skin friction on the shaft.

It must be recognised that when we attempt to evaluate the skin friction on a pile driven into clay we are concerned with a failure condition which is not one of relative movement between the pile and the adjacent soil but it is one of slipping between a skin of soil carried down by the pile and the adjacent soil which is itself heavily sheared and compacted.

The effects of soft clay and sand coating on steel tube piles driven into stiff London Clay were studied in a research project undertaken for the Construction Industry Research and Information Association (CIRIA) in UK (I). The results of this research and studies of published records of piles driven through different overburden soils into stiff clay have been embodied in a set of design curves relating adhesion factors to the undrained shear strength of clay(2).

9 Ft. BELOW TOP OF BOULDER CLAY

(1) Tomlinson, M.J.,The Adhesion of Piles in Stiff Clay, CIRIA Research Report No 26, London, 1970.

(2) Tomlinson, M.J. "Some effects of pile driving on skin friction",Conference on behaviour of Piles,Institution of Civil Engineers, London,1970 and Author's reply to the Discussion.

Chairman Prof. A Kezdi

Thank you Prof.Tomlinson. Now I invite Mr. van Weele from the Netherlands.

Mr.A.F.van Weele (The Netherlands)

A large percentage of all foundation piles in the western part of the Netherlands are subject to negative skinfriction. Several pile foundations have been installed in which use was made of precast concrete piles coated with a special bitumen layer of approx. 10 mm thickness to reduce this negative friction to only a few tons per pile instead of over 50 tons without coating. One of the questions raised for discussion is to what extent such a layer can be preserved during driving.

It is a fact that in general during driving the stresses between pile and soil are a maximum and therefore higher than during actual loading. As,however,the negative friction along the upper parts of the pile reaches its ultimate value rather soon after the pile's installation, the shearstresses during driving as well as during the pile's use are practically the same. The only difference is the rate of deformation. The bitumen used for coating should act as a solid material during fast deformation and as much as possible as a fluid during slow deformation.

This has proven to be not too difficult to solve. An entirely different behaviour can be expected when the piles are not driven by impact but by vibration. It is well known that during vibratory driving the characteristics of many soils are changed in such a way that much less friction is developed along the pile's surface. Under such conditions the risk of damage to the pile's coating is much less.

I also like to draw your attention to another problem. In our country during more than 30 years all kinds of piles have been used which are equipped with enlarged bases. For prefabricated concretepiles these enlarged bases have a projected area which is usually up to a maximum 3 times the cross-section of the pileshaft. Such piles are ideal in situations where the soft compressible top layers are underlain by the bearing stratum. The large shoe may have a decreasing effect on the shear between soil and pileshaft, but its main advantage is the substantial increase in the pile's endbearing capacity. This design enables us in many cases to utilize the full strength of the pileshaft and not to overstress the bearing layers. A disadvantage is its more difficult driving than that of a normal pile,especially when driving raking piles.

For vibratory installation of prefabricated piles a special pileshoe has been developed which can have any size and needs not to be limited to 3 times the shaft's cross-sectional area. This pileshoe which in its form looks like a lattice acts as if it is fully open during dynamic loading,but shall act as if fully closed during static loading. By a correct choice of size of each compartment in relation to its height and eventually the inclination of its outside such a "closed behaviour" is obtainable in all kinds of bearing layers.

During vibratory driving the lattice shoe behaves much better than the closed shoes during impact driving,thus eleminating the known disadvantages.

Such piles can be compared with columns founded on isolated footings. The only difference is the depth of foundation. Such piles can be installed as easily as traditional piles without enlarged bases and are very suitable to withstand negative friction.

Chairman Prof. A.Kezdi.

Thank you Mr. van Weele for your discussion Now Mr. Horvat, please, deliever your contribution.

Ir.E.Horvat (Netherlands)

Theoretical studies and field investigations have demonstrated that negative skin friction on foundation piles can be reduced if the piles are provided with a slip layer of bitumen. The grade of reduction is determined by the material properties and the thickness of the bitumen.

For the foundation of new Shell plants near Moerdijk around 5000 of this kind of piles were used. Prefabricated concrete piles, 38x38 sq.centimetres, 19 meter long,were provided with a bitumen layer along the upper 10 metres. The bearing capacity of these piles is delivered mainly by positive skin friction along the part of the pile which is not coated (see fig.1).

The practicing engineering requirements concerning these piles can be summarized as follows:

1) The nagative skin friction must be reduced to a neglectable value.

2) The slip layer must be reasonably cheap and it must be possible to apply it in a simple and reliable way,also on prefabricated piles before these are brought into the soil by driving,pushing or turning.

3) During a few days' storage of the coated pile the slip layer must not flow too much.

4) During driving the slip layer must not crack or shear off from the piles as a result of impacts and shear forces.

5) The slip layer must not be forced upwards or downwards by differences in horizontal soil pressure.

6) Coarse grained soil layers must not penetrate into the slip layer.

In the development of a special type of bitumen for this purpose at KSLA (Shell Research Laboratory Amsterdam) optimum use has been made of a specific relation between mechanical properties,loading time and temperature of this visco-elastic material. It is thin-liquid at high temperatures (easy to apply), flows very slowly at ambient temperature and medium loading times (storage), it

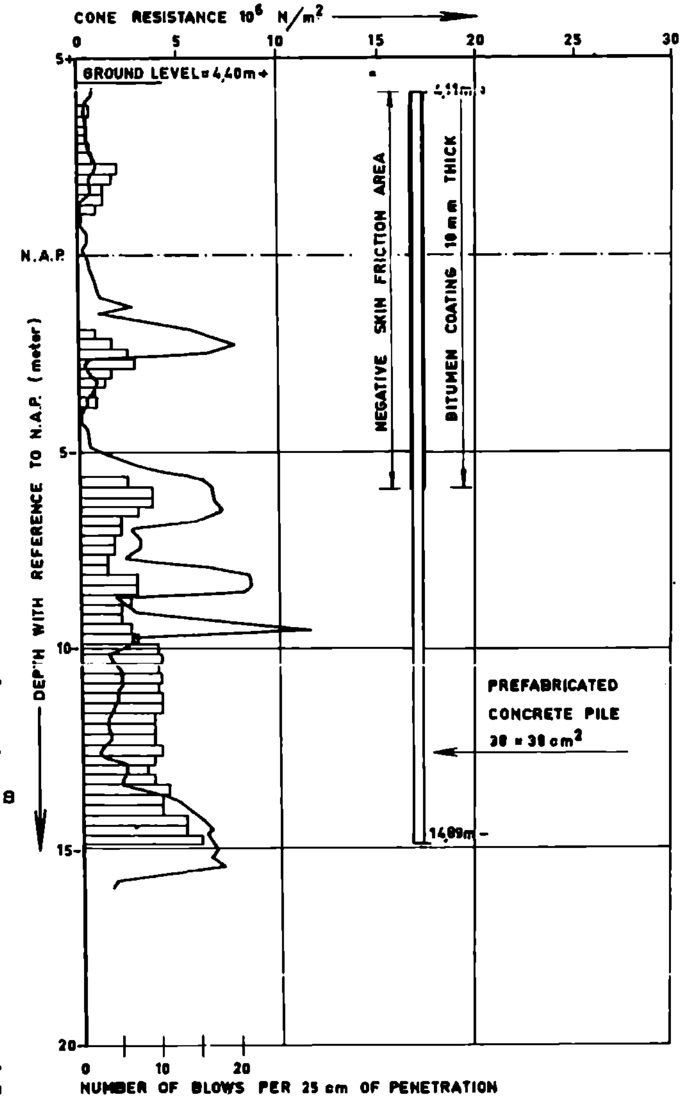


FIG. 1

RESULT OF A CONE PENETRATION TEST WITH THE PILE TYPE, DRIVING DIAGRAM, PENETRATION DEPTH AND LENGTH OF THE BITUMEN COATING.

is almost elastic at average outdoor temperatures and short loading times (driving) and has a low viscosity at soil temperature and very long loading times (low frictional resistance against soil settlements).

In establishing the requirements the bitumen of the slip layer has to meet a layer thickness of 1 cm was taken. Thinner layers (<0.5cm) will often be too vulnerable and thicker layers (> 1.5cm) will flow too much during storage of the piles. Besides, the following general assumptions were made:

- The permissible negative friction on a pile having a length of about 40 m and a cross-section of about 50x50 cm is assumed to be about $5x10^4$N(5 tons) if the average rate

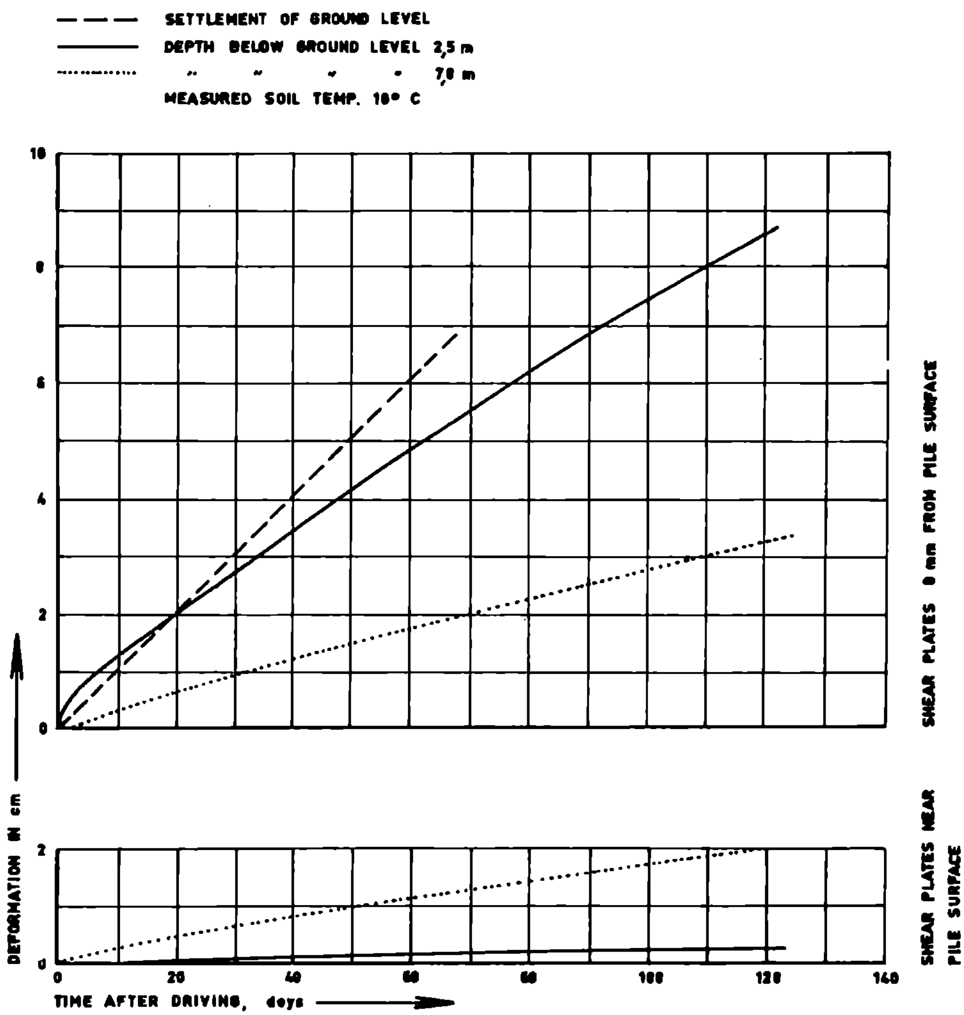


FIG. 2

VERTICAL DISPLACEMENT OF THE VARIOUS SHEAR PLATES IN THE BITUMEN LAYER.

from „Reduction of negative friction on bearing piles by bitumen slip layer"
by CLAESSEN and HORVAT

of settlement of the soil along the entire pile is about 10 cm/yr. Hence, the viscosity of the bitumen at about 10°C (Soil temperature):

approx. $2x10^9$ Ns/m^2

- The maximum flow of the bitumen after about 3 days storage of the pile at 20°C is taken to amount to 0.5–1 cm. The minimum permissible viscosity of the bitumen is then:

  approx. $2.5x10^7$ Ns/m^2

- It is assumed that during pile driving the bitumen must be able to withstand a strain of approx. 10^{-2} due to 10^3 loads at approx. 0.02 s applied at a temperature of approx. +5°C. The stiffness modulus of the bitumen under these conditions must then be:

  approx. 10^8 N/m^2

With the aid of Van der Poel's nomograph* these requirements can be expressed in terms of the conventional bitumen specifications; penetration and softening point R & B. It

*Van der Poel, C. "A general system describing the visco-elastic properties of bitumens and its relation to routine test data", Journal Appl. Chem. 4 (1954), 221.

then appears that none of the current grades of bitumen meet these specifications. The requirements mentioned above are met by a bitumen grade which is -roughly - characterized by:

Penetration at 25°C:40-50 0.1mm
Softening point R&B: 60-70°C

This type of bitumen has the following properties:

Viscosity at 10°C : approx. 1.5×10^9 Ns/m^2
"-" -"- 20°C : approx. 10^8 Ns/m^2

Stifness modulus at 5°C and 0.02 s: approx. 9×10^7 N/m^2

It has been found that such a bitumen is not forced upwards or downwards by differences in horizontal soil pressure (order of magnitude of displacements 1 mm/yr) and that layers of coarse-grained soil hardly penetrate into the bitumen layer (the penetration is only 1 cm after as long as about 130 years, the K value of the soil being 5×10^{-4} m/s).

Naturally the specifications of the bitumen can be modified to meet the special conditions of a particular application as regards permissible friction forces on the pile, rate of settlement of the soil, temperature of the soil, temperature during driving, manner in which the pile is brought into the soil, temperature during driving, manner in which the pile is brought into the soil, etc.

In order to check the assumptions and calculations a number of piles was test-loaded and the shear in the slip layer was measured. For the measurement of shear in the slip layer sets of shear plates were mounted at distances of 0 and 8 millimetres from the pile shaft, and at locations corresponding to depths of 2,5 and 7 metres below groundlevel. On one of the piles thermocouples were installed to measure the soil temperature.

The vertical displacement of the shear plates at the various levels was measured at intervals of about fourteen days over a period of about four months after pile driving. At the beginning shorter intervals were taken. At this time the piles were not yet loaded.

Fig.2 shows for one pile the vertical displacement of the various shear plates as well as the settlement of the ground surface, both expressed as a function of time. After a short period all shear plates shear at an approximately uniform rate, which decreases with depth. Also, the shear rate at the surface of the bitumen layer at different depths can be derived from fig.2. From the shear rate and the viscosity of the bitumen the shear stress acting on the surface of the slip layer can be calculated according to

$$\tau = \eta \times V/h$$

in which

η = viscosity,
V=shear rate at the surface of the slip layer,
h= thickness of the slip layer.

In this way the shear stress along the slip layer was calculated for three piles from the shear rates measured at various depths. By integration over the surface of the bitumen layer the shear force-in other words, the residual negative skin friction-was calculated for each of the test piles. The values obtained vary between 2,5 to 3 tons; this means a reduction of the negative skin friction by 90% to 95%.

The results of the test loads showed an average settlement of the (individual) loaded piles of 4 millimetres under the working load of 60 tons. One of the test loaded piles was extracted for the purpose of inspecting the bitumen slip layer. The bitumen slip layer proved to be in good condition; the thickness of the layer varied from 8 to 11 millimetres.

Furthermore the settlements of a number of constructions were measured during the construction period and the following 2 years. The average settlement of these constructions was 9 millimetres during the construction period and 3 millimetres in the following 2 years. The settlement of the groundlevel was 150 millimetres in the same period of time.

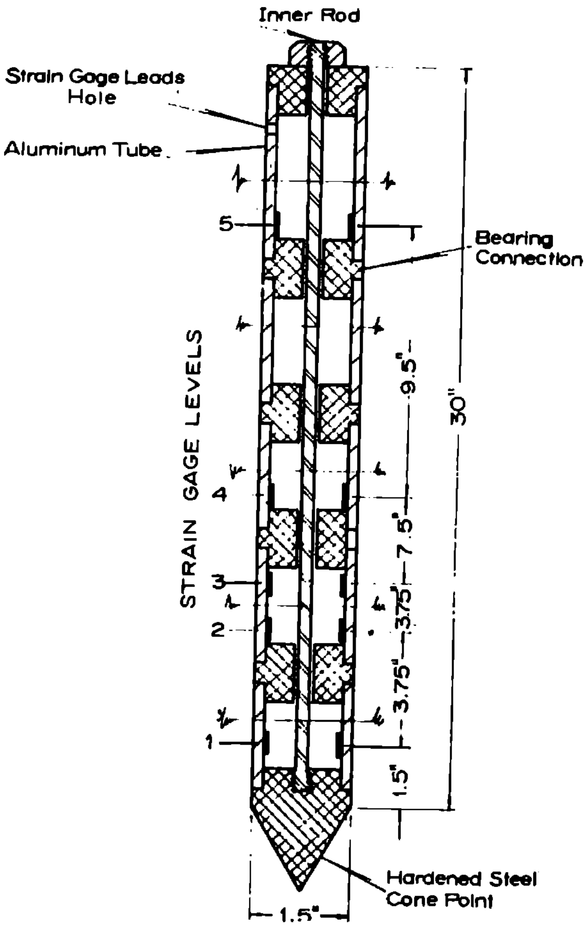
Chairman Prof.A.Kezdi.

Thank you very much Mr.Horvat for your information. Now I wish to call upon Prof. Aleksandar S.VESIC, from Duke University, USA.

Prof.Dr.Aleksandar S.Vesic, (USA)

I should like to present some comments relative to the topic 2 proposed by the General Reporter.- It has been recognized for a long time that the penetration resistance in sand depends on its angle of shearing resistance o and on the vertical ground stress q. Over the last decade we have come to full realization of the fact that the penetration resistance is also greatly dependent on the modulus of deformation and the volume change characteristics of the material. Finally, we started assembling some evidence that the horizontal ground stress or the coefficient of earth presure at rest K_o also may have an effect totally unaccounted for in classical theories.

For the last two years we have been engaged at Duke in a study aimed primarily on investigating the effect of K_o and separate evaluation of effects of strength and compressibility of sand on its bearing capacity at great depth. Our research includes laboratory model studies, correlation with field data, as well as the development of appropriate theories. The following slides show the experimental set up developed for this purpose and some of principal results.



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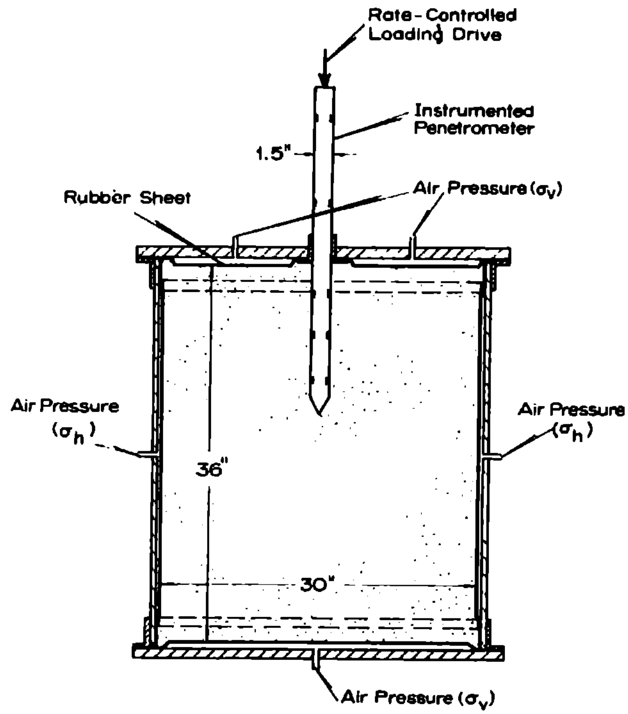
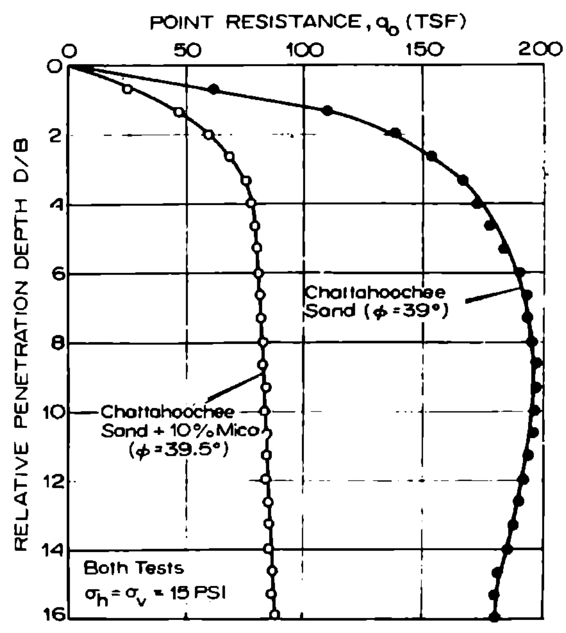
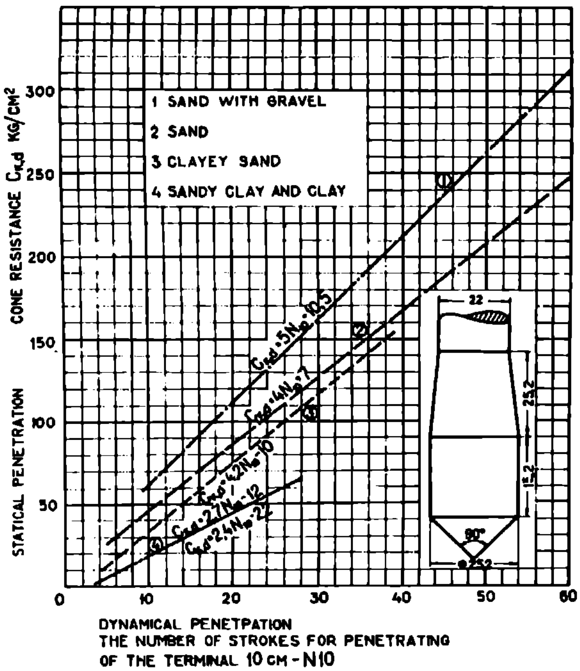
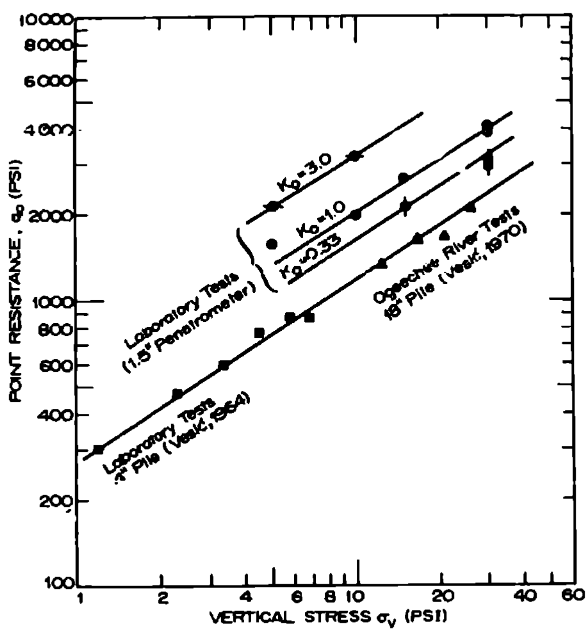
Model penetrometer

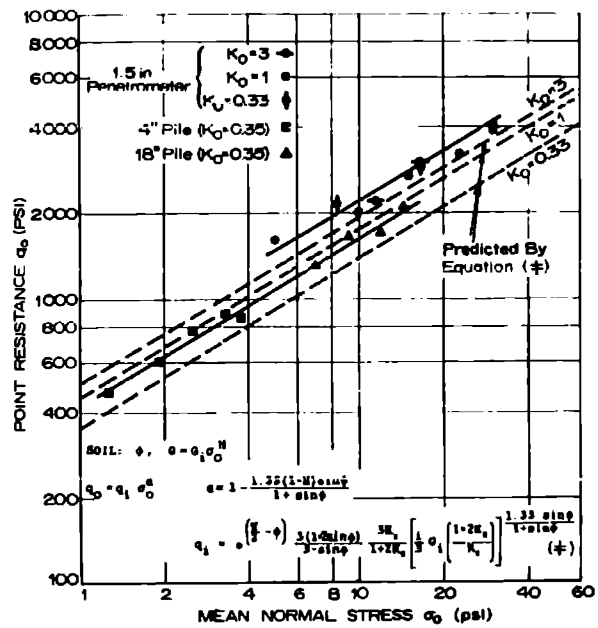


Experimental setup




1 SAND WITH GRAVEL
2 SAND
3 CLAYEY SAND
4 SANDY CLAY AND CLAY
CONE RESISTANCE C KG/CM²
STATICAL PENETRATION
DYNAMICAL PENETPATION
THE NUMBER OF STROKES FOR PENETRATING
OF THE TERMINAL 10 CM - N10


Ko=3.0
Ko=1.0
Ko=0.33
Laboratory Tests
(1.5" Penetrometer)
Ogeechee River Tests
18" Pile (Vesić, 1970)
Laboratory Tests
4" Pile (Vesić, 1964)
POINT RESISTANCE, qo (PSI)
VERTICAL STRESS σv (PSI)


1.5 in Penetrometer
Ko=3
Ko=1
Ko=0.33
4" Pile (Ko=0.35)
18" Pile (Ko=0.35)
Predicted By Equation (≠)
POINT RESISTANCE qo (PSI)
MEAN NORMAL STRESS σo (psi)

Conclusions:

Significant factors influencing penetration phenomena include horizontal and vertical ground stress, as well as strength, deformation and volume change characteristics of the soil.

Penetration resistance of Chattahoochee sand increases roughly as the 0.6-power of mean normal ground stress. The phenomenon of vertical arching around the pile tip has an additional effect on this relationship.

The attempts to correlate relative density of sand with penetration soundings without regard to the K_o-value of the natural deposit do not offer much promise. A preconsolidated deposit will always indicate higher penetration resistances for the same void ratio than a normally consolidated deposit.

The results obtained so far indicate a unique relationships between penetration resistance and the mean normal ground stress. Unique relationships exist also between the skin resistances of the friction sleeve and the conventional Dutch cono shaft and the mean normal stress; the former being about twice as large as the latter.

Chairman A. Kezdi. (Hungary)

Thank you very much Prof. Vesic for your discussion. Now we will hear Mr.Mariupolsky from the USSR

Mr.Mariupolsky L.G. (USSR)

"Transition coefficients from the data of static sounding to bearing capacity of piles"

The analysis of 153 load tests of driven piles made concurrently with sounding tests allowed to introduce corrections to the recommendations of the Building Standards of 1967 for the determining the bearing capacity of the driven piles by the data of static sounding. According to previous recommendations the bearing capacity of a pile (P_1) was determined by using two constant coefficients. For determining the soil resistance under a pile point the coefficient of 0.5 to the specific resistance of the soil under a cone was taken. When determining skin friction of a pile, specific friction along the pile and the sounding tube was assumed to be equal, i.e. the transition coefficient was taken to be unity. The correction to that recommendations was made by assuming that both coefficients are variable and depend on soil strength (Trofimenkov Ju.G.and Mariupolsky L.G.,1973).

This allowed the bearing capacity of a pile (P_2) determined by the data of the static sounding to be very close to the bearing capacity (P_{st}) determined by load tests (Fig.1)

An analogy of physical phenomena in penetrating a sounding tube and settling a pile under the load gave impetus to search a direct relationship between the total soil resistance in sounding and the ultimate resistance of the pile.

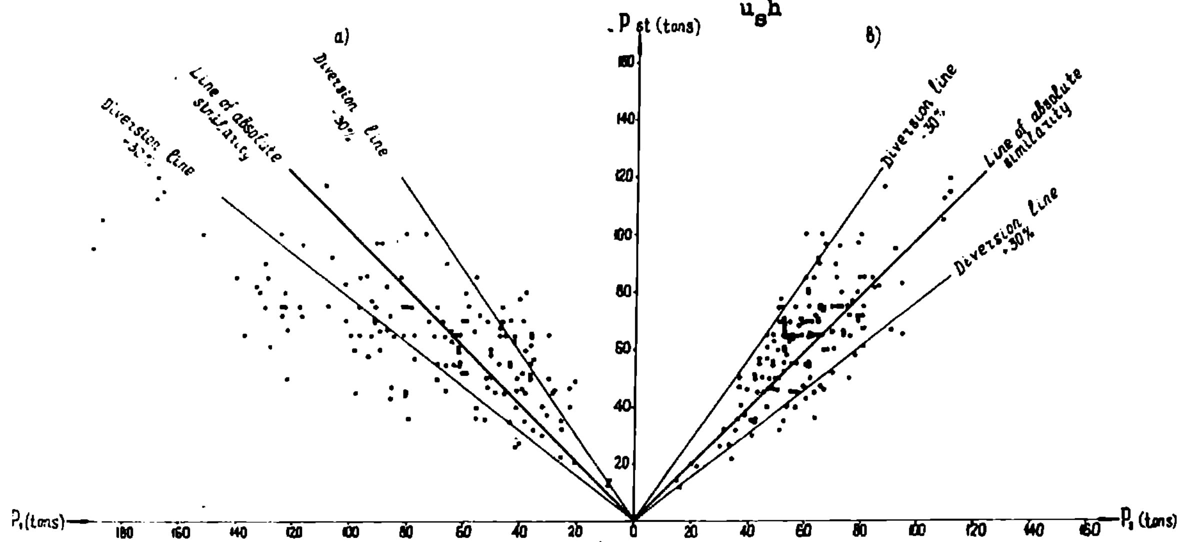
The analogy mentioned above allows the relationship between the sounding (P_s) and the pile resistance (P_o) to be expressed as

$$P_o = kP_s, \quad \text{where} \qquad (I)$$

k- transition coefficient from the total sounding resistance to the pile resistance

The comparison of the data of the sounding and the load tests of piles and their models*) shows that the transition coefficients k, is directly proportional to the ratio of perimeters of a pile and a sounding tube and that it depends on the strength of the soil around the tube. The soil resistance, f_{st}, is determined by the ratio of the total penetration resistance to the surface area of the sounding tube as the first approximation:

$$f_{st} = \frac{P_s}{u_s h}, \qquad (2)$$



*)a model pile is a steel tube with conical point driven at the same depth as a pile but smaller in cross section

where

u_s - sounding tube perimeter

h^s — depth of penetration of a sounding tube at the level of the design pile point

The design formula may be expressed as follows:

$$P = k_o \frac{U_p}{u_s} P_s, \qquad (3)$$

where k_o - coefficient depending on the soil resistance f_{st}.

The evaluation of the results of load tests and deep sounding has shown that the coefficient, k_o, is within 1 to 2 and may be accepted as

$$k = 2 - 0.1 f_{st}; \qquad (4)$$

when $f_{st} > 10\ t/m^2$, it should be assumed that $k_o = 1$.

The values of the pile resistance calculated by the formula (3) are compared with the values determined by the results of the load tests and are given in Fig.2b and Table I.

Table I

The results The method of the determination of the bearing capacity of a pile	Deviations from the results of the load tests		
	Deviation more than 30%	The mean value of deviation in %	
		above nominal value	below nominal value
When using constant coefficients	70 (46%)	40	25
When using two variable coefficients	23 (15%)	20	16
By total tube resistance	37 (24%)	28	21

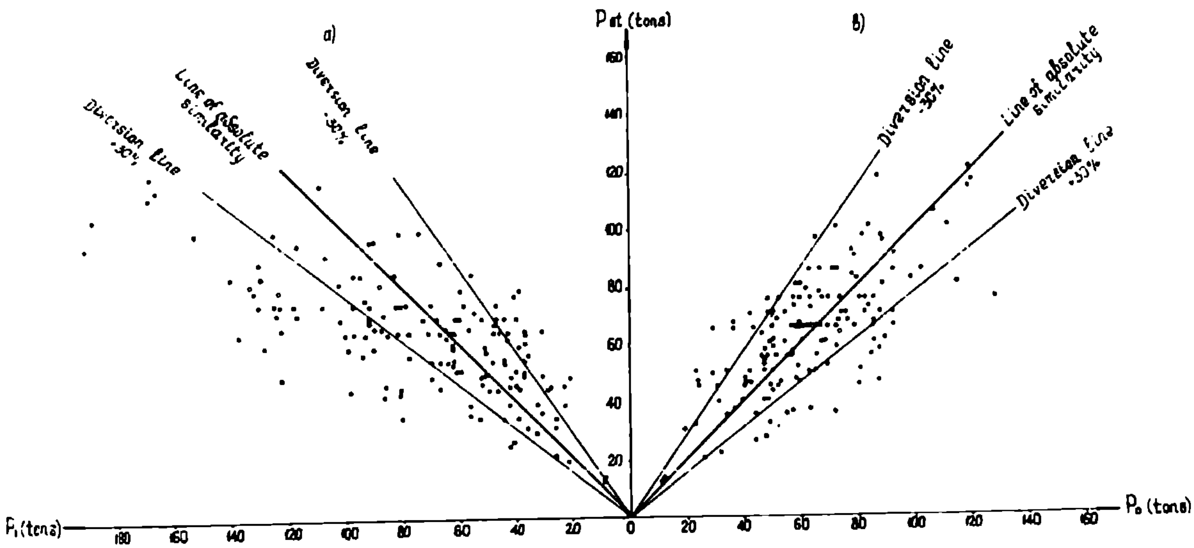


Fig. 2

As it can be seen from Figs.1,2, and Table I, the pile resistance determined by the total tube resistance is more accurate than that determined as a sum of a pile point and skin friction resistance with two constant coefficient.

It is expected that further increase of the experimental data will allow the proposed recommendations of the pile resistance by the total soil resistance to make more accurate. It gives a prospect of simplifing greatly the sounding apparatus as in this case the need of separate determination of the soil resistance under a cone and along a sounding tube is eliminated.

REFERENCE

The Proceedings of the VIII International Conference on Soil Mechanics and Foundation Engineering, v.II-1, 1973, Moscow.

Chairman. A.Kezdi

Thank you Mr.Mariupolsky for your contribution. Now I call Prof. Veder.

Prof. Christian Veder (Austria)

In Vienna the construction of an "International Office- and Conference Center" (IAKW) is under way. The building area is situated close to the Danube River. As there is an old river bed with Loose sediments and rubble fill of irregular depth, the design of the foundations had to encounter some difficulties. The whole complex consists of several Y-shaped towers of different height (up to 110 m) which are arranged round a circular conference hall (Fig 1). Four towers of the first stage are in construction. In order to make the foundation rather rigid, the design of my colleague Borowicka and myself

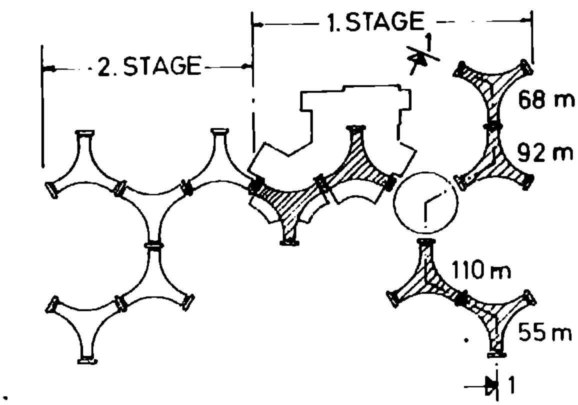


Fig.1: LOCATION AND HEIGHT OF BUILDINGS

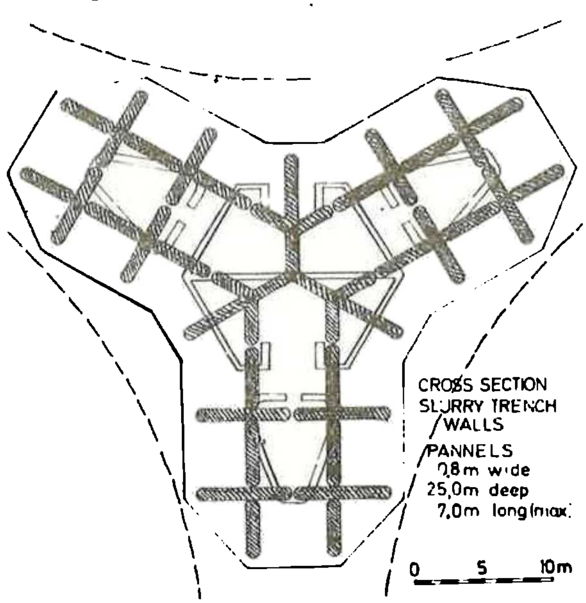


Fig. 2 Foundation central part

was estimated to be a good solution, that is to form a rectangular pattern of slurry trench walls. Size and centerline distance were calculated from the vertical loads so that the specific load for 1 m runs 350 t/m (in cross section) corresponding with that which leads 10 mm total settlement in the load test. The centerline distances were big enough as to avoid- owing to the test results- a direct influence from one panel to the other. The total settlements considering the group effect are by experience supposed to be small enough to ensure that the differential settlements for the max. extension will not exceed the limit value of 2 cm. The foundation depth was designed at the 25 m below ground level so that the first layer of fine silty sand would be penetrated in any case. The pattern of slurry trench walls for the 110 m building is shown in Fig.2.

Chairman A.Kezdi.

Thank you Mr.Veder for your discussion. Now, Mr.Chernak, please.

Eng. Boris Černak. (Czechoslovakia)

On the basis of strain gauge measurements an analysis of behaviour of a pile for the new Danube bridge in Bratislava, Czechoslovakia, was made. The pile of rectangular section of 0,6x3,0 m and 25,7m long was made by the "slurry trench construction" method. The shaft was excavated with a grab in water saturated gravels /left side in Fig .1/ /$\emptyset'=38^o$/, stiff silty clays /depth 6 to 13 m, $I_p=21,7$, $I_C=0,98$, $c_u=0,57$ kg.cm^{-2}, depth 19 up to 23 m, $I_p=34,2$, $I_C=0,88$, $c_u=0,41$ kg.cm^{-2}/, fine clayey sands and silty sands /$\emptyset'=28^o$/. Bentonite suspension affected the shaft walls in the region of the pile head for 126h, in that of the pile base for 30 h. The shaft width was determined with a lever cavernometer before concreting /left side in Fig.1/. In a gravel layer in depth from 2 to 5m was found a shaft enlargement up to 160 cm.

The pile was loaded in four loading cycles up to 2.040 tons. The distribution of pile load in the 4-th loading cycle /Fig.1./ shows a high load transfer into the gravel layer in the region of the pile shaft enlargement. The load transfer in clays and sands increased up to 1.020 tons. For the higher loads a decrease in load transfer occured in clays and sands, and the load increment was transferred prevailingly through the pile shaft enlargement into gravel and through the pile base into sand.

The base resistance versis base displacement curve /Fig.2, curve a/ shows that the contact between the pile base and the sand was close. The skin friction in clays and sands /Fig.2, curves c/ through f/ reaches the maximum value already for a 2-3mm displacement and decreases later up to the residual value. The skin friction reduction to zero value in the segment close above the pile base /Fig.2 curve f/ can be related to the

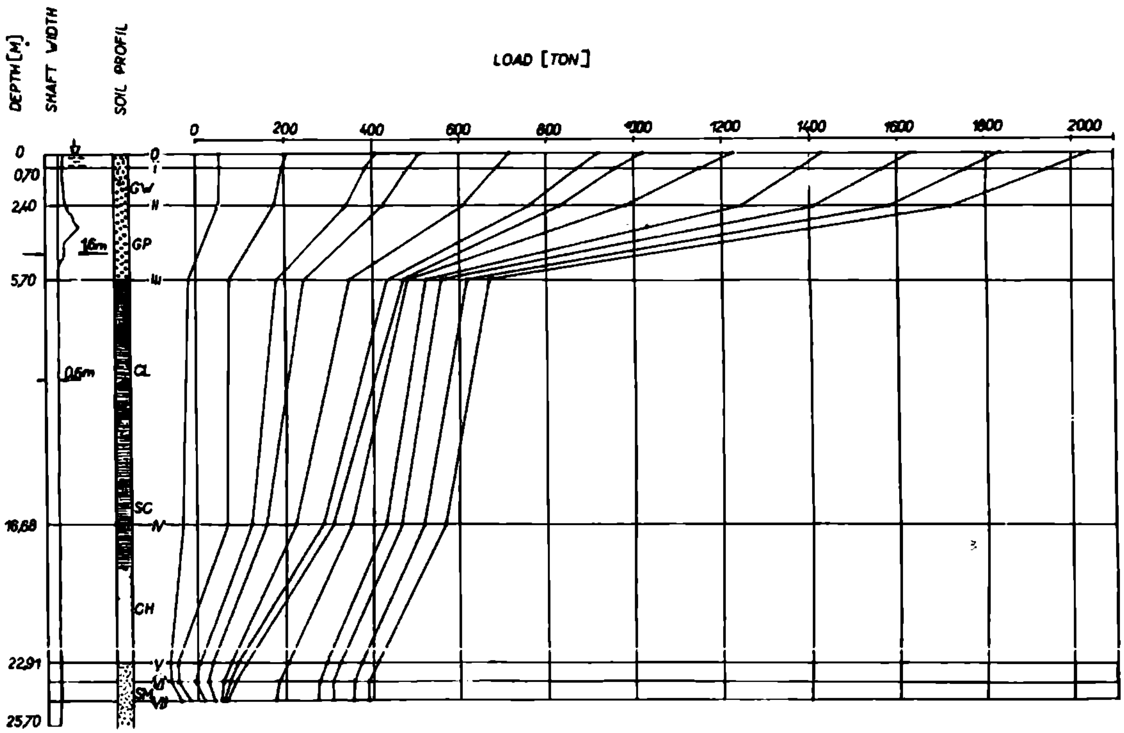
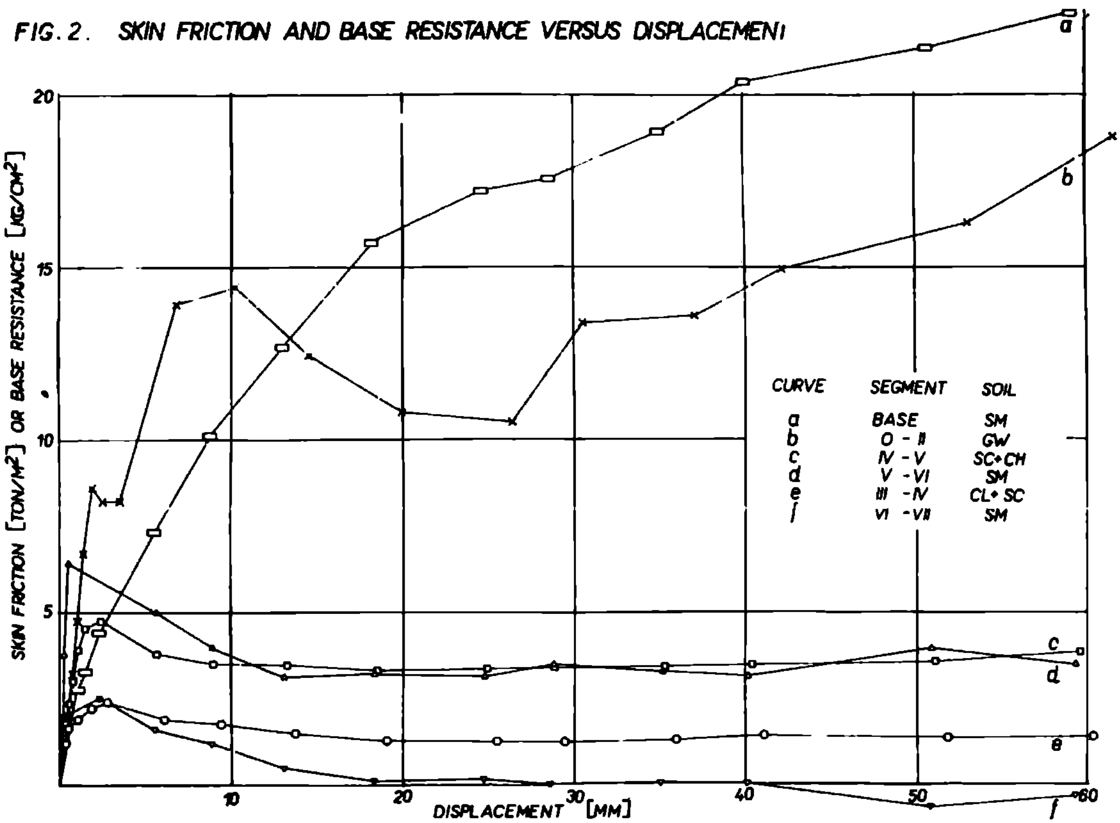


FIG. 1. DISTRIBUTION OF PILE LOAD [4-TH LOADING CYCLE]

FIG. 2. SKIN FRICTION AND BASE RESISTANCE VERSUS DISPLACEMENT



base resistance mobilization. The irregular skin friction curve in gravel in the depth 0-2,4 m /Fig.2, curve b/ may be occasioned by the pile surface unevenness and the change in the pile width in that segment as well as by the nature of concrete /bentonite/ gravel interface. The skin friction in gravel still increased in the region of displacement from 27 to 62 mm.

In Fig.3 is shown the portion of load tran-

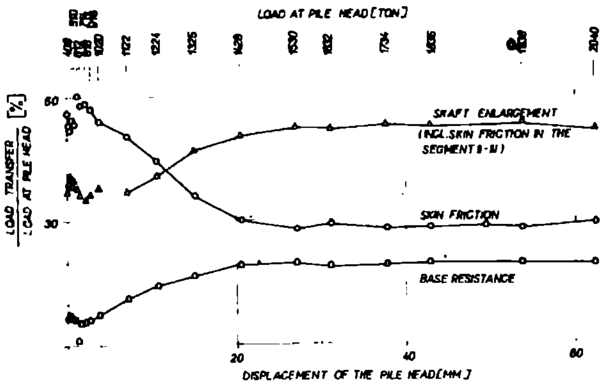


FIG.3. LOAD TRANSFER/LOAD AT PILE HEAD VERSUS DISPLACEMENT OF THE PILE HEAD

sferred by the skin, the base and by the pile shaft enlargement. The skin friction portion influences in an important way the behaviour of the pile in the region of small displacements. It decreases with increasing displacement while the portion of the shaft enlargement and the portion of the base resistance augments. The portion of load transferred by the skin, the base and by the pile shaft enlargement does not change for the displacements exceeding 20 mm.

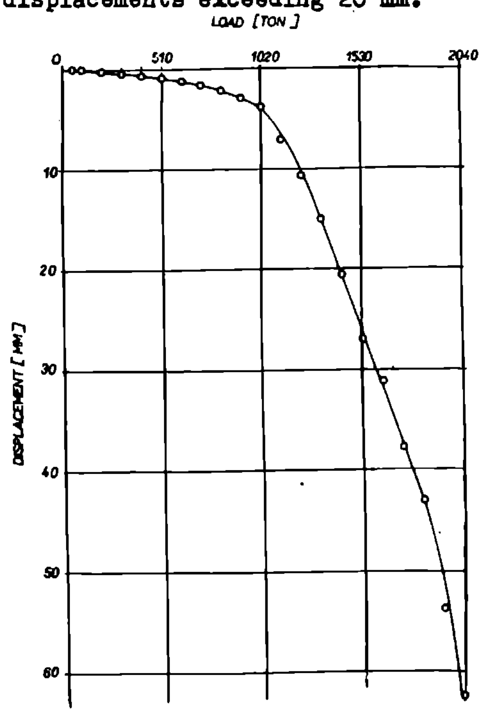


Fig.4
Load/displacement curve

On the basis of strain gauge measurements it is possible to explain the load/displacement curve /Fig.4/ as follows:

a/ the course of the first branch of the curve is affected by skin friction, mainly in clays and sands,

b/ the linking intermediate part is curved owing to the decrease of the skin friction in clays and sands to the residual value. This decrease reduces the skin friction in gravels which increases up to 10 mm displacement,

c/ the course of the obliquely bent branch of the curve is given by the load transfer by shaft enlargement as well as by base resistance. Secondarily in this part of the curve applies the skin friction in gravel while the skin friction in clays and sands after the decrease to the residual value does not change any more with displacement.

This strain gauge measurements were carried out and evaluated by the Research Institute of Civil Engineering in Bratislava, the loading test was organized by the Chair of concrete structures and bridges of the Technical University in Bratislava.

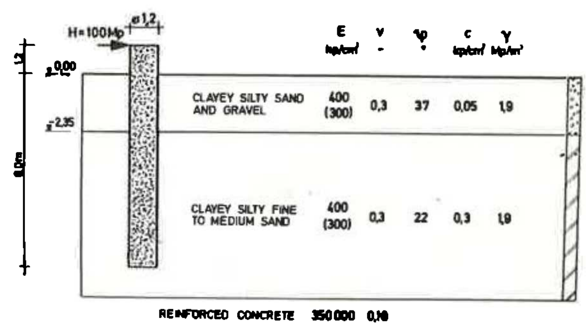
Chairman A.Kezdi.(Hungary)

Thank you very much for your interesting remarks, Mr. Černak; now, please, Prof.Wittke of Karlsruhe University.

Prof. Dr.-Ing.Wittke W. (GFR)

According to the subject 4 proposed by the General Reporter we want to present briefly the results of a large scale in situ test on a horizontally loaded large diameter bored pile and a corresponding Finite Element calculation (Wittke et al., 1973).

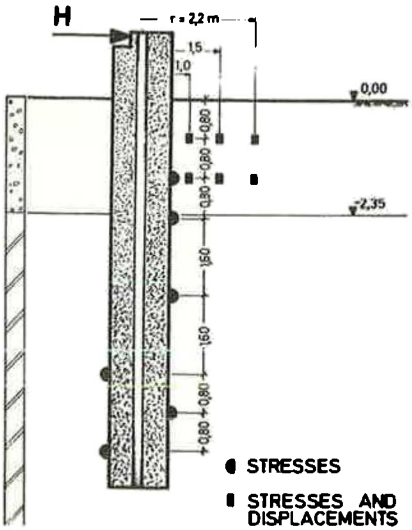
In an in situ test performed by the German Railways an 8m long reinforced concrete pile with 1,2m diameter was horizontally loaded by 100 metric tons (fig.1). The subsoil con-



sisted of an upper layer of a clayey, silty sand and gravel and a lower layer of clayey, silty fine to medium sand.

The mechanical constants of these two layers have been selected on the basis of in situ and laboratory tests as well by experience gained from other projects constructed under similar conditions. For the upper layer Young's modulus and Poisson's ratio were: $E=400$ kp/cm^2, $\nu=0,3$. For comparative calculation a reduction of Young's modulus to $E=300$ kp/cm^2 was assumed. The adopted angle of internal friction and the cohesion are: $\varphi=37^{o}$, $c=0,05$ kp/cm^2. The parameters assumed for the lower layer as well as for the concrete are also compiled in fig.1.

The instrumentation of the in situ test consisted of Gloetzl pressure cells installed along the interface between concrete and subsoil and in two horizontal sections within the upper soil layer in front of the pile. In these sections also devices for measuring the resulting displacements have been located (fig.2)

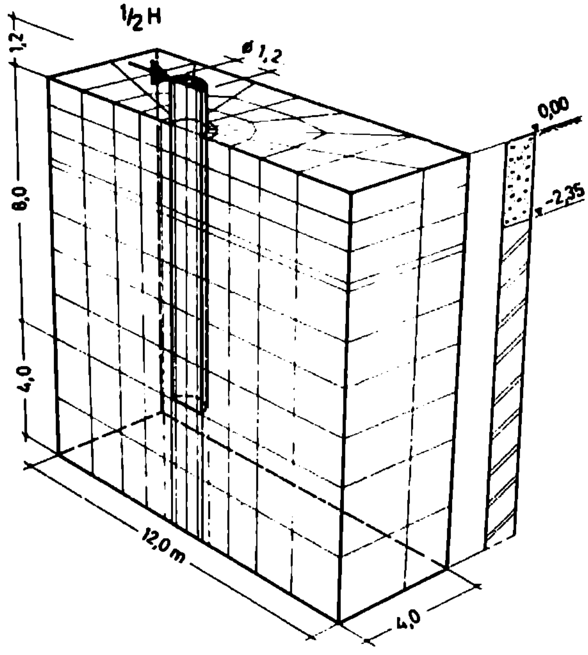


Within the Finite element calculations, due to the symmetry of the problem, only one half of the pile and the corresponding subsoil was to be investigated (fig.3). The Finite element mesh consists of 3-dimensional, eight cornered elements subdivided into tetrahedrons (fig.3). The procedure applies a bilinear relationship for the stress strain behavior of the subsoil as described in paper 45 of session 2.

The calculated pressure distribution in the vertical section of symmetry at distances of 0,2m and 1,7m from the interface is plotted in fig.4. The results of the in situ measurements in the two horizontal sections mentioned above coincide with the results of the calculations.

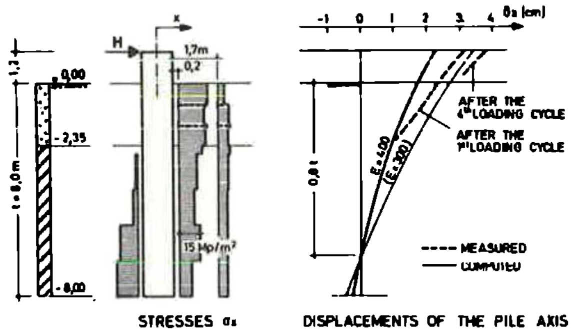
In the right hand side of fig.4 the horizontal displacements of the pile axis measured after the 1-st and the 4-th loading cycle as well as the calculated displacements (E=400 and 300 kp/cm^2/ are represented. Also here coincidence between measurement and calculations is reasonable.

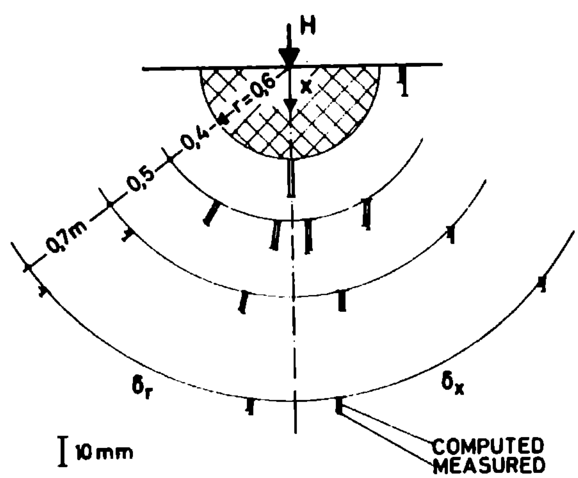
In fig.5 the measured and calculated displacements in the horizontal section at 1,6m



1056 ELEMENTS
732 NODAL POINTS
2196 UNKNOWNS

COMPUTING TIME: IBM 370/165 16 MINUTES





depth are given. The left hand side of the figure shows the radial and the right side the displacements parallel to the load H. Also here a good agreement could be achieved.

In calculation we like to state, that this method seems to reveal reasonable results at reasonable costs and after our opinion it should be applied mainly for evaluation and extrapolation of results of large scale in situ tests as well as for the design of horizontally loaded piles.

It will however be difficult in practical cases to provide reliable input data. Therefore we intend to reexamine other in situ tests applying the same procedure in order to gain more experience on the determination of input data. On this line we also may have to improve the applied bilinear stress strain relationship and especially for vertically loaded piles include the influence of volume changes of soil during shear.

REFERENCES

Wittke, W., J.Spang, W.Rodatz, S.Semprich (1973), "Bemessung von horizontal belasteten Grossbohrpfählen nach der Methode Finiter Elemente", Der Bauingenieur, in print

Chairman Prof. A.Kezdi.
Thank you Prof. Wittke. Now I would like to invite Prof. Prakash.

Prof. Shamsher Prakash (India)

1. I shall confine my remarks to the item "5" proposed by the General Reporter, i.e. on the question of additional soil tests for earthquake loading.

2. We in India, are very much concerned with this problem. In coastal regions, which are seismic too heavy structures are supported on piles. Also we know earthquakes induce random motion. Therefore, the problem is a complex one.

3. We attacked this problem from 2 directions (i). Development of analytical method to predict the response of the pile. By response we understand the displacements, strains and stresses (ii) use field data to verify this analytical method.

4. In the course of this solution, we were confronted with the question of the "relevant soil properties" and the "relevant soil pile constants".

These are at present to be determined by performing a field test on a 'working' or a "test pile", A similarity may be drawn with the load test on a pile.

5. This essentially is an in-situ test and is certainly in line with the recommendations of the main session 1.

Chairman Prof. A.Kezdi.
Thank you Prof. Prakash for your remarks.
Now Mr. Klestchev , please.

Eng. P.E.Klestchev (USSR)

As a rule experimental investigations in natural conditions are the basis of constructing rated models of beddings and foundations.

The All-Union Research Institute of Mining Geomechanics and Mine-Surveying (AURIM) with the participation of the author, has carried out extensive experimental investigations on the interaction between the beddings with the foundations and the constructions of buildings of various constructive schemes deformating under the influence of underground mine workings.

Specifically, in order to substantiate the possibility and expediency of adopting pile foundations when constructing buildings upon coal-bearing territories and to define theoretical notions and methods of estimations of pile foundations under complicated conditions of deformations of beddings in the Karaganda Coal Basin (in Karaganda) such investigations have been carried out on two experimental buildings, equipped with various measuring instruments.

Thus, in order to estimate the vertical loads from the weight of the building and their redistribution in the process of uneven settlings of the base caused by the displacement of the earth surface over the worked up space upon the head of each pile within the limits of three compartments of the buildings, 277 stringed dynamometers had been installed (Fig.1). About 10% of the piles

Fig.1. Part of installation of stringed dynamometers upon the heads of piles of the experimental houses in the Karaganda coal basin.

forming part of pile foundations of the buildings were equipped with low-ohmed tensormeters to measure the exertion in various sections of the piles and with stringed meters of contact pressure in order to determine the side pressure of the soil upon the piles. About 750 high-ohmed tensometrical plates had been installed at the ties between the steel frame works and the joint ties of wall panels and overlappings. Besides this, in order to investigate the interaction of deformations of pile foundations and the soil, a complex mine geodesic surveying observation station composed of 364 wall and 470 soil bench-marks, 850 photogrametrical mark, etc. had been set up at their base.

Following the construction, the coal-bed New K_{18} with a complete setting of the roof, has been worked up under the experimental buildings at a depth of the workings H=410m, excavated thickness of the coal vein m=1.4m and the angle of descent $\alpha=8^{o}$.

Evoked by the mine workings, the process of displacement of the earth surface on the plot of the experimental buildings is characterized by the diagrams in Fig.2.

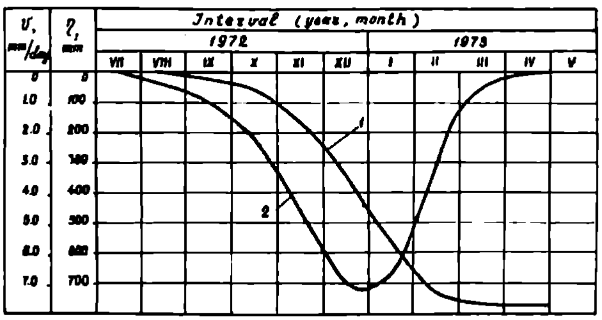


Fig.2. The development of settings of foundation (1) and the rate of their intensification (2) on the plot of the experimental houses in the process of their working up

In the process of the construction and the subsequent working up of the houses, the magnitude of vertical and horizontal deformations of the buildings and the soil at their bases, exertions on various sections of the piles, the forces of side pressure of the soil upon the piles, longitudinal load upon the piles, exertions in steel frame works ties, ferro-concrete belts and joint ties between separate constructions of the buildings, generalization of the deformation of the elevated parts of the buildings within the limits of each storey, etc. were measured.

The information obtained, allowed us to disclose the character of work of pile foundations above mine workings, consisting of that equally with the longitudinal pressing loads from the weight of the building and the horizontal loads, applied at the corners of the piles, the latter experience simultaneously the exertion of additional longitudinal loads resulting from the crooking of bases and the exertions arising from the curving of the piles under the influence of the side pressure of the displacing soil. The physico-mechanical and deformatory characteristics of soil foundations in the process of working up of the territory do not remain constant, exerting vital influence upon the tensed state of the soil massive around the piles and the carrying capacity of the piles.

On the basis of analytical and experimental investigations carried out, it became possible to construct a calculated model of the system of "Pile foundation-deformed under the influence of working up of the base", in accordance with which the horizontal strain on the pile can be determined in conformity with the well-known determination by Schleicher, applied to a quarter of linear deformated space, possessing a high pliability, under side cutting into it of the pile.

At the same time the experimental data obtained permits us to consider that the piles are in their way a sort of shock-absorbers when transmitting strain coercion from deformed base upon the building, due to which the damage of the latter from horizontal deformations and uneven settlings of beddings sharply diminishes. The latter indicates the indisputable advantages of pile foundations when building under specific soil conditions and on territories above mine workings.

It should be further noted, that the construction of buildings of foundations, deformed by underground mine workings, is an important and pressing problem of today in many countries and it would be expedient to discuss the state of this problem at the next in turn congress on the mechanics of the soils and the foundation building.

Chairman Prof.A.Kezdi
Thank you very much Mr.Klestchev; Prof. Tassios, please.

Prof. Th.Tassios (Greece)

Following the suggestions of our General Reporter of this theme (§ 5 of his report), the present discussion comprises additional information concerning the design load and deformational behaviour of vertical piles, subjected to horizontal cyclic loading.

Twenty-four piles have horizontally tested, under various conditions, in fullscale.

The piles (Frankipiles, 52 cm in diameter), were driven (down to a depth of 10,00 m), through soft silty and clayey material, having an average coefficient of horizontal reaction of the order of 0,3 kg/cm^3.

On the basis of results of horizontal loading under various test-rates (Fig.1), a considerable increase (of the order of 300%) of bearing capacity of the pile is expected, for a rate corresponding to an earthquake.

In fact, by extrapolating to a value of 10^5 t/h approximately, one could find an increase of at least 300%. Nevertheless, cyclic loading results in a reduction of the critical value of load (i.e. the strength) of the pile, of the order of magnitude of 50% in the case of our investigation.

Consequently, it remains a clear increase

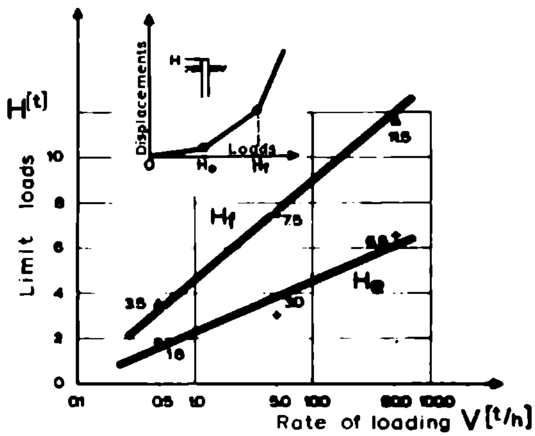


Fig.1: Limit loads of horizontal loading of vertical piles, as a function of the rate of loading:

$H_e^{[t]} = 2{,}3\,[1+\log V^{[t/h]}]$, $H_f^{[t]} = 4{,}4\,[1+\log V^{[t/h]}]$

of strength of about 150%, which, (taking into account a factor of safety equal to 3/4 of its usual value), corresponds to a permissible horizontal load twice as high as the conventional static load.

Anyhow, further reductions of this allowable load are expected, due to the vibrational shearing of alluvia and the strength loosening of them, under earthquake loading induced from the bedrock up to the alluvia themselves. From this aspect, the remarkably complete method of seismic response of piles presented by Prakash (paper 3,31) at this Congress, should possibly be modified accordingly, in order to take into account these relative movements of alluvia layers.

Finally, related to the necessary dynamic computations, an average damping ratio equal to 4% have been found for various levels of horizontal, fully reversible, loading (Fig.2).

The order of magnitude of this damping, equals approximately half the values found by Agarwal (paper 3,2) and diagrams offered by Oteo (paper 3,28) at this Congress. This is probably due to the fact that, in our case, only extreme values of deformation were noted, underestimating the real area of each loop.

Obviously, this is a relatively new branch of the profession. Consequently, compilation of further dield data related to the horizontal repeatitive loading of piles will be needed in the near future.

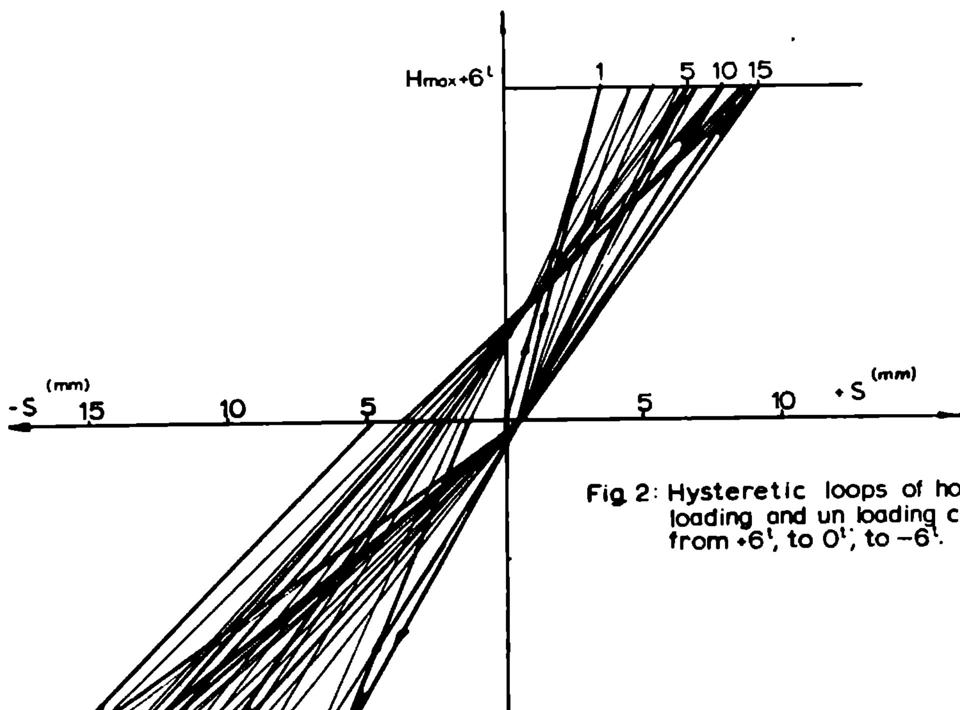


Fig. 2: Hysteretic loops of horizontal loading and un loading cycles from +6t, to 0t, to -6t.

Chairman Prof. A.Kezdi.
Thank you very much Prof.Tassios for your interesting contribution. The next contribution will be in charge of Prof.Dinesh Mohan.

Prof.Dinesh Mohan (India)

I have heard with great interest the remarks of the General Reporter and I have also benefitted greatly from the discussions that have taken place on the subject of deep foundations.

In spite of various advancements made in the field of pile foundations over the last four years we have still to find a suitable answer to two main problems (1) the settlement of pile foundations in cohesionless soils, and (2) bearing capacity of piles from simple field or laboratory tests. Cumbersome pile load test is still required,and here again it is doubtful if load on a single pile predicts the behaviour of a pile group which support most of the heavy structures.

I will also like to take this opportunity to briefly mention our work in India on multi-under-reamed piles which have been a development of single under-reamed piles originally designed by us for foundations in Indian black cotton soils, which, as some of you know, are very expansive in nature. These piles are suitable for heavy structures,both for sandy and clayey soils,and are very much more economical than uniform large diameter bored piles. They have also been successfully used in foundations for electric transmission line towers, microwave antenna towers. Recently we have used them in a sattelite tracking tower foundation and also in hydraulic structures as anchor piles for a floor of a dry dock.

A few months back we carried out a load test on a multi-under-reamed pile in clay-45 cm shaft diameter,7,6 m long,with four bulbs of 112 cms. The reaction was also obtained through a framework anchored down to 12 double under-reamed batter piles. The pile took a load of 300 tons and had not reached ultimate when anchors yielded. This was against an ultimate load of 320 tons worked out from soil properties.

Under-reamed piles have thus come to be used in a very large manner in India and a very large economy has been effected in foundations with their use. I will be glad to furnish full details of construction and design to any one who is interested.

Thanking you,Mr.Chairman,for giving me the floor.

Chairman Prof.A.Kezdi.
Thank you very much Prof.Mohan for your interesting contribution. The next contribution will be in charge of Mr.Bartolomey of the USSR.

A.A.Bartolomey (USSR)

Dear Chairman,ladies and gentlemen, colleagues!

It should be noted that general speaker Professor Zeevaert's conclusion that the further rising of the efficiency of deep foundations is possible due to the calculation of foundations on bearing capacity but not on permissible deformation of construction is of an esceptional importance. The detailed complex field test results of piles soil interaction and settlements of different pile foundations. Accomplished by us showed that when determining bearing capacity of pile foundations according to the ultimate permissible deformation of constructions and buildings, loads may be increased by 20-50% but in some cases more than twice if grillage contacts with soil. (Bartolomey 1971,1972a).

Reliable data are needed on settlements and their time dependence when designing pile foundations on permissible deformations of buildings and constructions.

On the grounds of experimental designing of more than 20 buildings taking into account the ultimate allowable settlement, settlement observation, the deformation of these buildings,it is established that the closest results between the predicted and actual settlements of buildings are given by the methods according to the load depth and the shape of its transmission along the lateral surface and in the plane of the piles, changing of soil characteristics in the result of the sinking of piles (Bartolomey, 1972b).

Fig.1 shows the results of settlement observation of a ninestory building,the type 1-P-4470-25/65 and gives comparison of time dependence of designed and real settlements. The foundations were designed according to the ultimate allowable settlements of a building. It gave possibility to cut piles twice. The allowable design load on piles is 80 tons. The basic load on piles according to the data of measurements with the help of tensometric hoops happened to be of about 64 tons. When designing pile foundation on the ultimate deformations the establishment of modulus of deformation of soil in active zone is of great importance. Investigations accomplished by B.I.Dalmatov (1968), N.M-Doroshkevitch and V.V.Znamenskey (1972) demonstrated that Bussinesco-Shleyhor formula is of no use for determination of the modulus of deformation under the pile foundations as this formula does not take into account the depth of load and the character of its transmission to the soil by piles.On the basis of analytical solving of space and plane problems and on the basis of using the results of the static tests of piles,we suggest the calculation formula of modulus of deformation of soil under the pile foundations according to the character of the transmission of the load along the lateral surface of the piles and in the plane of the tip of the pile and the phisico-chemical changings of the properties of soils in the re-

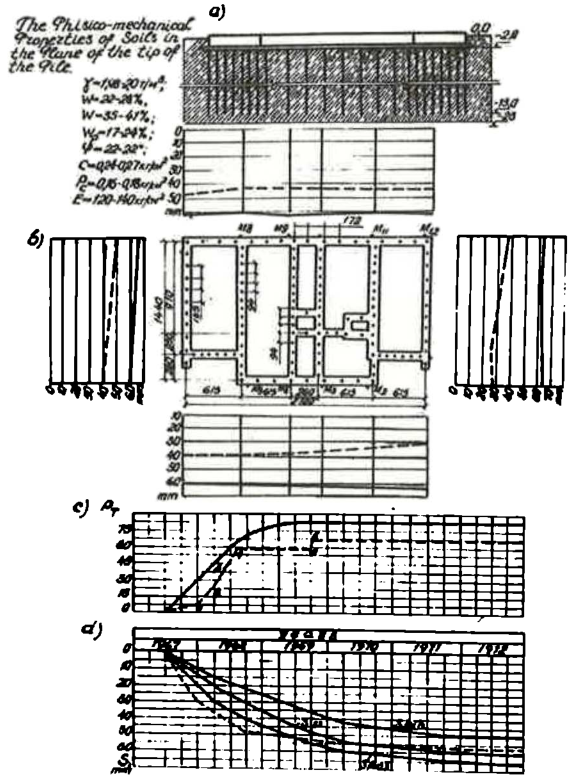


Fig.1. Results Settlement Observations of nine-story building, the type 1-P-447C-25/65. a) vertical section; b) plan of the foundation and the settlement of sings - - - settlements for the period of construction, ______ the same for the period of observation; c) the chart of an average rate of the load on pile _____ designed load, - - - actual load on pile according to the data of measurements with the help of tensometric hoops; d) the chart of settlement time dependence ______ maximum, average and minimum settlement of singns, - - - time dependence of the designed settlement of foundation

sults of the sinking piles and the increase of settlements for a definite period of time (Bartolomey,1972).

Now some words about the transitional coefficiences as the result of static and dynamic sounding to the bearing capacity of piles. From our point of view it is better to use the results of group sounding or to use when examining data of individual tests coefficiences of mutual influence of piles according to the length and distance between them as the data of individual tests and sounding do not reflect the real work of the piles in the system of foundations.

The working out of more effective and well-grounded methods of designing of the pile foundations on the ultimate permissible deformations of buildings and constructions demands reliable methods of determination of the phisico-chemical propertie of soils in the active zone, the solving problems on soil mechanics for the calculation of settlements of pile foundation taking into account non-linear dependence between stresses and deformation of soil medium, soil creep, filter fenomena, compression of void water, the initial water table gradient, the transmition of the load along the lateral surface and in the plane of the tip of the pile. It gave possibility to cut down expenses when designing pile foundations.

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Chairman Prof. A. Kezdi.
Thank you Mr. Bartolomey.
In closing this session I want to call on Prof. Zeevaert to make concluding remarks.

Prof. Zeevaert L. (Mexico)

Ladies and Gentlemen!

Concerning concluding remarks on the topics for discussion, I may say that our colleagues today, have submitted valuable contributions to Session 3 regardless of the short time given for their reports on the accepted topics proposed in my General Report.

I think, that in spite of the excellent investigations reported on negative skin friction using coatings of different materials, it is still necessary to classify the results of tests performed to fined in the laboratory the soil-coating friction coefficients and techniques, for different types of coatings. It is also important to investigate the time element and driving effects for different coating products that may be used economically. Negative friction forces may reach high proportions if the coating fails to work as expected.

There are still several uncertainties in the use of bearing capacity factors in deep foundations when the pile or pier base is supported on soil showing compressibility. The compressibility under the pile tip has a two fold action: first, to reduce the angle of internal friction and, second, to shorten the length of the potential surface of rupture under the pile point. Thus, the bearing capacity factors may be considerably reduced.

The cone standard penetration tests results to determine point bearing and lateral skin friction, are proposed to be affected by correlating factors for different soils. However further investigations are necessary to to establish fixed values. It is my personal impression that this procedure has good possibilities for areas with well defined subsoil conditions. As one may visualize, the correcting factors may be different for the same relative density soils having different confining conditions, and grain strength and shape, as volcanic glasses, high content mica soils or coral sands, etc.

The method of construction of open caissons and trench walls using thyxotropic clay jackets offers a very valuable tool for the solution of foundation problems, from compensated foundations in deep compressible soil deposits, to those encountered when the bearing stratum is not deep seated. These methods may provide economical solutions in specific cases. They should be better known and encouraged.

Finally, it appears that the only means in the present to solve problems connected with environmental forces, is to learn as precise as possible, on the static and dynamical mechanical properties of the subsoil materials for the different soil strata encountered affecting the foundation design.

One the problem is diagnosed, and soil mechanical properties known, theoretical working assumptions may be established compatible with the statics and deformational behavior of the problem in question.

I wish to end stating that future field tests and measurements and their correlation with calculations, based on working hypothesis using well determined mechanical soil parameters, as discussed in Session 1, will bring us closer to a better understanding in the economical and practical design of deep foundations. I thank you very much.

Written contributions

Prof. Tcheng Y. C.E.B.T.P., Paris, France
Eng. Bolle G., S.P.I.E.Batignolle, C.I.T.R.A.

FONDATIONS PROFONDES Y COMPRIS FONDATIONS SUR PIEUX

Notre intervention porte sur un procede particulier de fondation profonde. Il s'agit des tubes creux mis en oeuvre par battage sans obturation a leur base.

Dans la premiere partie nous presentons les resultats d'essais en semi-grandeur dans une Station d'Essais, et ensuite, nous citerons les travaux realises a l'aide de ce procede.

a) Experience:

Nous avons rempli soigneusement a l'aide de sable une cuve de 6.40 m de diametre et de 10,40m de profondeur.

La compacite du sable etait rigoureusement constante; elle etait egale a 1.41 10^4 N/m^3

Nous avons introduit dans ce milieu un certain nombre de tubes ouverts dont les ca-

racteristiques sont les suivantes:
Diametre exterieur en cm:

26,7; 21,6; 10; 6; 4.

Diametre interieur en cm:

25,4; 16,4; 6,8; 3,93; 1,96.

Sur le graphique ci-joint nous pouvons constater que,avec les dimensions de tubes utilisees et dans les conditions particulieres de l'experience le pouvoir portant d'un pieu creux est absolument identique a celui d'un pieu de meme diametre muni d'une pointe fermee a sa base. Cette observation n'es evidemment valable que pour les dimensions des pieux utilises. On constate qu'au fur et a mesure du fonçage des pieux il se forme un bouchon a l'interieur de ces derniers. La hauteur de ce bouchon croit d'abord lineairement a partir de la surface jusqu'a une certaine profondeur au-dela de laquelle elle reste constante.Ce bouchon compense donc efficacement l'absence de la pointe.

b) <u>Travaux executes</u>:

Les pieux tubulaires utilises pour la construction d'ouvrages,en general maritimes,sont rattaches a deux types principaux:

1°- tubes metalliques
2°- tubes en beton precontraint,prefabriques par elements.

Les chantiers executes avec le premier type de pieux sont nombreux et nous ne les detaillerons pas.

Les principaux chantiers executes par notre Societe,avec le second type de pieux sont les suivants:

- Quai a TANDJUNG PRIOK (Port de Jakarta) -Indonesie-et
-Quai a SEMARANG (Indonesie) ou le terrain est constitue par 24m de vase surmontant une argile plus consistante.
-Quai a BANJERMASIN (Indonesie) ou l'on rencontre une couche de sable suffisamment epaisse vers 32m de profondeur sous la vase.

Pour ces trois ouvrages les pieux sont des tubes de diametre 70/90cm battus.

-Quai mineralier a SETE (France)

Cet ouvrage est fonde sur des pieux tubes de 80/110cm de diametre et 27m de long battus avec un marteau D 44 vers 26m de profondeur. Ils sont ancres d'environ 6m dans des marnes calcaires.

Dans le cas des ouvrages de SEMARANG et surtout de SETE, la force portante des pieux tubulaires,comparee a celle d'un pieu plein equivalent a ete etudiee theoriquement puis controlée par des essais de charge en grandeur, sur le chantier. A SETE, les charges appliquees ont atteint 460 tonnes.Nous avons constate que la hauteur du bouchon reste constante a partir d'une hauter de 15 a 16 fois le diametre interieur du tube; la force portante est alors equivalente a celle d'un pieu plein (voir Annales I.T.B.T.P.n° 290-Fevrier 1972-BOLIE et CASSAN.).

D'autre part,des essais faits a SEMARANG avec des pieux de meme diametre a pointe ouverte et avec sabot conique ont confirme la plus grande facilite de mise en place des premiers.

<u>CONCLUSION</u>:

Les experiences en laboratoire et les observations faites lors des travaux sur chantier ont fourni des resultats tout a fait comparables. Les pieux creux possedent pratiquement le meme pouvoir portant que celui des pieux pleins pour les dimensions etudiess, sous reserve que le bouchon interieur soit stabilise lors de la mise en oeuvre.

Ce procede est appele a un avenir prometteur car il presente des avantages du point de vue economique et de la facilite de mise en oeuvre.