MAIN SESSION 2
2-e SESSION PLENIERE
ВТОРОЕ ПЛЕНАРНОЕ ЗАСЕДАНИЕ

INTERACTION OF SOIL BASES AND STRUCTURES (PREDICTION OF SETTLEMENT, DESIGN OF MASSIVE FOUNDATIONS BASED ON THE LIMITING STATE, DESIGN OF FLEXIBLE FOUNDATION BEAMS AND SLABS)

Chairman: Prof. Edward de Beer (Belgium); General Reporters: Prof. M.I.Gorbunov-Possadov and Prof. S.S.Davidov (USSR)

Participants: P.A.Rochette (U.K.), H.Sommer (W.Germany), A.Vesic (USA) E.A.Palatnikov (USSR), E.Toshkoff (Bulgaria), G.Lasebnik (USSR), Yu.K.Zaretsky (USSR), S.Hansbo (Sweden), G.K.Klein (USSR), B.G.Korenev (USSR), H.Ohta (Japan), J.P.Giroud (France), D.Krsmanovic (Yugoslavia)

Chairman Prof. Edward de Beer (Belgium)

At the opening of this session, it is my first agreeable duty to introduce to you those which are on the rostrum with me:

the vice chairman of this session, Professor Dusan Krsmanovic, Professor at the University of Sarajevo, and which is universally known for his outstanding contributions covering soil-structure interaction, not only in the field of soil mechanics, but also in the field of rock mechanics.

the general reporters Professor Gorbounov-Possadov and Professor Davydov, which however I will introduce to you in more detail somewhat later.

the scientific secretary, Mr. Fedorovsky, research engineer at the Research Institute of Foundations in Moscow, and disciple of Professor Gorbounov-Possadov

the representative of the scientific committee Mr. Mikheev, Science Director at the Research Institute of Bases and Underground Structures, Gosstroy USSR.

It should have been our privilege that Dr. Bjerrum, past president of our International Society should have given his comments concerning the subject of this session. In all sessions we are aware which great loss our International Society has undergone by its premature death.

In this session 46 papers have been introduced, and 24 written discussions were received. Because of the limitation of time only 12 contributions have been retained, which have been chosen by the general reporters in accordance with the 5 topics they proposed for discussion.

The names of the participants chosen for discussion are written on the black-board. Each participant will have a maximum time of 5 minutes. After these 5 minutes the microphone will automatically be switched out.

It is really a great honour for me to have been invited by the Russian Organizing Committee to take the chair for Main Session 2, devoted to the problem of the Interaction of soil bases and structures. It can be stated that this is certainly one of the most difficult problems in the field of Soil Mechanics and Foundation Engineering, as its correct solution implies a very thorough knowledge of the mechanical properties, not only of the soil layers involved in the problem, but also of those of the building materials.

The availability of electronic computers makes it possible to solve problems, which some years ago could practically not be treated. One of these problems is the calculation of the pressure distribution underneath rafts and beams, taking into account the rigidity of the soil and the superstructure.

In order to get a correct answer from the computer it is however ssential to be able to programme the exact geometrical data, and the exact mechanical properties of the construction materials, and of the different soil layers involved.

All these factors are very complicated as indicated on the scheme of fig.1

Concerning the soil a distinction has to be made between immediate settlements, hydrodynamic settlements, and secondary or secular settlements (creep phenomena). The properties of the building materials are also time-dependent (deformability under dynamic loading, permanent loading, shrinkage and creep; variability with the age).

Concerning the soils it must be stated that they are non-linearly deformable media and that in some parts of the medium plastic regions can be formed. The complexity of the deformability properties of the soil can be illustrated by fig.2, taken from a contribution of Professor Kerisel, and showing that the modulus of deformability is a function of the spherical stress and of the deviatoric stress.

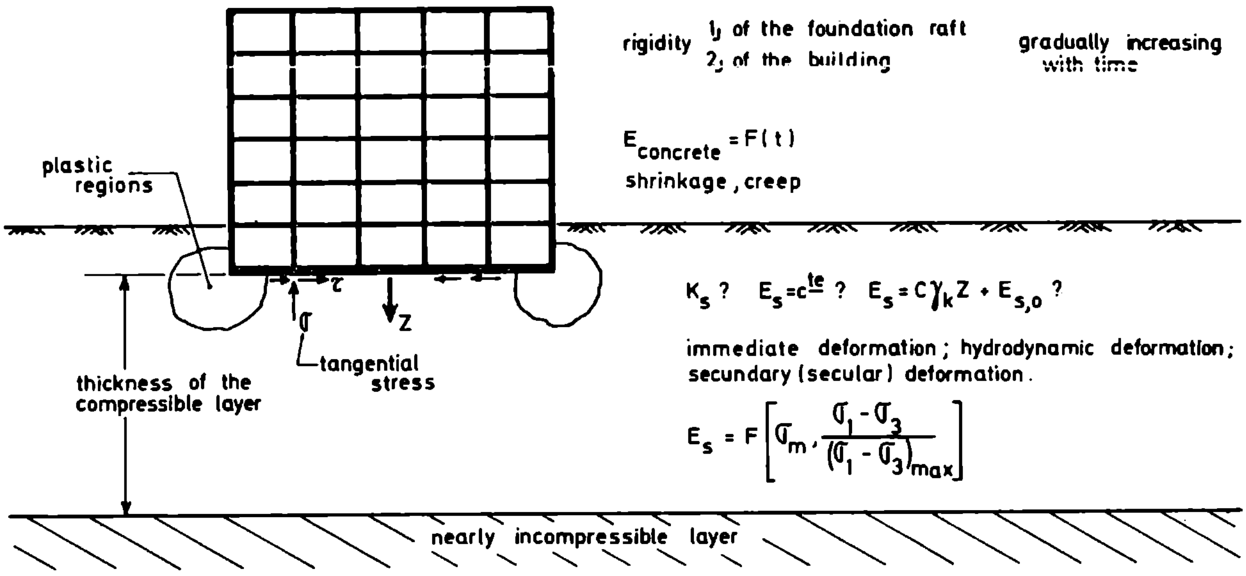
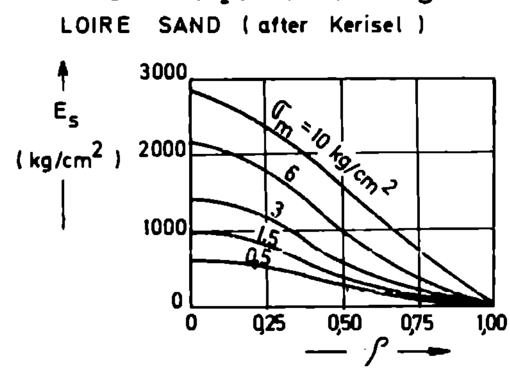


FIG. 1

In the calculation of the pressure distribution in the contact face, generally the tangential stress components are neglected.



$$\sigma_m = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$$

$$\rho = \frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_{BR}}$$

FIG. 2

The rigidity of the building gradually increases with time both during and after erection, and also sôme plastic properties of the building materials have to be considered.

The consequence of these intricate factors is that, even now, very important simplifications have to be introduced in order to solve the problem.

For solving the problem of the pressure distribution, Terzaghi and Peck used the old method of the modulus of subgrade reaction, but gave an indication how to choose the value of this modulus starting from some mechanical properties of the soil and the geometrical data. Over the years several scientists have tried to improve the method of the modulus of subgrade reaction, which is still largely used in practice.

Another approach was to consider the soil as a medium with a constant modulus of deformability and a constant Poisson's ratio.

Others have assumed the soil to be a material in which the modulus of deformability linearly increases with depth. The solution found in this way lies somewhere between that of the modulus of subgrade reaction, and that of the constant modulus of deformability.

In recent times Repnikov, followed by Schultze, has presented a combination of both simplified models: the real medium is replaced by a continuum with a constant modulus of deformability, containing a system of springs, with a constant modulus of subgrade reaction. This combination of course also gives results located between the two extremes.

It is impossible to enumerate here the names of all the scientists who have made substantial contributions to the problem of pressure distribution. We must though mention, besides the two very eminent general reporters Professor Gorbounov-Possadov and Professor Davydov, the work of Ohde-Kany, Grasshoff and Professor Schultze in Germany, Professor Krsmanovic in Yugoslavia, Professor Vesic in the USA, a group of scientists of the German Democratic Republic, and our School in Ghent.

In recent times two very interesting symposia have been held concerning this subject; one in Sarajevo in 1969 under the impulse of Professor Krsmanovic, organized by the Academy of Sciences of Bosnia-Hercegovina; the other in Dresden in 1972, organized by the Academy of Sciences of the German Democratic Republic.

Both symposia can certainly be considered as very valuable introductions to this main session of the Moscow Conference.

Although the results obtained from the methods of the modulus of subgrade reaction, and the constant modulus of deformability can be quite different, as is for instance illustrated by the fig.3; it must be stressed that

$\ell = 10{,}00$ m $b = 1{,}00$ m $d = 0{,}50$ m

$I = \dfrac{b\,d^3}{12} = 0{,}0104\ \mathrm{m}^4$ $E_b = 210.000\ \mathrm{kg/cm^2}$

$\dfrac{E_s}{E_b}\left(\dfrac{\ell}{d}\right)^3 = 100\,\mathrm{t}$ $E_s = 2625\ \mathrm{kg/cm^2}$

Total force on the beam = 100t

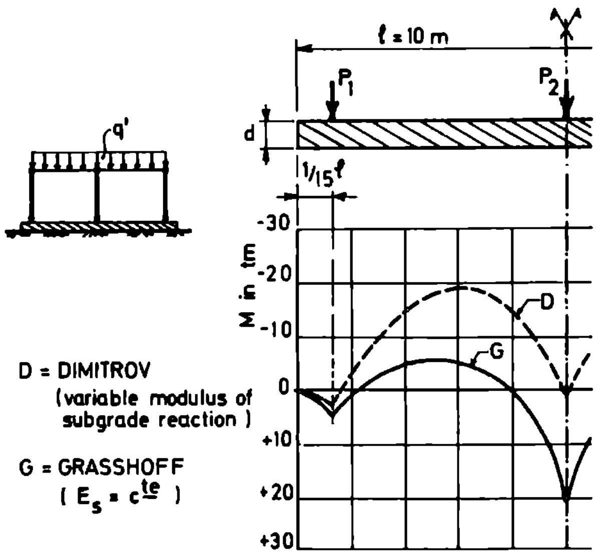


FIG. 3

until now practically no accidents or damage have been reported for buildings calculated by one of these two methods. Therefore it must be concluded that in some way or other there are some hidden margins of safety.

For reason of economy it therefore becomes of the utmost importance to be able to surround better the problem and, now the difficulty of calculation can be overcome thanks to computers, the principal objective must be to better know the real properties of all materials involved and to be able to introduce them correctly into the calculations. In order to get this knowledge it becomes imperative to observe and measure, in a still more scientific way, the behaviour of buildings during and after their construction.

It is a great pleasure for me to introduce to you the general reporters Professor Gorbounov-Possadov and Professor Davydov.

Besides many original contributions concerning other important subjects in the field of Soil Mechanics, Professor Gorbounov-Possadov has shown for several years a continued interest in the problem of the interaction of buildings and soils, and contributed very interesting publications on the subject. Professor Davydov is a world famous specialist in the field of the calculation of structures.

Because of the language barrier we do not always have the opportunity to follow the eminent contributions of the Russian scientists, and therefore we are looking forward with the utmost interest to the general reports of Professor Gorbounov-Possadov and Professor Davydov.

May I now call on Professor Gorbounov-Possadov to present his general report.

INTERACTION OF SOIL BASES AND STRUCTURES

(Prediction of Settlement, Design of Massive Foundations Based on the Limiting State, Design of Flexible Foundation Beams and Slabs)

Prof. M.I. Gorbunov-Possadov (USSR)

The importance of the problem of interaction of bases and foundations requires no comment. This is the final link in the chain of the studies necessary to provide the proper work of bases warranting the pre-assigned strength and maintainability of a building or structure. The problem can be solved exclusively by combined utilization of all achievements in the field of designing foundation settlements, beams and slabs on an elastic base, and by applying progressive methods for designing reinforced-concrete structures. Many of these problems however, have been almost completely ignored in the general reports at the previous conferences.

The discussion of the indicated problems at a separate session is particularly timely because the number of investigations in this field has considerably increased in recent years. This is due to a steady growth of loads transmitted to the soil coupled with the need for weak and non-uniform bases.

In designing bases, Soviet scientists have for many years proceeded from the assumption that in most cases buildings and structures are threatened by non-uniform settlement of the base rather than by the loss of bearing capacity. This idea is finding ever-increasing support in the engineering circles of other countries. Rapid development of computing techniques has opened up new vistas for solving problems which only recently seemed insoluble. Among these are the design of frame buildings taking into account the non-uniform settlement of supports resting either on separate foundations or on a continuous slab. Of great help was the development of the finite-element method, which enables solving base problems with extremely complex boundary conditions, not only within the framework of the theory of elasticity, but also within the framework of more complex continuum theories, which are more and more often used in representing the actual complex properties of soil bases.

At the same time, despite the great amount of work done, the problem of a soil base model ensuring sufficient agreement between calculation results and actual conditions has not yet been brought to a complete and successful solution. The previous simplified models the Winkler hypothesis and the homogeneous elastic weightless half-space — are now considered inadequate in many cases. They are supplanted by solutions based on models of a compressible layer or elastic half-space with the deformation modulus increasing with depth. Also, interesting investigations on the solution of the problem in the non-linear version have been launched.

I. Settlement Prediction

As a rule, when the time factor is ignored, final settlement is now calculated by methods of the theory of a linearly deformable medium (the theory of elasticity). Considerable progress toward simplication of calculations in this field has been made in recent years. For instance, Harrison, Gerard and Wordle have compiled an extensive set of programs for determining stresses and displacements in a half-plane and a half-space. The load is assumed to vary widely.

Giroud and Wadowič have elaborated tables and graphs for finding displacements and stresses in a homogeneous half-space under a load distributed linearly over a rectangular area.

In his paper submitted to the Conference Milovič describes the results of similar research carried out for a strip load and for a load over a circular area on a compressible layer.

Absy has established tilts of a rigid rectangular settlement plate on an elastic half-space by the finite-element method. The magnitudes of these tilts practically coincide with those incorporated in the USSR State Standards.

For the compressible-layer model, Giroud has compiled extremely detailed tables for determining settlements of rectangular rigid foundations with a rough footing.

Schultze and Sheriff have submitted to the Conference the results of settlement observations for 48 civil and industrial buildings. These authors conclude that engineers should no longer calculate settlements on sand by the methods applied to clays which used the results of soil tests with triaxial devices. The best results are obtained by calculation methods based on static and dynamic sounding.

In the case of protracted continuous foundation slabs, theory-of-elasticity settlements are especially grossly over-predicted. According to observations, such as those carried out by T.A. Malikova, for a wider (10 to 15m and up) slab, the pressures being the same, the settlement is practically independent of the slab width, whereas it should be proportional to the latter according to the theory of a linearly deformable medium. It can thus be considered an established fact that the familiar plot relating the settlement to the width of a square plate should be continued so that the settlement curve will asymptotically approach a horizontal line as the width increases (Fig. 1).

The divergence between the theory of a linearly deformable medium and the actual conditions is caused by the fact that with large surface areas deep-lying layers come into play which remain practically underformed under an external load.

The following hypotheses have been advanced to explain the low deformability of deep-lying layers as compared with the theory of elasticity.

1. Some scientists assume that the base is deformed only when the structural strength of the soil is affected, and at great depths the additional stresses from an external load are too low for this. Stefanoff and Krastilov have submitted to the Conference

On the basis of the finite-element method and the bilinear law for deformations, Malina has solved the problem of a rigid strip foundation on a sandy base. Each element was assumed to be elastic until the stresses in it reached limiting stresses after Mohr-Coulomb. Then the excess was transmitted to the adjacent elements by the iteration process until the stresses proved to be physically feasible everywhere.

We stated the problem in the same way. But we used for this purpose a fictitious load consisting of three-dimensional force couples applied to the plastic region, as described in our communication to this Conference.

II. Design of Beams and Slabs on Compressible Base

In this field, scientists continue their search for mechanical soil models which would best reflect the actual conditions. It is known that design by the Winkler method does not yield satisfactory results since it neglects the distributive ability of a soil base. Conversely, the elastic-half-space modulus grossly overpredicts this ability, especially in the case of the plane problem. Besides, the elastic half-space leads to stress concentration near the beam and slab edges. Because of these features, the positive bending moments (Fig.3) by the elastic-half-space method (curve II) greatly exceed the Winkler moments (curve I).

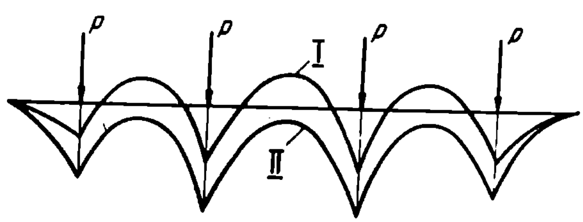


Fig.3. Schematic diagram of bending moments in foundation slab or beam calculated by: 1-Winkler hypothesis, 2-elastic half-space hypothesis.

After Winkler, the negative moments in the spans are approximately equal to the positive ones beneath the supports. This is not true, since measurements show that, as a rule, foundation beams and slabs buckle downwards. The true bending-moment plot must pass between curves I and II.

One of the widespread ways to find a compromise between the two theories is the rigidity coefficient method, i.e.the method of variable modulus of subgrade reaction so selected that it roughly corresponds to the calculated plot of soil surface settlement under a uniform load (Grasszhoff, Klepikov, Rivkin).

In the F.R.G., great popularity is enjoyed by the Kany method in which the bearing area of a structure is divided into a number of elements. To determine the settlement of a soil element from a load distributed over it, the usual Schleicher formula is applied, while the effect of the same load on the neighbouring elements is established with the aid of the author's empirical formula.

A calculation method using a scheme of a compressible base in the form of a finite-thickness layer (Davydov, Egorov, Samarin, Krasheninnikova) is widely applied. On the basis of the finite-element method, Milŏvič, Touzot and Tournier have given the numerical values of the stress components in a finite-thickness compressible layer completely adhering to an underlying absolutely rigid base when a rigid rectangle is loaded with an inclined, eccentrically applied force. In his paper submitted to our Conference, Milovič determined displacements and stresses in a layer under a load uniformly distributed over an annulus.

Of great importance in calculations, especially in plane conditions, is taking into consideration plastic deformations in the soil beneath the foundation edges. Experimental determinations of the distribution of reactive pressures have been carried out by many Soviet authors, including Evdokimov, Shiryayev, Lipovetskaya, Murzenko, Lazebnik, Krivorotov and others. A survey of tests conducted in other countries was made by Schultze as far back as 1961. Here, principal attention should be given to experiments in actual conditions. The effect of the rigidity of the superstructure, however, sometimes hinders determination of the actual load transmitted by the supports.

Interesting investigations on the distribution of reactive pressures in actual conditions have been conducted by Zeiffert, particularly in the case of foundation slabs for silo blocks (the obtained plots for a base of loess-like clay are saddleshaped).

The general state of the art of designing structures on an elastic base is vividly reflected in the transactions of the 1969 International Symposium convened in Sarajevo on the initiative of Professor Krsmanovič.

Nonveiller's paper submitted to this Conference makes it possible to account for many different factors in designing beams on a compressible base, such as elastic compression and the initial swelling coefficient of the soil. To this end, curves for soil compression under pressures are plotted under separate cross-sections of the beam. The Boussinesq rule or the anisotropic theory is used to establish the law of transmission of an external load.

A method for designing circular slabs lying on a Winkler base subject to a horizontal (thermal) load was proposed by B.I.Korenev (1971).

The problem of interaction of the foundation structure and the superstructure is of exceptional importance in defining the precise stress distribution both in the foundation and in the structure itself. In strip and continuous slab foundations on an elastic half-space or a compressible layer the calculated positive moments are then considerably reduced, since the rigidity of the superstructure increases the loads on the edge columns and reduces those on the middle ones. Here we will mention the works of Meyerhof and Hametsky for frame structures with

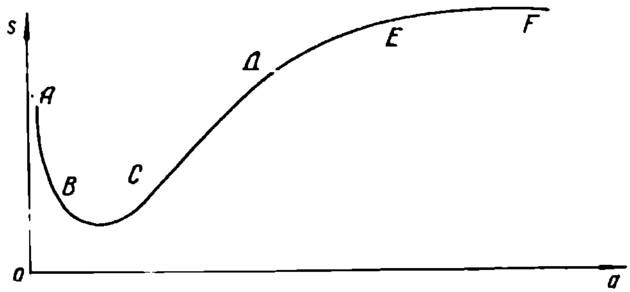


Fig.1. Schematic diagram for dependence of settlement on width of square test plate at constant mean pressure, e.g. 2 kg/cm² AB- section of highly developed plastic deformations at a 30cm; CD-section of deformations corresponding to scheme of linearly deformed medium at a from 70cm to 3-5m, EF-section asymptotically approaching horizontal straight line at a 10m.

tables and graphs for determining the corresponding compressible layer thickness.

2. Other researchers attribute this to the non-linear strain-stress relationship in soils. The paper of Kriegel and Wisner submitted to our Conference explains precisely in this way the results of their measurements of soil displacements at different depths under all buildings, silos and chimneys. They have established a law for non-linear deformations which, in their opinion, explains the results obtained.

3. Klein, Fischer and some other authors assume that the deformation modulus increases with depth, even in homogeneous soils. Experiments with plates in test holes carried out by Schultze in 1965. However, failed to confirm a sufficiently rapid increase in the deformation modulus. Therefore the use of the increasing-modulus scheme can only be justified by the possibility of obtaining close enough values of calculated settlements, and not by their agreement with the actual variation in deformation properties with depth.

4. In my opinion, the explanation lies in the fact that the influence of the soil self-weight is not restricted to precompression but extends to the magnitude of displacements from the external load, these displacements are the more hindered, the greater the load from the soil weight experienced by the base element. The experimental data obtained by Cherkasov, Mikheyev et al and the results of experimental investigations on a centrifuge submitted to this Conference by Polshin, Rudnitsky and others seem to support this view.

The papers submitted to the Conference give much space to the compressible-layer scheme. The above-mentioned paper by Malikowa contains experimental data which give an idea of the thickness of this layer. The layer depth may tentatively be estimated at 15m. In his report to this Conference Dalmatov indicates that in the case of weak soils the compressible layer is much thicker.

Many papers deal with calculation of settlements under loads causing nonlinear soil deformations. Two laws of non-linear strain-stress relationship are considered. (Fig.2)

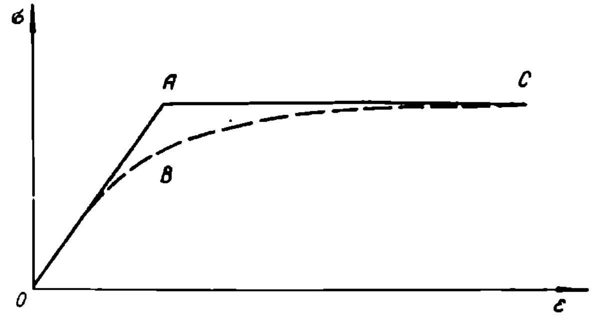


Fig.2. Dependence of deformations ε on stress σ. OAC-in solving mixed problem of theory of elasticity and theory of plasticity of soils; OA- section of elastic deformations; AC- section of plastic deformations. OBC- in solving problem with the same smooth law of stress-strain dependence.

According to the first law, the relationship assumes the shape of a certain smooth curve which coincides in its initial portion with the linear relationship by the theory of elasticity and then asymptotically approaches a horizontal line corresponding to a complete development of plastic deformations. According to the second, simplified approach, the strain-stress relationship law has a bilinear shape: the initial portion coincides with the linear relationship by the theory of elasticity, and then the law is described by a horizontal line corresponding to plastic deformations. Thus, according to the second law each soil element can be either in an elastic or in a plastic state alone.

In the USSR, a method for solving problems covered by the smooth non linear law was first suggested by Vinokurov, who made use of Ilyushin's method of small elastic-plastic deformations.

This solution received a further development in the paper by Shirokov, Solomin, Malyshev and Zaretsky. They considered various empirical dependences of the shear modulus on two stress invariants. They also obtained plots of nonlinear settlements of a rigid circular plate, contact pressures and equal-shear-modulus lines. The same authors have submitted a paper to our Conference covering a strip plate as well as a circular one. Desai stated the problem in similar terms. His results are in good agreement with experimental data.

Later on, Desai extended the method to the case of bilinear relationship, suggesting an iteration method for this purpose. This method is described in the paper by Wittke and Samprich submitted to our Conference. The authors have applied methods used for solving a complex three-dimensional problem on a rod anchor.

uprights supported by separate foundations. Larnach has compiled a computer program for this case, and the calculation takes only 30 seconds. For the case where the lower girder of a frame is a foundation beam calculation methods are given in the works of Kiselev, Zhemoohkin, Sinitsin, Simvoulidi, Gorbunov-Possadov. The same problem has been solved for tunnel framing by Davydov and Kany. De Beer, Grasshof, and Kany have published a book where they describe their methods for designing frames on a compressible base and compare different results, appropriate examples are given.

Designing continuous slabs for support networks has recently acquired great importance. This type of foundation is gaining popularity due to the development of multistory-building construction, especially in the case of weak bases. Continuous-slab design with the use of various methods is covered by the paper of Klepikov, Bobritsky, Rivkin and Malikova submitted made of the finite-element solution by Hudson and Stoltzer. Using this method only 20 to 30 seconds is needed to calculate a slab on a powerful computer. In the algorithm, of interest is the isolation of elements in bending only ("beams") and in torque only ("shafts"). The base is Winklerian, with a variable modulus of subgrade reaction. An auxiliary drafting device plots all the curves.

Prof. S.S.DAVYDOV (USSR)

III. Taking into Account Time Factor

In calculating the interaction of clayey bases and structures, it is very important to take into account the nature of progress of settlement in time. It should first be noted, however, that despite the great achievements in settlement calculation with soil consolidation taken into account, these investigations have not yet reached a stage at which the problem of progress of deformation and the stressed state of both the base and the structure could be attacked.

A comparison of calculated and actual settlement of structures on watersaturated soil bases and of their stabilization time shows that in most cases the theory of filtration consolidation of a soil mass does not reflect the actual behaviour of the soil under load. Various modifications of the consolidation theory have been advanced which allow for some additional factors.

Attention should be given to works dealing with specific features of consolidation of precompacted watersaturated clayey soils. In consolidation of clayey soils of rigid-plastic or even plastic consistency the predominant effect is exerted by the viscous resistance of the soil skeleton. Here, the compaction times of layers under compression are not proportional to the squares of their theicknesses. The effect of the thickness on the settlement stabilization time in the case of the one-dimensional problem was studied in detail in the work of Maslov and Le Ba Lyong. Using the results of many experiments, they showed the dependence of the consistency and the plasticity number of a clayey soil.

The importance of taking into account the effect of the rheological properties on the deformability of watersaturated soils is also clearly revealed by comparing the portion of settlement which has occurred after the dissipation of the pore pressure (secondary consolidation) with the magnitude of filtration consolidation. An analysis of the experimental data obtained shows that the process of compression of the pore liquid occurs in agreement with the filtration consolidation theory exclusively for soils not subjected to precompaction. Precompaction results in the formation of an ever-increasing number of structural bonds, which influence the mechanism of transmission of the external pressure to the pore liquid.

In Zaretsky's work the dependence between the skeleton deformation, total stresses and pore pressure is presented in the general form. It is shown that the given tensor relations can be generalized to the case of taking into account the rheological properties of the soil skeleton.

In their paper submitted to the Conference, Hata, Ohta and Ioshitani make an allowance for soil dilancy in considering problems involving the settlement of a soil base. They also substantiate the expression relating variations in the porosity coefficient to the mean effective normal pressure and shear stress over an octahedral area. These investigations are closely connected with the works of Marayami Matsuoka and are based on the energy theory proposed by Roskou.

The utilization of the Biot theory (1941) is demonstrated in a very interesting paper by Yamagushi and Murakami prepared for our Conference. The authors consider the problem of consolidation of a layer of limited thickness under the effect of a strip and a circular plate. In Zaretsky's investigation of 1967 this problem is analyzed not only under the assumptions of the Biot theory, but also with an allowance for the effect of the creep of the soil skeleton on the strained-stressed states of the layer under consolidation and on the dissipation of the pore pressure.

IV. Designing Reinforced-Concrete Foundations by the Limiting-Equilibrium-and -States Method

Soviet scientists have advanced and introduced into building practice a new reinforced-concrete theory based on taking into account elastic and plastic properties of materials.

When applying modern methods for estimating the strained-stressed state of the soil in foundation design we must also use the new conceptions of the work of concrete and reinforced-concrete foundations as systems which are at limiting equilibrium and are working under limiting conditions. In a reinforced-concrete foundation, on an elastic and elastic-plastic base in bending, plastic hinges and formed at cross-sections with maximal bending moments, mainly due to the development of plastic deformations in the compressed zone of the concrete.

At the site of a plastic hinge, an angular displacement is imparted to the structure as a result of which the soil pressure beneath the foundation and the bending moments at its cross-sections are redistributed - the bending moment at the cross-section of the plastic hinge decreases, while the soil pressure increases. Due to the application of the limiting-equilibrium method, the structure of the reinforced-concrete foundation adapts itself to operation on a yielding base, and the total expenditure of foundation concrete and reinforcement decreases. (Fig. 4).

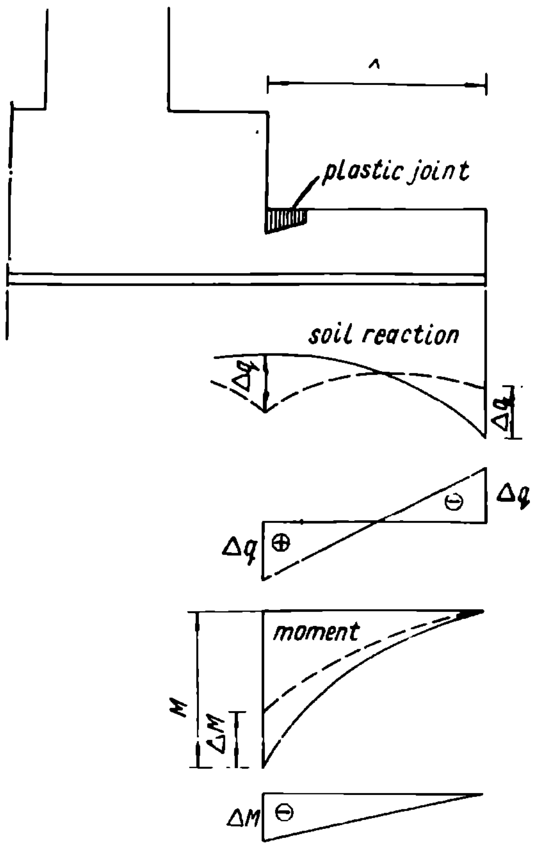


Fig. 4. Diagram for strength design of flexible foundation

However, the decrease in the bending moment at the cross-section of the plastic hinge should not exceed 30 per cent as proved by experience in designing reinforced-concrete structure (Fig. 5).

The cross-sections of a reinforced-concrete foundation are selected and the stresses checked at limiting states - for strength, stability, resistance to deformation and which is particularly important because of the ever-increasing corrosivity of ground water.

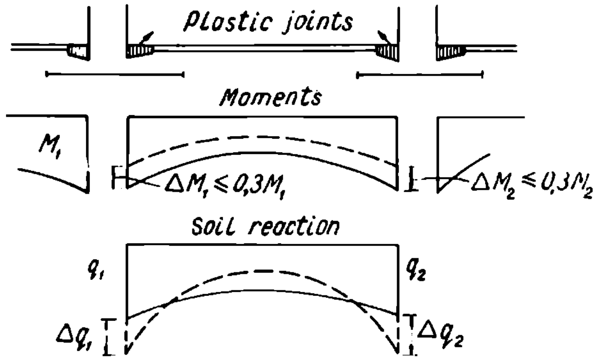


Fig. 5. Diagram for strength design of massive foundation

Massive foundations. Investigations of prefabricated reinforced-concrete foundations for the walls and supports of buildings have revealed new and very important features of their interaction with soil bases and of elements of composite bases. On their basis, methods for calculating and designing reinforce concrete foundations have been worked out in recent years which take into account the following:
- redistribution of reactive pressures beneath the base footing due to inelastic deformations in the reinforced concrete,
- the forces of friction between the base footing and the soil directed from the edges to the centre of the foundation,
- the forces of friction between the horizontal contact planes of the elements of composite foundations

Shell-type foundations. All over the world, many types of reinforced-concrete shells in contact with the soil are being widely investigated, developed and utilized. The shells take the form of separate structures or are combined with plane slabs and serve as:
- pillar foundations for frame buildings,
- foundations for tower-type structures,
- continuous and floating foundations,
- shell piles (cylindrical and conical),
- anchor foundations,
- back-pressure walls,
- tank floors,
- tunnels, tanks, hoppers, etc.

The shell is the most rational structural form which operates three-dimensionally and admits of high loads with a minimum expenditure of material; its curved shape enables reducing the stress concentration at contact with the soil, decreasing settlements, achieving a more uniform stress distribution in the base, diminishing the soil pressure on the retaining walls.

Shells may be made of high-strength materials of small thickness without risking the loss of stability.

V. Interaction of Soil with Underground Structures

The construction of underground structures in towns and cities is developing at a rapid pace. Thus, in Moscow, experimental construction of motor-car garages and parking lots, approaches to them and other underground structures is sheduled to be completed by 1975. In Paris, France's first "entertainment, sports and culture city" is to be built at a depth of 20 to 30 metres in the centre of the capital. The same site will accommodate the National Library Fund of 1 million copies with an underground railway line leading to it. Tall buildings with multi-story basements and other underground facilities are already under construction. The material submitted to the Conference includes an interesting paper by Kerisel and Loupiak on the building of a huge underground station in Paris in the vicinity of the Grand-Opera under extremely arduous conditions. In Geneva, an underground garage for 500 motor-cars has been built, it has the shape of a cylindrical open caisson of diameter 57 metres and depth 28 metres. Also in Geneva, the world's first underwater garage has been built under the Lake of Geneva. The garage has four stories and holds about 5,500 cars. The city of Chicago has an underground garage for 2,000 cars, Los-Angeles for 5,000, and Canada has a parking lot in the form of a 14-story underground building. Widescale underground work is being conducted in Japan, Yugoslavia and other countries.

All this attracts the attention of civil engineers to the solution of problems involving the interaction of soil with underground structures. This is becoming one of the main goals of current construction investigations.

The authors deeply appreciate the cooperation of Yu.K.Zaretsky, D.Sc.(Eng.), who has undertaken the onerous job of elucidating the problems of consolidation in our report. They wish to thank Professors N.A.Tsytovich, M.N.Goldstein, B.I.Dalmatov, P.D.Evdokimov and also Yu.G.Trofimenkov, Cand.Sc.(Eng.) and R.A.Tokar, Cand.Sc.(Eng.) for their valuable advice during the preparation of the report.

Conclusions and Recommendations

Development of the theory and methods of calculation.

1. It is essential to further develop the theory of a nonlinearly deformable soil medium both on the basis of experimental smooth laws of deformation and on the basis of the simplified bilinear scheme. For this purpose it is advisable to make use of all the facilities provided by an electronic computer, the finite element method and the method of equivalents. It should be kept in mind that the aim of these investigations is to work out practical methods of calculation on the basis of strict solutions.
2. It is necessary to extensively develop the methods of designing foundation beams and slabs on a compressible base by the means of electronic computers, and to apply these methods in building practice. Such design methods should take the interaction of the whole complex: superstructure, foundation and base, into account.
3. Further in situ studies are required of the performance of foundation beams and slabs with an integrated study of their bending, of the distribution of the reaction pressures, the stresses in the reinforcement and the concrete (the bending moments), the magnitude of the loads transmitted by the various supports.
4. It is necessary to work out methods of calculation that take into account the stress redistribution due to plastic deformations (formation of fissures) in foundation structures.
5. A further development of methods of optimal design of foundation structures is required. Here, the most expedient trend is the integrated optimization of the parameters of the super- and foundation structures with their performance taken into account.
6. It is necessary to investigate the joint performance of foundation structures and saturated soil bases, taking into account the consolidation and the creep of the soil base, and the creep of the material of the foundation structure.

Design Practice

1. In calculating the foundation settlement under separate supports use should still be made of the summation method, which takes the compaction of separate soil layers into account by means of the equations of a linearly-deformable medium.
2. Settlement calculation of long continuous slabs under groups of supports should be carried out on the basis of actual rapid damping of the deformations with depth, making use of either the scheme of the compressible layer or the scheme of the modulus of deformation which increases with the depth.
3. Every effort should be made to implement rapid development of algorithms and programs, by means of which the supperstructure, the foundation and the soil base can be computed as an integral unit.
4. It is recommended that foundation beams and slabs on elastic bases be designed on the basis of the theory of a linearly deformable medium. Use can also be made of methods where the mechanical properties of the soil base are characterized by the rigidity factor whose law of distribution is determined from the predicted settlements, or by other methods, that take into account the distribution properties of soils.
5. In calculating foundations of considerable width it is advisable that the plastic deformations of the soil which develop under the ends of beams and the edges of slabs be taken into account, and calculation methods be applied that take into account the plastic phenomena in reinforced concrete (plastic hinges).

b. Topics for Discussion

1. What model of a soil base more adequately suits its natural properties in calculating settlements? Ditto for designing struc-

tures on an elastic base?What changes should be made in the models for designing continuous foundation slabs of great length?

2.What is the actual outlook for compiling programs for computers by means of which the superstructure,the foundation and the soil base can be calculated as an integral unit? What features should the algorithm of these programs have for various types of structures?

3.In what cases is it necessary in calculating the foundation settlements to take into account the time factor and the creep of the material of the superstructure?

4. How and in what cases is it necessary to take into account the existence of a zone of the limiting state and the creep of the soil? Ditto for the interaction of bases and superstructures,taking into account the occurance of plastic hinges in the reinforced concrete?

5.What are the most efficient methods for taking into account the nonlinear character of base deformation in calculating settlements and designing structures on elastic bases?

6.What is the outlook for making use of a soil base model in the form of a ponderable nonlinearly deformable half-space for calculation?

The authors are very sorry that all the material received by them could not be used in this Report,mainly because much of it was received too late.

Chairman Prof.E.De Beer
Thank you Prof.Gorbunov-Possadov and Prof. Davydov for the report you've made.
We will now hear the prepared discussion.
May I now ask Mr.Rochette, to be so kind as to give us his discussion

Mr.P.A.Rochette. University of Birmingham, England

Summary. Interaction with rigid foundations can be taken into account in foundation design,by considering it as a superimposed phenomenon on the distribution of stresses below the edges of the foundation. It mainly produces a horizontal redistribution of the pore water pressures; in itself, it is then far less complex and variable than the transmission and dissipation of the pore pressure with depth.In other words, the simplicity of the superimposed interaction is no longer apparent when the distribution of the void ratios and settlement with depth have been over-simplified. The purpose of this communication is to present the mechanisms of settlement and consolidation as being non-uniform and highly dependent on the drainage conditions: A) The conventional case of vertical drainage (fig.1,a to c) B)Assumption of radial (Fig.2) or other drainage pattern (fig.3). It is shown how the final distribution of the void ratio, although generally assumed uniform, varies in fact with the drainage condition. This is derived from the result (fig.1a) of a hydrodynamical approach of consolidation established in 1969 and to be published separately.

1. Existence of a denser zone in the vicinity of an impervious end

(fig.1a). For a uniform load p_1, at the approach of the lower impervious end, the theor shows a relative decrease of pore pressure,constant over say a quarter of the layer thickness. The final void ratio then decreases progressively with depth and takes a lower constant value near the impervious boundary. The same trend is observed for a higher pressure p_2.

2. Drainage at both ends produces higher void ratios but increased settlement. In fig.1b,dashed lines reproduce the case of upper drainage described in fig.1a, and continiuous lines emphasize the symmetrical effect of double drainage. This second case, gives higher values of the void ratio. For a pressure p_2-p_1, the settlement at each depth corresponds to the distance between the continuous lines; it remains higher for double drainage.The additional settlement due to the double drainage, is proportional to the difference between the two shaded areas of fig 1b.

3. Mechanism of consolidation for vertical drainage

The initial and final lines (p_1) and (p_2) for the void ratio have been drawn in fig.1c, as ADD'A' and CEE'C' respectively. A new mechanism of consolidation is portrayed in two stages: (1) The rapid formation of denser ends (see arrow 1: e_A decreases to $e_{A'}$; e_C to $e_{C'}$), due to escape of water from both ends. During this process the density, initially higher in the middle third (see ADD'C') becomes uniform within a few days (in the field; see BDD'B'); then it stabilises, in a matter of weeks at CDD'C',with unchanged density at the central third. (2) Long-term pore pressure dissipation of the central third:DD' gradually takes the position EE' (see arrow 2) During this stage the soil, initially denser at the ends (CDD'C')becomes uniform in a matter of months (see CC'); the process of slow dissipation outward of the pore pressure reaches stabilisation over years only,after densification of the central part (relatively lower pore pressure,sometimes negative; see CEE'C').

4. Mechanism of horizontal drainage

Fig.2a illustrates the distribution of the void ratio, uniform in depth but with denser central column,since the vertical axis can be considered as an impervious boundary in the case of a radial drainage.fig.2b shows the lines of equal void ratio;the mechanism of consolidation therefore consists of the successive dencification of anular rings:the rapid consolidation of the outer zone(see arrow 1) is followed by the slower progress inward (see arrow2;relatively short term if the horizontal permeability is large);a central column of constant void ratio of minimum value is eventually formed below the rigid foundation.

5.Mechanism of partly vertical and mostly horizontal drainage.Fig 3 shows the combined effects of horizontal drainage (emphasized in fig.2),and of vertical drainage (see fig 1). The void ratio in fig 3a decreases in all directions towards a small central zone.The mechanism of consolidation is illustrated by the lines of equal void ratio in fig.3b;three successive stages can be distinguished:(1)Rapid consolidation from the edges along arrow 1;(2) short term consolidation progressing horizontally at a decreasing rate (see arrow 2);(3) long term consolidation vertically along arrow 3,since the vertical permeability is generally many times smaller than the horizontal permeability.

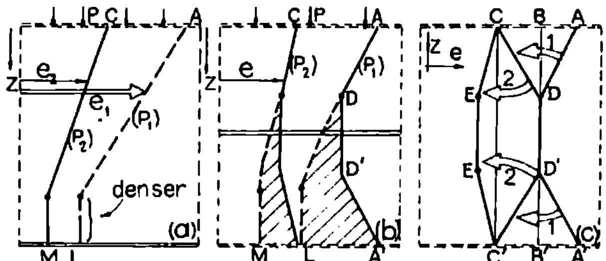


FIG.1.TYPICAL VARIATION OF VOID RATIO WITH DEPTH Z UNDER PRESSURE P_1, THEN UNDER PRESSURE $P_2 > P_1$, ASSUMING VERTICAL DRAINAGE AT UPPER(a) OR BOTH(b) BOUNDARIES; GENERAL PRINCIPLE OF CONSOLIDATION COMPUTATION; (IN (c);RAPID FOR ARROW 1; SLOW FOR 2).

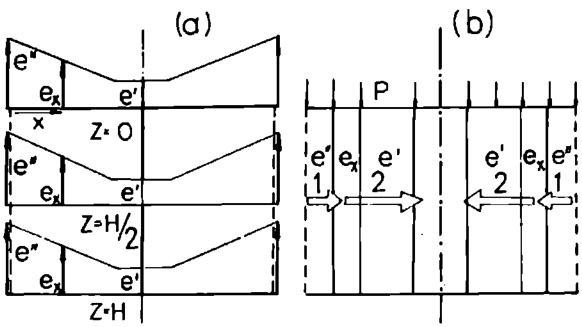


FIG. 2 VOID RATIO DISTRIBUTION FOR HORIZONTAL DRAINAGE ($k_h \gg k_v$)
(a) DENSER CENTRAL COLUMN; (b) LINES OF EQUAL e AND MECHANISM OF CONSOLIDATION (ARROWS: 1 RAPID; 2 SLOW).

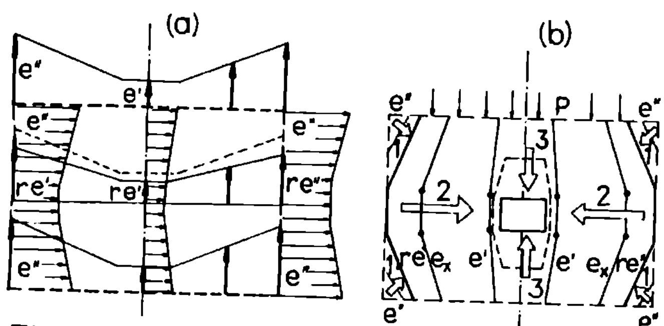


FIG. 3 VOID RATIO DISTRIBUTION FOR HORIZONTAL AND VERTICAL DRAINAGE ($k_h \gg k_v$)
(a) DENSER TOWARDS CENTRE; (b) LINES OF EQUAL e AND MECHANISM OF CONSOLIDATION (ARROWS: 1 RAPID; 2 SHORT TERM; 3 LONG TERM).

Chairman Prof. De Beer.

Thank you Mr.Rochette. Now we will hear Dr.Sommer from West Germany.

Dr.Ing.Sommer H.(West Germany)

CONTRIBUTION OF THE DIFFERENCE BETWEEN MEASURED AND PREDICTED SETTLEMENTS

In Frankfurt/Main we are constructing buildings with thirty and fourty stories on the medium-stiff to stiff clay,consistency index I_C=0,9, with the bearing pressure of 3.5 till 5 kp/cm^2. The high-rise buildings are located closely next to the existing buildings.

A 100m high building surrounds an existing building as it is shown on the left side of Fig.1. There is also an existing building

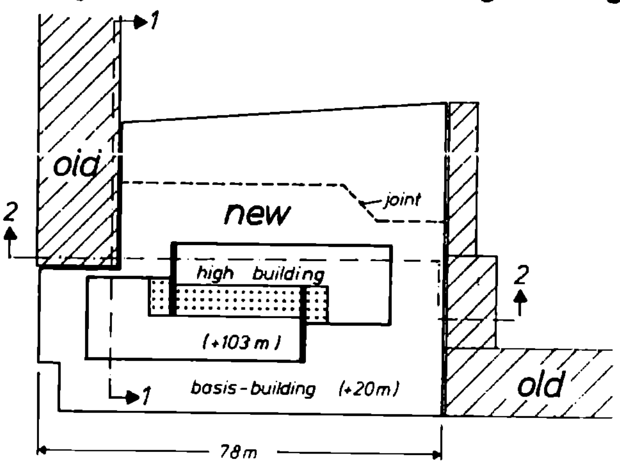


Fig.1. Location of the new building and old ones

on the right side. The high-rise buildings is founded in 13m depth, the existing buildings in 4.5m. During the construction the existing buildings were protected by a concrete cut-off wall, anchored in clay. In two sections (1 and 2) we can see the settlements of the high-rise building and the existing buildings during and after the construction. The high-rise buildings settled to a total of about 9cm, while the existing buildings settled only 3.5cm. The step-change settlement (section 1) was 5.5cm between these two buildings (Fig.2). The angular-tilting of the left existing building toward the high-rise building amounts 1:1500 (Fig.2). In longitudinal section (2-2) the left existing building makes a torsion of about 1:1000 towards the high-rise building (Fig.3).

The step-change settlement between the high-rise building and the right existing building was about 5cm and thus very similar to that of the left side (see longitudinal section 2-2).

The close-in-measurements between the high-rise building and the right existing building showed a great difference in settlement.

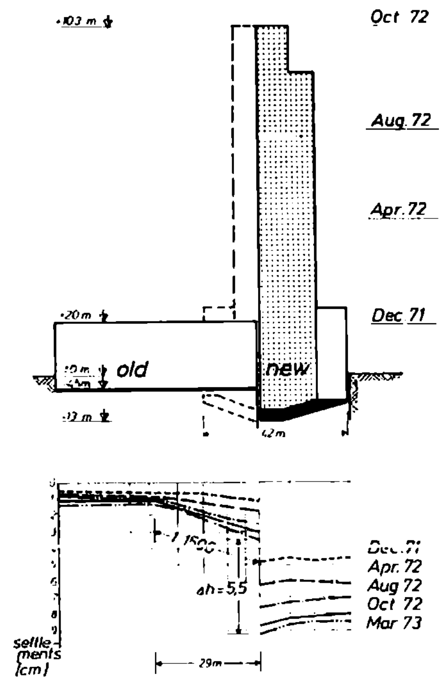


Fig.2. Settlements in cross-section

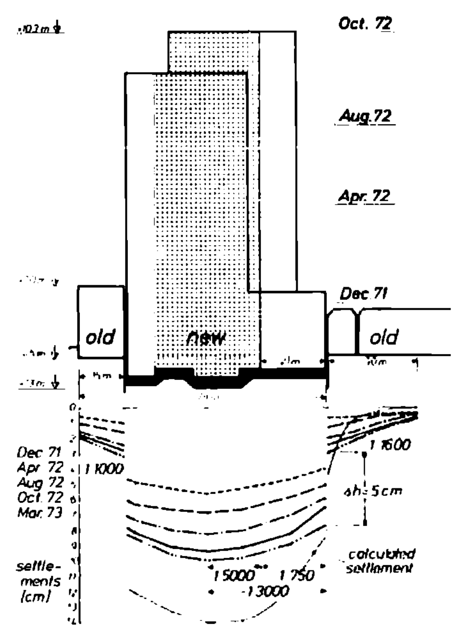


Fig.3 Settlements in longitudinal section

If we assume the high-rise building shows a settlement of 100%, the concrete cut-off wall, located closely between the two buildings, has 50%, and the existing building has 30% (Fig.4). At another site we additionally test if the soil below these footings of the old building which are close to the high-rise building settles more than the footings and causes a void.

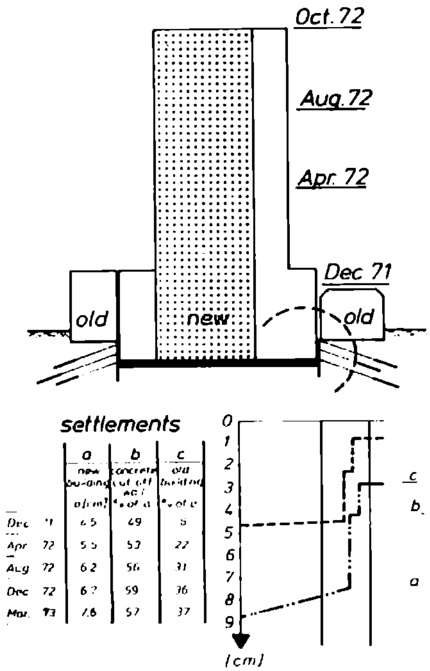


Fig.4. Settlements between new building, cut-off wall, and old building

According to the half-space calculation on the right side of the longitudinal section (2-2) the angular-tilting of the existing building was larger and the step-change in settlement to the high-rise building was smaller. In the light lines of Fig.4 one can see the calculated settlements, in the heavy lines the measured settlements. In the calculated settlements there is a larger tilting and a smaller step between existing and new building.

According to the calculations the old buildings should have cracked; this was not the case, however. We therefore must modify our modell of relation between soil and the structure in order to have a smaller difference between prediction and measurement.

Chairman Prof.De Beer.

Thank you Mr.Sommer. The next contribution will be in charge of Prof.Aleksandar S. Vesic, Duke University,USA

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

My comments are related to the topic 1 proposed by the General Reporter and deal with the problem of selection of an appropriate subgrade model for analysis of mats and continuous footings. The general report stresses the fact that the effect of rapid decrease of soil deformability with depth can be taken into account by either introducing an ideal soil mass with a modulus increasing with depth of by considering the soil as a compressible layer of finite thickness. However the problem of selecting the effective thickness of soil which is assumed to be compressible remains somewhat unsettled.

We have in recent years investigated in some detail the case of slabs resting on a compressible soil of finite thickness. Starting from the basic solutions of this problem derived for an infinite slab by Hogg (1944) and for a semi-infinite slab by Pickett and Badaruddin (1955) we have found that the bendingmoments and deflections, as well as other statical influences along the slab are well simulated by the simpler model of a Winkler subgrade having a coefficient of subgrade reaction k equal to:

$$k = \frac{1.38\ E_s}{(1-v_s^2)H}$$

Here E_s and v_s represent, respectively, the modulus of deformation and the Poisson's ratio and H the thickness of the compressible soil. The above expression is valid as long as the thickness H does not exceed 2.5 stiffness radii l_o of the soil or as long as

$$H \leq 1.38 \sqrt[3]{\frac{E\ (1-v_s^2)}{E_s(1-v^2)}}\ h$$

where h is the thickness and D,v deformation parameters of the slab.

Comparisons of this solution with the Gibson (1967) solution for modulus increasing linearly with depth show that, by taking, for Gibson soil situation, an average E_s over the depth $H=2.5\ l_o$ and by neglecting the compressibility of the underlying soil, one obtains only 15% higher values of k or only 4% higher values of bending moments. This comparison shows that a representative soil model in all situations where E_s is variable, but generally increase with depth, can be obtained with sufficient accuracy by taking an average E_s-value for a depth of influence equal to 2.5 stiffness radii of the slab. Such an approach allows for direct determination of k from results of pressuremeter or static cone penetration tests. It is important, however, at all times, to take into consideration the increase of the modulus from stresses induced by slab loads, as well as to consider the stress history of loaded soil and construction sequences in order to introduce appropriate E_s-values. Some edge effects, similar to those found for beams (Vesic, 1961) do exist and have sometimes to be introduced separately.

For subgrades consisting of distinct horizontal layers of different compressibility we have developed analytical techniques using high-speed electronic computers. These methods consider the slab as an assembly of discrete elements and the subgrade as a layered elastic solid (Saxena,1970). We find,however, that a simplified analysis,based on above mentioned concepts,turns out to be satisfactory as long as we are not dealing with structures of extraordinary importance in soil conditions uniform enough to justify refined laboratory testing.

Chairman Prof. De. Beer. Thank you very much Prof.Vesic for your contribution. The next contribution will be of Mr.Palatnikov, USSR.

Mr.Palatnikov E.A. (USSR)

The theory of analysis for the constructions based on soil foundations made a great step forward due to wide recent research in many directions. Still the comprehension of this important part of mecanics continues to be far from satisfactory from the practical point of view let alone complete.

One of the main problems in this field is the creation of a rational computational model of foundation that can help to determine the distribution of contact stresses well enough for practical purposes and thus help to estimate strains and stresses in the foundation structure.

It is hardly reasonable to want a model of soil foundation to simulate exactly all the mecanical properties of the environment. Up to now we cannot yet take into account all the effects of the loading of soil foundations that usually are not homogeneous either in the plane or throughout the depth.

For a long time the Vinklor model was the only one used here and the high safety of constructions computated with its help has been confirmed by the world building practice for a century. Any attempts to replace this model by others result in heavier foundations and waste of materials sometimes rather great, the strength of the structures not always being excessive. If, for example, the slabs are loaded on the corners and edges,the use of new models reduce the reliability of the structure.

The analyses based on any combinations of the new models lead to contact pressure concentration near the edges of rigid dies,beams and slabs. This main feature of such analyses has not been confirmed with direct experiments on contact stresses, conducted by different researchers in natural environment on fairly homogeneous sands and sandy loams using large-scale models.

The hypothesis suggesting early development of plastic deformations near the edges of foundations has not been confirmed either. The majority of the scientists have agreed that the contact stresses are distributed under rigid dies and slabs practically in accordance with the linear law which is true to a wide range of loads. That corresponds to the Vinkler model. Experimental contact stresses that occur under flexible slabs and beams are approximated with the Vinkler model with accuracy sufficient in the sense of the problem,departures from the theory being due to natural heterogeneity of the soils located in the plane of the slab or the beam. Thus the results of experiments and building practice allow to recommend the Vinkler model as the main computational model that also helps to take into account the heterogeneity of soils located in the plane of the slab.

In order to solve definitely the problem of the foundation model it is necessary to intensify research of foundation structures in real conditions and in different soils. The experiments must be integrated and last long enough. They should include determination of contact stress, study of deformation, stressed state and temperature conditions of foundations as well as measurement of soil deformations in different layers. Soil foundation of the experimental structure should be thouroughly examined, the greatest attention being paid to the changeability of heterogeneity of soils located in the plane of the foundation. For the first stages of investigations it is advisable to choose such the objects that will be placed on comparatively homogeneous soils and have a hyperstatic scheme of loads transmitting out of the upper structure down to the foundation. It is absolutely necessary that the temperature conditions of the foundation should be studied for they quite considerably influence the contact stress distribution, deformation and stressed state of the structure.

The best is to make a coordinated plan of research and to have a special report on the results at the next Congress.

Chairman Prof. De Beer. Thank you Mr.Palatnikov. Now we will hear Mr.Toshkoff of Bulgaria.

Dipl.Ing. Emil TOSHKOFF, (Bulgaria)

Dear Mr.Chairman,
Mr.General Lecturer,
Dear Delegates,

At the contemporary stage of development of building science the designing of raft foundations /flexible foundation beams and slabs/ should be subjected to two main requirements:

- taking into consideration the coaction of superstructure /at least approximately/
- programming and automation of calculating process /applying computers/

The model of lineally-deformable space with a limited depth of the compressive stratum is indisputably the most appropriate for soil base. In my opinion, however,the model of Winkler could be admitted too with automatized calculating of raft and mat foundations and only for determining the inner efforts in the slab. In this case Winkler's constant $K/T/m^3/$ ought to be determined in the converse way-through the settlement of a definite fragment of the whole foundation,calculated in the traditional method. For seismic effects the constant K is increased $3 \div 5$ times.

The structures laid on common foundations should be distributed at least in 3 groups:

I-st group- with flexible superstructure /for example, wholly columnar buildings, buildings with columns and single shear walls and so on/. In this case the rigidity of superstructure could be ignored or counted by general halfempirical coefficients and the foundation could be considered as a detached slab on flexible base.

II-nd group- structures with halfrigid superstructure /for instance constructions with large-size precast panels/. This case is the most complicated since the coaction of the superstructure is indisputable but is no susceptible to schematizing especially as regards to programming of calculating method.

The joints between the panels, the speed and the method of making them monolithic, the slots in them and so on, create a lot of not cleared up problems. That is why at the present stage of carried out research on the problem we could content ourselves to take into consideration wholly the rigidity of cellar monolithic walls and very cautiously and with approximation- the coaction of the superstructure. Only the joint work of constructors, soil mechanics and programists could lead to a more perfect method of research.

III-rd group -buildings with a rigid superstructure /for instance buildings with bearing reinforced walls in both directions interconnected and beginning practically from the level of the foundation/. In this case the definition on the general scheme "slab on a flexible base" looses its sense. The spans of the foundation could be calculated for the overturned soil reaction intended as for a rigid body with the dimensions of the whole building. The walls of the building are regarded as fixed supports of the foundation. Nevertheless some experimental observations show that such buildings give certain deflections in the building period which is explainable. Some constructional measures are required for the purpose, the most important being in our opinion, laying of sufficient and unbroken lower and upper reinforcement-nets in the slabs. Those parts of the foundation which come under slots in the walls require also additional reinforcement.

The average settlements of the buildings are dependant on the type and rigidity of the superstructure and the common foundation, which is correct as a trend, although difficult to prove theoretically.

In our practice for instance it is not recommended that the average settlements of the building laid on raft foundation should be more than:

a/ for structures built after the lift-slab method /1-st group/- 10 cm.
b/ for structures with large-size precast panels /IInd group/- 8cm
c/ for structures built after the "sliding shuttering" method /III-rd group/-15 cm

Chairman Prof. De Beer.
Thank you Mr. Toshkoff. Mr. Lasebnik, please.

Mr. G. LASEBNIK, (USSR)

We agree that the previous simplified approaches which consisted in the employment of Winkler's hypothesis or homogeneous elastic half-space as a subsoil analogue are out of keeping with reality. We find it more proper to apply an experimentally established stress-strain non-linear law to all stages of soil action, which should include comprehensive large-scale and full-scale tests.

The deformity of the entire system, that is the structure-footing, and flexibility should be calculated proceeding from this mutual deformity in accordance with the construction stages, taking into account the subsoil characteristics.

Depending on the mode of application of loads to the slab-foundation, on the rigidity of the system, i.e. structure-basement, on the degree of deformity and homogeneity with regard to soil deformity, as well as on other factors, the charts of distribution of reactive pressures may vary /1/,/2/.

Under full-scale conditions we often encounter cases which are difficult to provide for in calculation.

To confirm this we shall provide the results of full-scale tests of the pressures under the slab-foundations of two buildings in Kiev, one having 12 storeys and the other 16 (see Figs.1 and 2). The width of the slab-foundations and their flexibilities are approximately the same. The depth of each is 5 metres and more. The weight of the edifice is applied to the slab via four columns which are arranged similarly. The shape of the distribution chart differing, however. This is due to differences in the uniformity of the basement. The site on which the building shown in Fig.I, a was erected, was formerly a ravine. The broken dash line shows the contour of the ravine sides. Earth pressure stressmeters located over the "bedrock" whose deformity is negligible, shows increased pressures. The basement of the other edifice consists of undisturbed-structure loam. No "peaks" were found at the edges of the chart of distribution (Fig.1b.)

The basement under the slab-foundation of great length is periodically subjected to loading and unloading in various sections in the process of construction.

The slab-foundation of building "a", dimensioned 106x13x1 m is not rigid in the longitudinal direction. The building is erected in parts, rather than over its entire area. In this case, construction loads produce forces which deform the solid slab. Fig.2. provides a diagram of pressure changes for each of the seven stressmeters in the process of erecting the building (first year of construction). The stressmeters record the highest pressures when the greatest load is located at the cross-section of their location. If the walls, columns etc. are erected to one side of the cross-

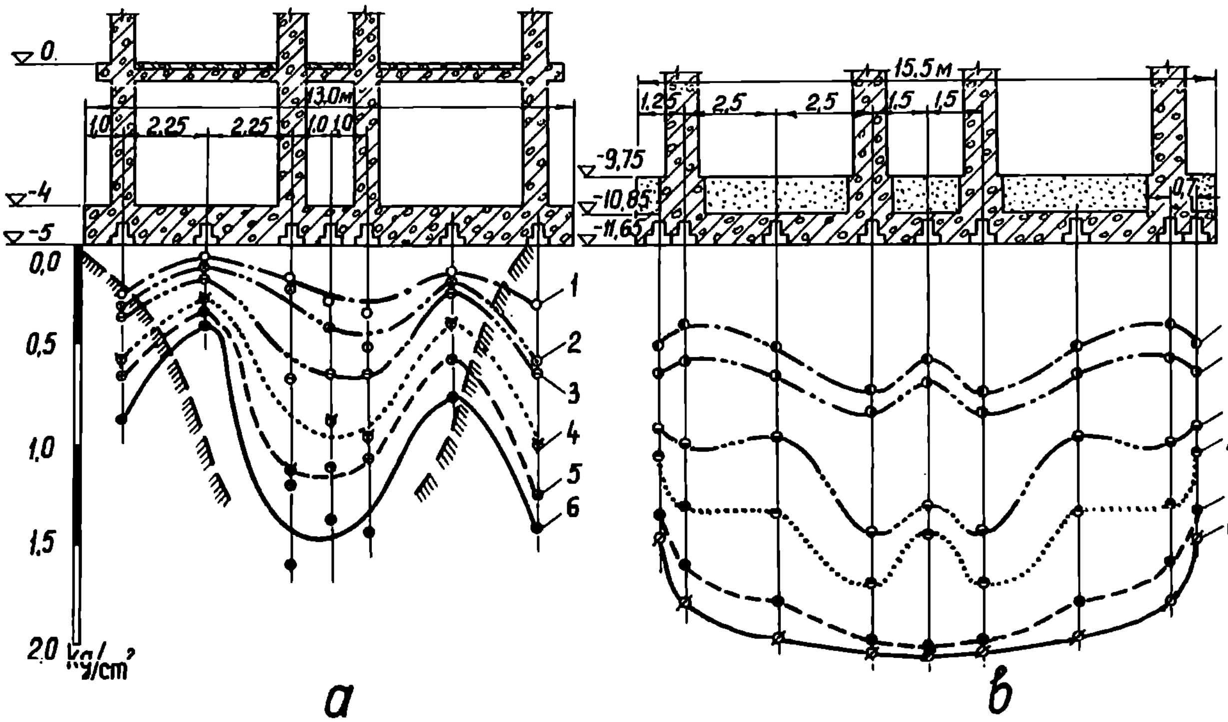


Fig. I. Schematic cross-sections of the slab-foundations of two buildings:building "a"- erected on the site of what was once a ravine which was subsequently filled up; building "b" - erected on a basement of undisturbed structure soil.
Charts of distribution of reactive pressures of soil at the following loads:
"a" I-cellar and ground floor has been built; 2- Ist floor; 3- 3rd floor; 4- 7th floor; 5-IIth floor; 6- building has been erected to roof.
"b" I- three floors of monolithic section has been built; 2- four floors of unit-construction section; 3- eight floors of unit-construction section; 4- twelve floors; 5-frame has been fully erected; 6- equipment has been installed,finishing work has been carried out partially.

section with the stressmeters the pressure decreases (see Fig.2). When the total load on the structure increases, the undulating deflections become increasingly smaller.

Such an observation can be made only in full-scale tests.

We would like to make an addition to the mention of our works by the authors of the general report. In our opinion,slab flexibility increases not only because of the fracturing of the concrete,but also because of an increase in the rigidity of the soil basement under large-area slabs during a load increase.

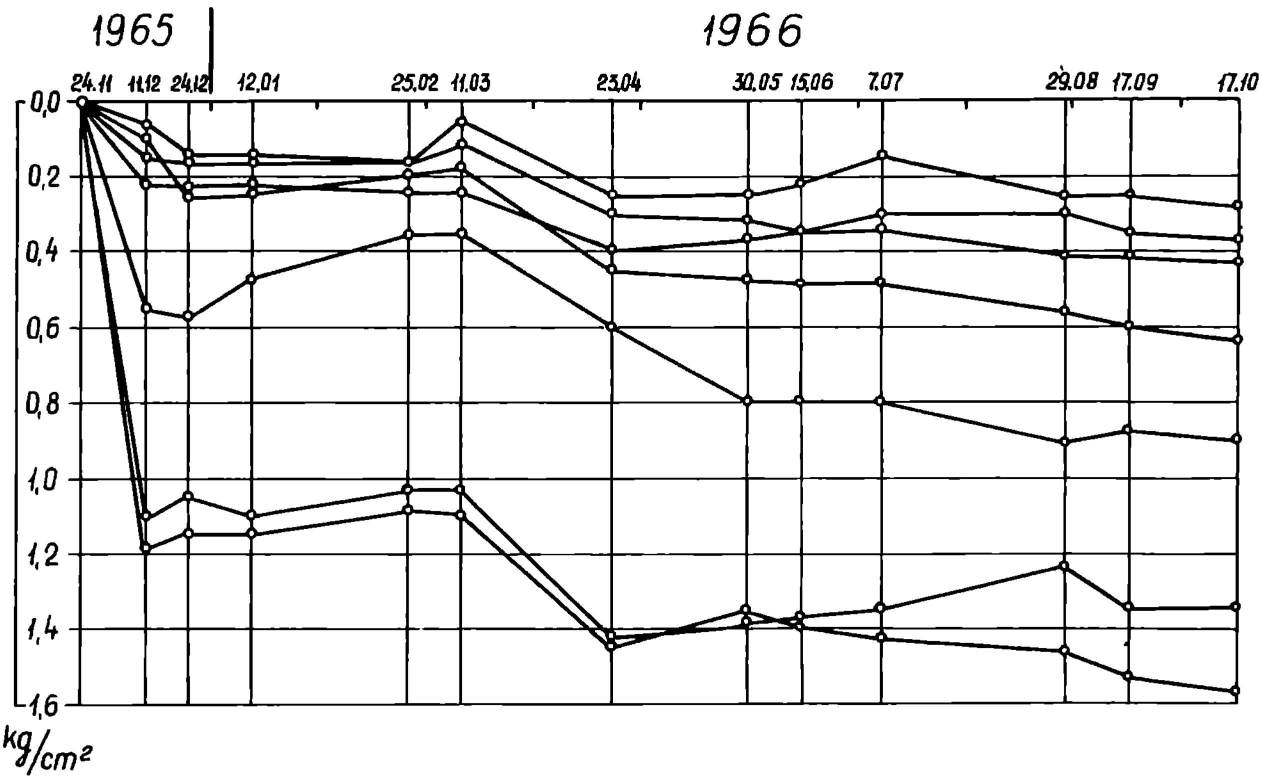


Fig.2. Diagram of change of readings of earth stressmeters in the course of time depending on the transfer of construction

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Chairman Prof.De Beer.
Thank you Mr. Lasebnik.

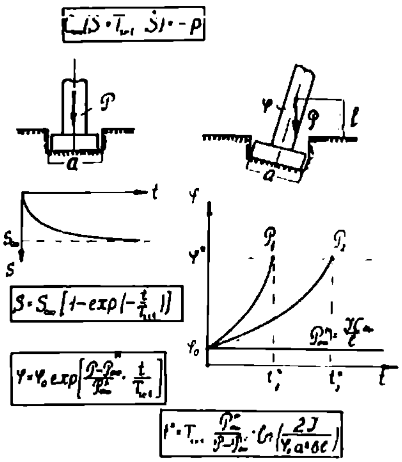
The next contribution will be in charge of Dr.Zaretsky, Research Institute of Bases and Underground Structures, USSR.

Dr.Sc. Yu.K.ZARETSKY, (USSR)

I should like to express my views on the necessity to have the time factor coverage in calculating foundation settlement. Unfortunately, it is precisely the time factor that prevents me from delaing with this question in detail. Therefore, allow me to call your attention only to the following.

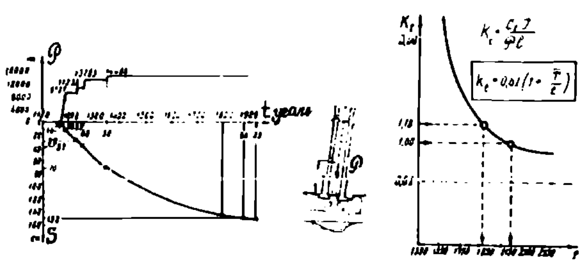
The opinion has been repeatedly stated that there is no necessity for using the theory of soil creep in foundation base design. This opinion is based on the fact that loads leading to unstabilizable deformations are not permissible. Therefore, in calculations it is sufficient to have available the stabilized modulus of deformation (or modulus of subgrade reaction). It can easily be shown that such an opinion runs to an extreme

Let us consider the simplest case of a Winkler base with linearly viscous properties as illustrated in Fig.1. Here the settlement of a centrally loaded foundation is determined by an expression describing a damped process of deformation. But when we consider the state of equilibrium of a column in the inclined position, we find that the angle of inclination grows at an increasing rate (see Fig.1.) If we conventionally take the loss of stability as the moment when the footing of the foundation breaks away from the soil base, we can easily evaluate the critical time corresponding to the moment the stability is lost. It follows from the formula in Fig.1 that the critical time for the



loss of stability depends on the weight of the column, initial angle of inclination, geometry of the column and the rheological properties of the soil in the base.

Figure 2 gives the result of an elementary calculation of the stability of the Leaning Tower of Pisa. If the settlement of this famous tower is due to creep in the soil of its base (or if the course of settlement of the tower can be interpreted in this manner) then the stability factor, equal to the ratio of the critical load to the weight of the tower, varies with time. The tower will fall when this factor becomes equal to unity.



Calculations indicate that this is to be expected in the year 2250 if no other factors will influence its stability.

A special feature of the phenomena being considered is that at a constant external load, but with an inclined position of the column, changes occur in the contact stresses. This process leads to a loss of stability of the structure notwithstanding the fact that the settlement would have been inevitably stabilized if the structure had not been inclined. The critical time for the loss of stability of a structure depends essentially on the rheological parameters of the soil.

Chairman Prof.De Beer.
Thank you very much Mr.Zaretsky for your interesting contribution.
I pass the word to Prof. Hansbo, Chalmers University of Technology;
Prof. Hansbo, will you please begin

Prof. Sven HANSBO (Sweden)

In settlement calculations and conventional design the interaction between building and subsoil is usually neglected. This, in the design the footings are considered to be resting on a rigid subsoil, i.e. no settlements are assumed to occur, whereas in the settlement calculations the column and footing loads obtained by this method of design are considered to be uneffected by the calculated settlements. That this might lead to serious errors has been proven by e.g. Chamecki and Meyerhof. The reason why this procedure is still accepted is no doubt that it is hardly possible to make a correct analysis, not even with an electronic computer. However, a big step forward towards a solution of the problem has been taken lately in Sweden. Thus, S.E.Beigler, employee at AB Jacobson & Widmark and in his free time doctor's student at Chalmers University of Technology, has made a computer program for statically indeterminate frames founded on a subsoil, having equal geotechnical characteristics in the horisontal plane. The program is made on the following basis. Effective stresses σ' in the soil due to the footing loads are calculated according to Fröhlich's method which is advantageous since different soils require only a change of stress concentration factor. The deformation modulus M of the soil is formulated as suggested by e.g. Janbu and is thus defined only by the modulus number m, stress exponent β and the stress level σ'

$$M = m\,\sigma_j\left(\frac{\sigma'}{\sigma_j}\right)^{1-\beta}$$

where σ_j=100 kPa

The settlements under a certain load is calculated by integration of the vertical strains underneath the load caused by the stresses previously calculated. As the footing loads depend on the settlements the stresses in the subsoil will also depend on the settlements. The same is true also for the deformation modulus of the subsoil but to a lesser extent. Thus the solution has to be reached by successive iteration. The result of such an analysis will now be exemplified.

Consider a five-story, three-bay frame made of concrete. The loading conditions and the moments of inertia of the frame are given in Fig.1. The concrete is assumed non-fissured and has a modulus of elasticity of 14,2.10^6 kPa. The regarded frame is situated somewhere in the centre of the building. The influence of secondary walls and constructions is disregarded.

The frame is assumed to rest on 2m of sand in loose state underlaid by 8m of normally consolidated soft clay. The sand has a density γ of 1,8 t/m^3 and a deformation (oedometer) modulus of M=3000 $\sqrt{\sigma'}$ kPa (M and σ' in kPa). The clay has γ=1,6 t/m^3 and M=10σ'. The ground water surface is just below the base of the footings. Permissible load on the sand according to the Swedish building code is 500 kPa. The results of the analysis without and with regard to interaction are given in Table. 1.

Table 1

a) Average load on footings in kPa

Footing	1	2	3	4
Width, in m	0,61	2,25	2,25	0,61
Conventional	104	262	262	104
Interaction	155	206	206	155

b) Settlements of footings, in m

Footing	1	2	3	4
Conventional	0.105	0,188	0,188	0,105
Interaction	0,129	0,157	0,157	0,129

c) Bending moments in columns at bottom of 1st floor, in kPa

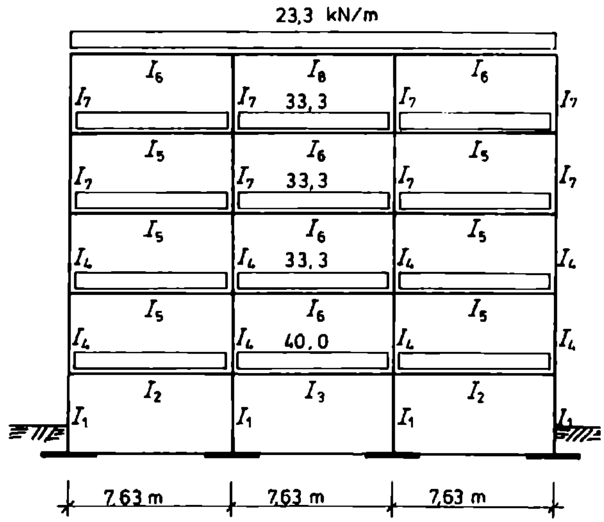
Column	1	2	3	4
Conventional	-52	9	-9	52
Interaction	-206	-123	123	206

d) Bending moments in the bottom beams of the 1st floor (left of columns), in kPa

Column	1	2	3	4
Conventional		-220	-198	-120
Interaction		113	-142	-428

As you can see the difference between the footing loads has decreased due to interaction and also the differential settlements whereas the change in bending moments has no specific trend.

Obviously, the errors in a conventional analysis can be serious and should not be neglected in sound building design. The direction of bending moments sometimes have changed and the absolute magnitude has increased by several hundreds of percents.
The effect of interaction is also essential when evaluating settlement data obtained in the field.



I_1 = 5,0 · 10^{-3} m^4
I_2 = 15,0 · 10^{-3} m^4
I_3 = 7,5 · 10^{-3} m^4
I_4 = 2,5 · 10^{-3} m^4
I_5 = 10,0 · 10^{-3} m^4
I_6 = 5,0 · 10^{-3} m^4
I_7 = 1,0 · 10^{-3} m^4
I_8 = 2,5 · 10^{-3} m^4

Fig. 1.

Chairman Prof. De Beer.
Thank you very much Mr. Hansoo for your discussion.
Now we will hear Prof. Klein, on the subject: "The main problems on the improvements of flexible beam and slab design on soil footing"

Prof. Klein G.K. (USSR)

Dear colleagues,

At present this problem is being carried on in four ways:

a) different calculated footing model creation;
b) the same for constructions;
c) the analysis of the interaction effect of a structure and a footing;
d) experimental research.

1. At present there is a great variety of different design footing models at our disposal: locally deformed, solid media, combined, granulated and etc. Along with some properties such as non-uniformity, anisotropy, non-linear deformation, single-valued reaction, creeping, stochasticity and etc. are taken into account.

2. For structures the corresponding properties of the materials they are made of may be

taken into consideration as well.

3. To consider interaction of a structure and a footing different mechanico-mathematical methods are used: methods of forces of displacements and a mixed one a final difference method and à final element method, variation method and an iteration one, etc.

4. General regularities of soil deformation research are included into the experimental analysis together with the measurements of settlement under rigid and flexible stamps and the measurements of settlements, deformation and supporting reactions of beams and slabs. The results of the experiments are rather variable and up to how it makes it difficult to chose any definite model.

The main objective for to-day is to continue experimental research with different soil footings, with different structures and to compare the results obtained to the calculation results for different models. At the samo timo it is necessary to improve; simplify and make more exact the calculation methods and to obtain data for the regulation of design mechanical characteristics of footings.

Chairman Prof. De Beer
Thank you very much Prof. Klein.
The next contribution belongs to Prof. Korenev.

Prof. B.G.KORENEV (USSR)

There is still much to be done in solving the problem of modelling the ground base. Extensive and coordinated research is needed in which experimental methods should be combined with theoretical ones. Of importance here should be not only the direct measurement of base settlements and resulting experimental equations of cores but also the determination of the latter in terms of measurements of equations of deflections and stresses of girders and slabs on a natural base and a subsequent solution of a reverse contact problem consisting in finding Fourie transformations of the model core as it was proposed by the author at the session of one of the natural congresses in mechanics in the USSR. The approach should be based on the known solution of the problems of unrestricted slabs. Let an unrestricted slab of constant thickness be supported by a solid homogeneous base, which properties are described by the core K(r) and if the function c(γ) is determined by

$$C(\gamma) = 2\pi \int_0^\infty r\, K(r)\, J_0(\gamma r)\, dr,$$

where $J_0(\gamma r)$ -is the Bassel function with zero index, then to find the core equation one can use, e.g. the equation of the elastic slab surface when loaded by a unit force which has the form (r) and is obtained from the experiment.

Denote

$$\bar{\omega}(\gamma) = 2\pi \int_0^\infty r\, \omega(r)\, J_0(\gamma r)\, dr,$$

then

$$C(\gamma) = \frac{\bar{\omega}(\gamma)}{1 - D\gamma^4 \bar{\omega}(\gamma)}$$

and the core K(r) has the form

$$K(r) = \frac{1}{2\pi} \int_0^\infty \gamma \frac{\bar{\omega}(\gamma)}{1 - D\gamma^4 \bar{\omega}}\, J_0(\gamma r)\, d\gamma,$$

where D- is the cylindrical slab stiffness.

The final result should be corrected in terms of the design purposes as a result of some compromize and certain averagings since though the diversity of models is inevitable for practical purposes it should be restricted.

It would be reasonable to comple on an international scale a general research programme in order to jointly conduct experimental investigations of grounds, beams and elastic slabs with a subsequent complex processing of the results. It would be necessary to find out also the field of application of linear and deterministic models as just these models provide great possibilities for detail description of the structural behaviour and, in particular, for taking into account crack formation in concrete and plastic deformations in reinforcement, for analizing temperature effects, for detail description of the way of load application, etc. It would perhaps be reasonable to form a working group on this problem.

I would like to illustrate the practical importance of a well advanced problem of calculating an unrestricted slab. If allows to introduce discontinuous solutions on the basis of relevant limit transitions to account of crack formation and plastic deformations. The same solutions permit to calculate in a comparatively simple way non-isolated slabs which are obtained from non-restricted ones by sectioning them along the lines.

Rather convenient methods of calculating elastic isolated slabs can be built up in terms of the above solutions. We shall describe in brief a possible design scheme. As a first approximation a non-isolated slab is considered instead of an isolated one and thereby some error is introduced due to the increase in the base stiffness near edges. To correct the error it is necessary to construct integral equations of the first order. There is a number of works on calculating slabs with account of plastic deformations and calculating slabs with sections. The works show the efficiency of the methods. However the application of these methods for calculating non-isolated slabs is not duely developed, the contents of the method is given in (I)

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Chairman Prof.De Beer
Thank you very much Prof. Korenev for your discussion.

Mr.Ohta will you,please

Mr.Hideki OHTA (Japan)

I would like to talk about the analyses of the deformation of loaded ground. Generally speaking, the fields of stresses and strains should be determined throughout the ground by solving the equations of equilibrium with the stress-strain relations.The most important subject in the deformation analyses of soil structure is to find out the stress-strain characteristics of soil. In the common way of practical design, the stress-strain relation for soil is assumed to be linearly elastic one. But if we need more likely feature of the stress and strain fields,we should use more realistic stress-strain relation,that is to say, the non-linear relation. A lot of non-linear stress-strain relations for soils have been proposed. Some of them are the empirical relations and some of them are those derived from logically self-consistent theories. Most of the theoretical stress-strain relations are in forms of incremental relations. At this session I presented a paper entitled. "Immediate and consolidation deformations of soft clay stressed by uniform strip load" in which the results of numerical computation on the deformation of clay layer are given using the incremental stress-strain relations derived from the theory of volume change characteristics of soils. In the computation, I assumed the stress field is approximately given by the theory of elasticity in order to simplify the process of computation. The obtained results seet to be acceptable in the practical engineering sense. But I could not find the failure zone located in the way suggested by the theory of slip line. In some reports of the observation of ground failure, the existence of the slip surface or slip zone is confirmed. Then I made a computation without any help of the theory of elasticity, that is, the stress and strain fields were computed with the incremental stress-strain relations by the finite element method. The results were not so far from those of prior one. Actively stressed zone and passively stressed zone were clearly separated, but any zone corresponding to the slip surface was not located at the position that we found it in the field observations. The failure zone was found only beneath the loaded area. These results were similar in their tendency to the results obtained by the other researchers who used non-linear stress-strain relations. The discrepancy between the computed results and field observations suggests the weak point of our techniques in computing the stress and strain fields with the non-linear stress-strain relation which includes the perfectly plastic state.

Chairman Prof.De Beer.
Thank you Mr. Ohta. The next contribution belongs to Prof.Giroud.

Prof. J.P.GIROUD (France)

Cette discussion concerne la prevision des tassements a l'aide de l'essai oedometrique. Cette methode presente au moirs deux inconvenients.

1° Le Fait de negliger les deformations laterales conduit a une erreur systematique

2° L'oedometre ne mesure que le tassement du au depart de l'eau et,par consequent,la prevision du tassement instantane d'un sol sature /volome constant/ est impossible.

Nontrons comment pallier ces deux difficultes. Considerons, par exemple,une Fondation circulaire de rayon R,reposant sur une couche de sol homogene d'epaisseur Fisnie H.

Pour plusieurs valeurs du rapport H/R, le tassement calcule a partir de l'essai oedometrique est compare a celui calcule par la theorie de l'elasticite en tenant compte de la deformation laterale de sul.

Le rapport de ces deux courbes donne un coefficient qui permet de corriger l'erreur commise lorsque l'on ne tient pas compte de la deformation laterale. Nous avons obtenu des coefficients analogues pour les Fondations rectangulaires.

Pour le second point /prevision du tassement immediat/, nous avous etabli,par la theorie de l'elasticite,des relations entre les proprietes du sol dans l'etat draine et non draine.

Ces relations nous ont permis d'etablier le rapport entre tassement immediat et Final.

Associons maintenant les deux resultats presentes dans cette discussion:

1° Correction du tassement calcule a partir de l'essai oedometrique.

2° Rapport du tassement immediat au tassement Final

On obtient ainsi un abaque permettant la prevision du tassement immediat directement a partir de l'oedometre /photo 6/.

Les traits pleins representent notre coefficient et les tirets celui propose en 1957 par difference vient du fait que,grace aux progres des methodes numeriques de calcul, nous n'avons pas ete obliges de Faire des approximations comme SKEMPTON et BJERRUM. Nous avons etendu ces calculs au Fondations rectangulaires.

Chairman Prof.De Beer.
Thank you very much Prof. Geroud.
Coming to the end of the session allow me to pass the word to Prof.Davidov (General Reporter). He will make some concluding remarks on the General Report.

Prof. S.S.Davidov (USSR)

Allow me first to deal with the discussion. Twelve specialists took the floor,all of them presented serious and valuable material that developed the topic of the General Report, and made concrete suggestions.

Prof.Giroud (France) discussed the investigation and comparison of characteristics of drained and undrained soils.

Dr.Rochette (Great Britain) dealt with problems of compaction and settlement of soil by means of a hydromechanical method. He established a variation of the pore pressure in soils and of their void ratio with time.

Prof. Zommer (F.R.G.) reported on the divergence between the assumed and actual settlements of buildings and structures, and illustrated his assumption on the basis of experience with multi-storey and other buildings, whose prediction was made by means of the theory of the half-space.

Prof. Vesic (U.S.A.) dealt with the problem of choosing a model of a base for designing strip foundations and continuous slab foundations. He recommended a base to be adopted in the form of a compressed layer of finite height and to take into account the influence of the edge effect.

Prof. Palatnikov, Lenin prize-winner (USSR) pointed out that in selecting a model of a base, all means should be taken to achieve a precise solution of the problem which would comply with the precision which the initial data is determined and recommended the accumulation and generalization of experimental data.

Prof.Toshkov (Bulgaria) spoke on the employment of computers and the calculation of preliminary settlements, regarding the base as a compressible layer of finite thickness.

For the case of a Winkler base, he recommended that the modulus of subgrade reaction be determined in reverse order by dealing with the layer being compressed by means of the theory of elasticity.

Lazebnik, Cand.of Tech.Science (USSR) proposed that use be made of experimental data in choosing the model of the base. He presented data on the measurement of pressures and settlements of two buildings in Kiev as the basis of his suggestion.

Dr.Zaretsky (USSR) substantiated the necessity of taking into account the time factor in evaluating the deformations of the base, citing a linearly viscous model as an example.

Prof. Hansbo (Sweden) reported on the interaction of the base and the superstructure and on the changes of the soil reaction and the bending moment depending on the settlement distribution. He presented programs for calculation by computers.

Prof. Klein (USSR) spoke on the design of building elements based on yielding bases. Various base models and mathematical methods of their design were considered. A short assessment of these methods was given.He recommended the conduction of further experimental investigations and the comparison of experimental data with that of theoretical calculations.

Prof.Korenev (USSR) discussed the selection of a design model for a base and pointed out that the problem of selecting an optimal model is still not clear. He illustrated his ideas by an example of a contact problem.

Prof. Ohta (Japan) presented an analysis of linear and nonlinear deformations of a loaded base. Illustrations were given in an example of the deformations of a light clay: actual deformations and those determined by means of the finite-element method.

Allow me now to present the conclusions and recommendations of the General Report, taking the discussion into account.

Chairman Prof. De Beer.
Thank you very much Prof. Davidov.
In closing this session I want to call on Prof. Krsmanovič for making concluding address. Prof. Krsmanovič, will you

Prof. Dušan Krsmanovič (Yugoslavia)

In the papers distributed for this session, as well as during the discussion that had been held,separate or group test and research results of pressure distribution and methods of estimations of civil engineering structures reposing on semi-space have been produced. That was done in a theoretical as well as in a practical way.

I think, however, that there is no uniform division of civil engineering structures regarding the reposing and pressure distribution below and surrounding civil engineering structures that for the present moment could be applied on soil media as well as on rock media.

Therefore, I shall try with these comments, to give the only possible division regarding the support in pressure distribution for all kinds of civil engineering objects.

As for my understanding, there is no principal difference in treating the problems of reposing, whether on soil or rock, therefore I think that it is valid for both media (continuums and discontinuums).

Grouping or some kind of systemization of can be done according to the kind of structures and according to the way of their reposing on media.

I consider that all cases can be comprised in five groups that will be discussed here in general.

1. <u>Group</u>

<u>Natural slopes without structures</u>

If a slope is unstable,generally it comes to sliding and to the formation of a sliding body (except the cases of flowing). Then we can treat the sliding body as an structure on the base on which it slides. If a sliding area or zone has a <u>cylindric shape</u>, there is no interaction if dislocations are not vast. This is not the case, however,with sliding areas of undefined shape,because then for the different positions of the sliding body appear different pressure distributions on contact areas. The change of pressure distributions may be very important depending on the rigidity of the sliding body, the shape of sliding area, type and properties of material, as well as of other factors. These influences could be partly or entirely neglected in the cases of shallow slidings.

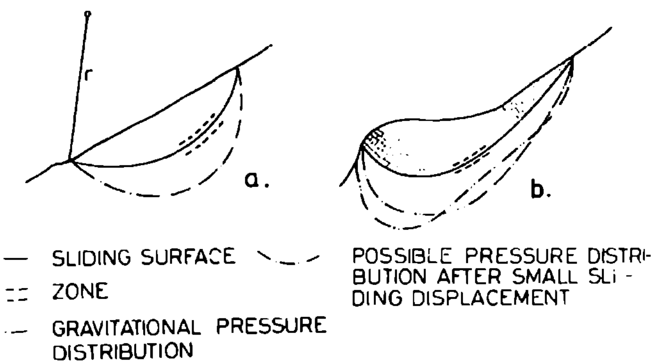


Fig 1

The whole process happens in the limit state of equilibrium and that in the vicinity of sliding areas. If it is in cylindrical shape (case a.Figure 1) interaction does nor exist, as for limited movements there are no major changes in pressure distribution on contact.

If the sliding area is not of cylindrical shape,but of undefined shape, (figure 1.b) to every position of the sliding body corresponds a different pressure distribution. Interaction exists between the sliding body and the base.As a result of different kinds of supporting of the sliding body on the base major changes of stresses appear in the sliding body on occasion of each of its movements, even the slightest.

In the first case,sliding areas may be treated as relatively well defined (a)and in the second case as badly defined in regard to their way of reposing.

2. Group

Direct or superficially reposed structures.

Diagrams of representative structures of this group are shown on figure 2.

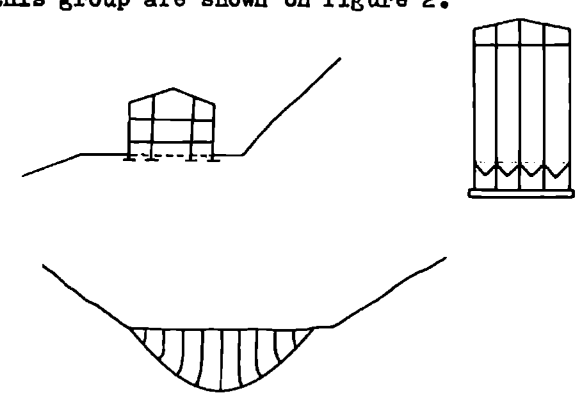


Fig 2

As regards the stress state in the base as well as on the contact surface,we are usually far from the limit state. Usually we are in the area of elastic behaviour of the semi-space and only somewhere in the area of elastoplastic stage,that means the structures are in the state of equilibrium,far from failure.

As such structures have their own rigidity which according to its deformation characteristics, for separate load cases,does not correspond to the deformation characteristics of the base, nearly always interaction appears.

This kind of structures are most frequently well defined as regards their support,thus the lines or surfaces of distribution may be calculated relatively exactly and the influence of interaction should be taken into account.

3. Group

Laterally pressed structures,and structures, superficially reposed

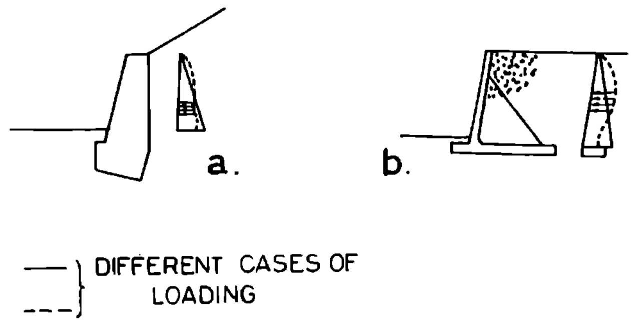


Fig 3

Typical representatives of such structures are shown on figure 3. In the semi-space surrounding the structures,stresses are partially in the limit state (lateral parts) and partially in elastic or elastoplastic state. The lateral pressure diagrams may be of different shapes,primarily depending on initial state of stress, on the deformational characteristics and the method of work execution. Pressure diagrams below the base depend mainly on the same parameters as the structures of Group 2.

Therefore interaction always exists and sometimes it is variable depending on time and other conditions. Applications of more exact estimates are sometimes possible, and often have to be accepted assumptions of simplification. Then the defining of structure regarding the reposing is considerably weaker.

Group 4

Structures supported in the media by three sides

Characteristic diagrams of such structures are shown on Figure 4.

Stress states in media surrounding the structure may be very different. Partially there may exist at limit state and partially in a state of equilibrium. That depends on the kind and hardness of the media materials, on the structures' shape, its strength, as well as on many other parameters.

Interaction exists, but the kinds of calcu-

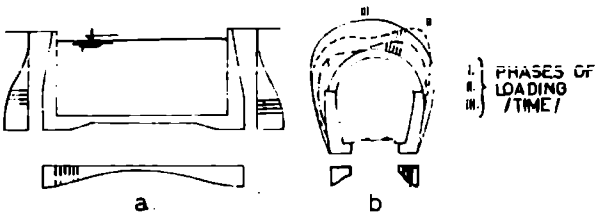


Fig 4

lations taking into consideration interaction are often very complex and complicated. Especially regarding rocks.

This kind of structures are, because of their complexity regarding the manner of the support, badly defined. Pressure distribution depends on a very great number of parameters.

In these cases it is convenient to know in advance the primary stress state of the medium that is surrounding the structure as well as the stress state during the construction and execution and later on, too. Then, to fix the zones of stress release depending on the type of material, on the kind of work and on weather conditions, provided that such phenomena appear. In some cases, there may also appear a commencement of local flowings.

Therefore the estimates of such structures are loaded with various assumptions of simplification that lead us to very approximate solutions.

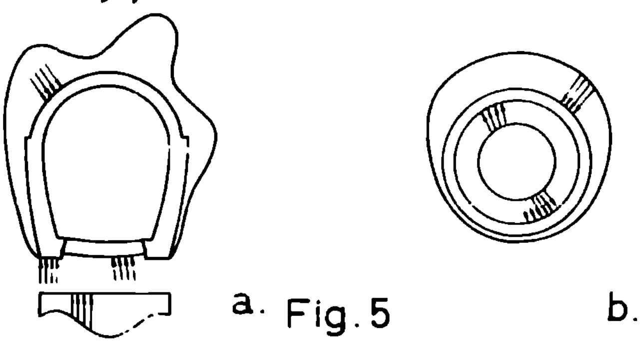
5 Group

Structures entirely encircled by media

Such structures are mainly traffic or hydrotechnical tunnels as shown on figure 5.

Concerning stress states, structures of this group are very similar to structures of group 4, if there is the question of structures similar to those shown in figure 5a.

Concerning structures shown on figure 5b, if there still appears an inner pressure, they have to be treated in a two different way. While at structures 5a interaction appears in the same way as at the other groups, at structures 5b, one has to take onto account still



a. Fig.5 b.

the inner pressure. The result of such interaction is then doubled, depending on the manner how the tunnel is loaded by water.

Such structures are very complex, especially because very often with them, as well as with the structures of group 4, loads are not known, which among others depends, on many parameters, time and other factors.

On the basis of the above-mentioned, you can see the multitude of problems which still have not found an approximate solution and which we should analyse and solve in the future.

The given structure division may however serve as a signpost for further work.

I thank you for your kind attention which you have given to my comments and want to excuse myself for the rather bad reproduction of the shown diagrams.

Written Contributions

L.ŠUKLJE (Yugoslavia)
On the paper "A Unifed Theory for the Consolidation of Clays" by J.G.Howley and D.L.Borin, Prof.Vol.1, Part 3, pp.107-119.

The "unified theory" as developed in the paper represents the numerical solution of the well known Terzaghi differential equation of one-dimensional consolidation of saturated soils accounting for a certain dependence of the coefficient of permeability on void ratio $k=k(e)$ and for a given rheological relationship $R(e,\dot{e},\sigma')=0$.

Examples of such solutions have been given in the paper by Šuklje and Kogovšek (1968), in the book Rheological Aspects of Soil Mechanics (Šuklje 1969) and in the paper by Šuklje (1969). In further publications the numerical procedure has been applied to the radial consolidation (Šuklje and Simončič 1972) as well as to the radially symmetric space consolidation (Šuklje and Kozak 1972). For references see the paper by D.Battelino in vol.1, part 1, pp.25-30 of the Proceedings of the present Conference and my short report on the "Use of Isotaches in the Consolidation Analysis" in the Specialty Session No.2 of the present Conference. Some results of a recent study on the "One-dimensional Consolidation of Parly Saturated Viscous Soils" have been presented in my oral discussion in the same Specialty Session.

In all above solutions the coefficient of permeability has either been taken constant or the terms containing the differential quotients of the coefficient of permeability with respect to the space-coordinates have been consciously neglected. This simplification of the numerical procedure had been allowed because the main purpose of the solutions was to clear up the influence of other factors upon the consolidation process, such as the length of the seepage path (layer thickness), the magnitude and the speed of the load increase, the initial void ratio (the rate of volume change respectively), the order of magnitude of the permeability and the saturation degree. The dependence of the permeability on the void ratio can, of course, fully be considered; some difficulties do appear, however, they are not essential.

Hawley and Borin have cited my paper presented in 1957 to the 4th International Conference where the suggestion had been given to represent the experimentally obtained stress-volume strain-time relationships by isotache sets and where the isotaches had been applied for the one-dimensional consolidation analysis of saturated soils (later- Šuklje 1964 and 1966- also for partly saturated soils). The analysis was related to the average void ratio change and pore pressure dissipation by using an appropriate assumption concerning the form of isochrones. The authors do not seem to be acquainted with further development of the isotache method as described in the previously mentioned publications.

The construction of a reliable isotache set is hindered by the sample disturbance during sampling, after sampling and during the load application, as well as by the need of extrapolating the consolidation curves beyond the time of the laboratory observation. All we can do is to make use of a good sampling technique, to transport and keep samples with care, to apply a slow, continuous loading interrupted by long intervals of the observation of the secondary consolidation, and to extrapolate the settlement versus time plots by taking into account the density and the history of the soil in the depth of sampling and in greater depths. If even when doing so, the initial volume change speed of the "undistrubed" sample at the "natural" water content is observed to be rather high, we shall approach the real consolidation process in a better way by starting at a lower void ratio corresponding to the probable speed at the end of the previous secondary consolidation than by starting at the "natural" void ratio at a greater speed which is due to disturbing effects. The consequences of the disturbance affect the reliability of any consolidation analysis and influence the consolidation process of thin and of thick layers, though in a different manner. The importance of the disturbance is only more evident when analysing the consolidation of a thick layer by taking into account its viscous behaviour.

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In recent years some attention was given by many Authors to the determination of the influence, in the general problem of soil-structure interaction, of the inhomogeneity on the vertical surface settlements of the isotropic, linearly elastic half space, see for example Brown and Gibson (1972). In particular, consideration was given to the case of a constant Poisson's ratio (ν) and a Young's modulus (E) increasing linearly with depth according to

$$E(z)=E_o+E_n z= E_o(1+\frac{z}{\beta b}) \qquad (1)$$

where

E_o= Young's modulus at the surface (z=0)

E_n= rate of increase of E with depth z

$\beta = \frac{E_o}{E_n b}$ = inhomogeneity index

b = width of loaded area

The numerical results allow one to recognize the following main patterns of behavior:

1. for β=0 and ν=0,5, the surface settlement profile corresponds exactly to that predicted by the Winkler subgrade reaction theory, Gibson (1967).

2. for $\beta \simeq 0$ and $\nu<0.5$, the surface settlement is quite completely confined under the loaded area (b.c) (Figure 1). In this case, the Winkler theory will still remain valid, for soil-structure interaction studies, at least as long as the surface displacement is of interest.

3. for $\beta > 5$ and all values of ν, the surface settlement nearly approximates that predicted on the basis of the Boussinesq's theory for the ideal homogeneous, isotropic, and linearly elastic half space.

In view of the above remarkable influence of inhomogeneity upon the magnitude and shape of surface settlements of the half space, the problem of evaluating the stresses and displacements due to the excavation of a vertical cut, in similar conditions for the soil, was examined. Due to the lack of a closed form solution, the finite element method was used according to the geometry illustrated in Figure 2. Plane strain conditions are assumed throughout the calculations with a Poisson's ratio equal to 0.33. The specific gravity is 2 t/m^3 and the modulus E_o 1500 t/m^2.

The Young's modulus increase with depth is now defined as

$$E(z)=E_o+\frac{E^x}{H} z=E_o(1+\frac{z}{\beta}) \qquad (2)$$

where

E^x= increment in the value of E, with respect to E_o, at z=H

$\beta = \frac{E_o}{E^x}$ = inhomogeneity index

H= height of vertical cut

The results obtained demonstrate the influence of soil inhomogeneity of the displacements and stresses due to the excavation of the vertical cut. The following preliminary observations can be made:

1. the horizontal displacements are influenced by the degree of inhomogeneity (i.e. the value of β). In particular for $\beta<1$ the point C at the crest of the cut moves toward the excavation; further, the ratio of the horizontal to the corresponding vertical displacement is affected by the value of β. A similar trend of behavior is followed by the point D, located along the wall (Fig.3).

2. for $\beta \leq 1$, both the vertical and horizontal displacements of the point C, when compared to the corresponding displacements for the homogeneous medium ($\beta = \infty$), are quite small. Further, for $\beta > 100$ the numerical results show that the behavior is similar to that predicted under the assumptions of homogeneity (Figure 4).

3. the displacements in the soil are depicted in Figure 5, where different scale factors have been used so that the phenomenological

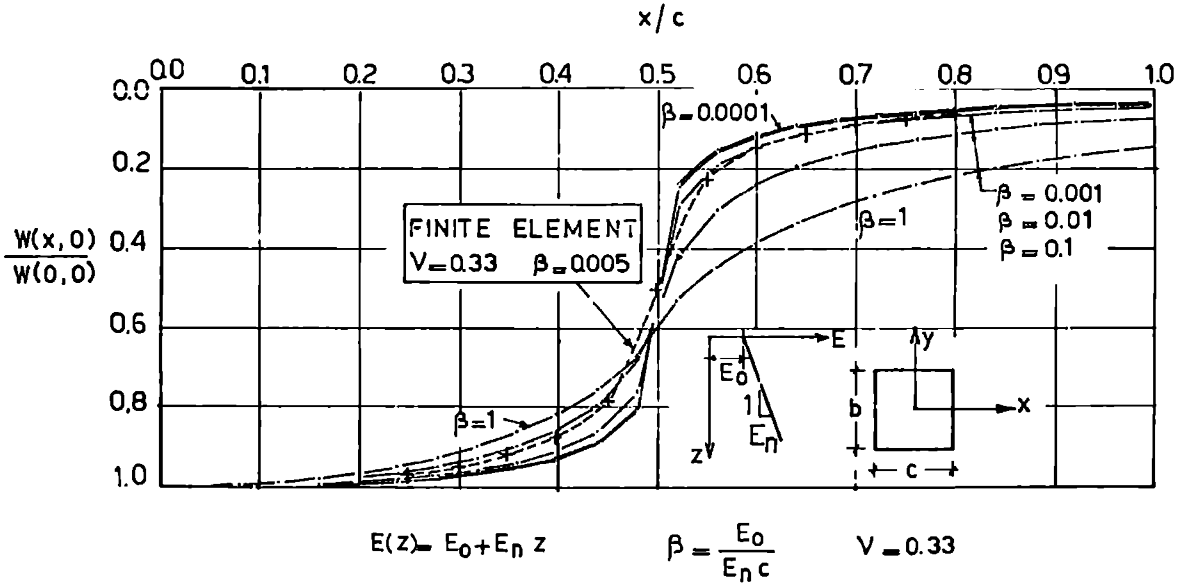

Fig.1. Settlement under a rectangular uniformly loaded area

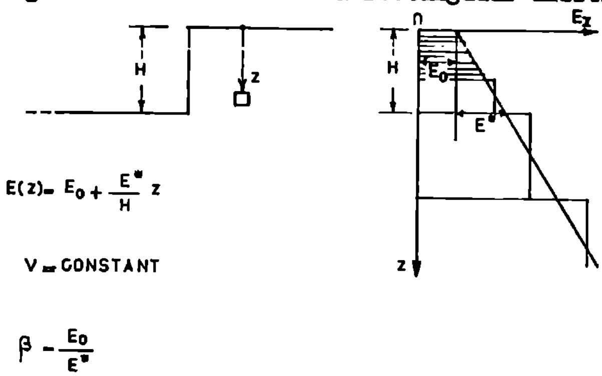

Fig.2. Assumed increase of the modulus E(z) with depth

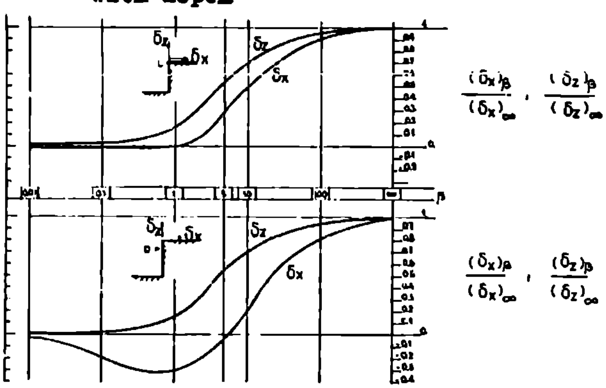

Fig.4. Horizontal and vertical displacements of points C and D for different values of β, referred to the corresponding displacements for the homogeneous medium ($\beta = \infty$)

aspects of the solution can be properly evidenced. The influence of β on the values obtained for the displacements is quite apparent.

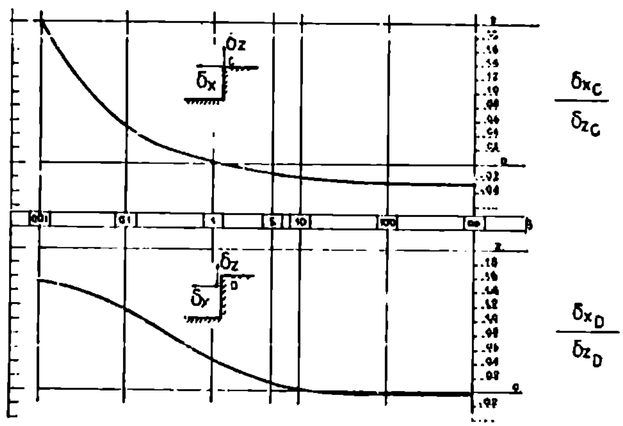

Fig. 3. Horizontal displacements of points C and D for different values of β, referred to the corresponding vertical displacements

4. the stress distribution in the region near the vertical cut is as well dependent upon the degree of inhomogeneity. As an example, let one consider the change in the vertical stress σ_z due to the excavation. According to Figure 6, the zone where no change occurs is more extended the smaller is β. In particular, for $\beta \leq 1$ an increase in the σ_z stress takes place at the wall bottom.

It is thought that the results reported deserve some attention in the analysis of flexible sheet-piles, mainly when consideration is to be given to the soil-structure interaction problem. It should however be noticed that the assumptions of linear elasticity and isotropy, introduced in the present study, might limit the practical relevance of the above remarks.

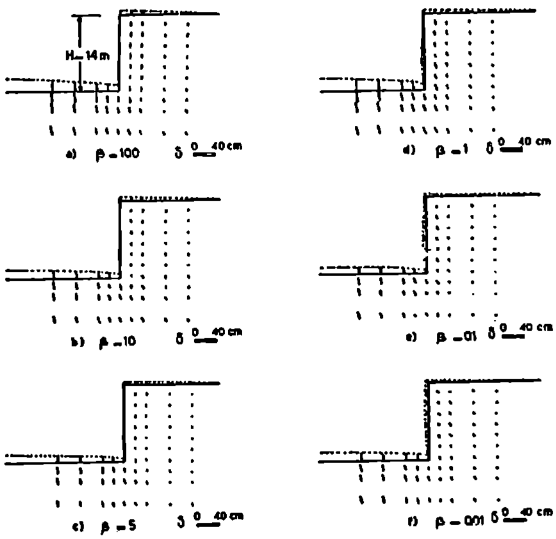

Fig.5. Schematic representation of the displacements due to the excavation for different values of β (notice that different scales are used for displacements)

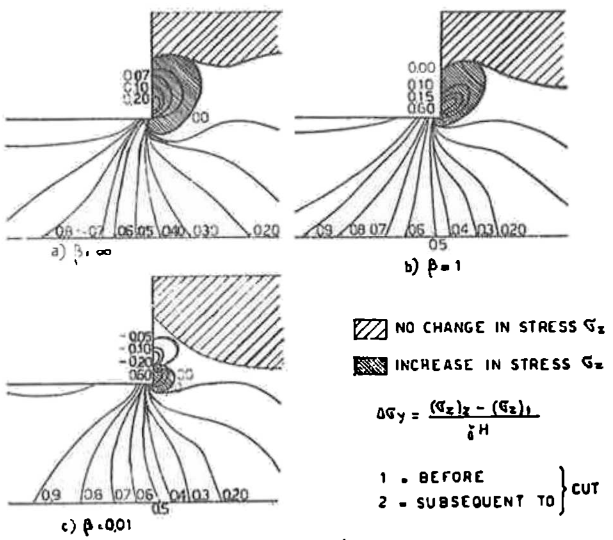

Fig.6. Change in stress σ_z due to excavation of vertical cut

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R.S.CHELIAPINE (URSS),

"L'influence de la composant peripherique de la resistance reactive du fondement sur les tensions et le deformations des pontres de fondation"

Les experiences faites sous la direction de l'auteur par l'ingenieur Kalouguine P.J. sur la mesure des tensions dans le corps du modele d'une haute poutre et des tensions de contact sous sa semelle,ont permis d'etablir une distinction importante de la repartition des valeurs mesurees des tensions horizontales σ_x de celles calculees theoriquement comme on le voit sur la figure 1.

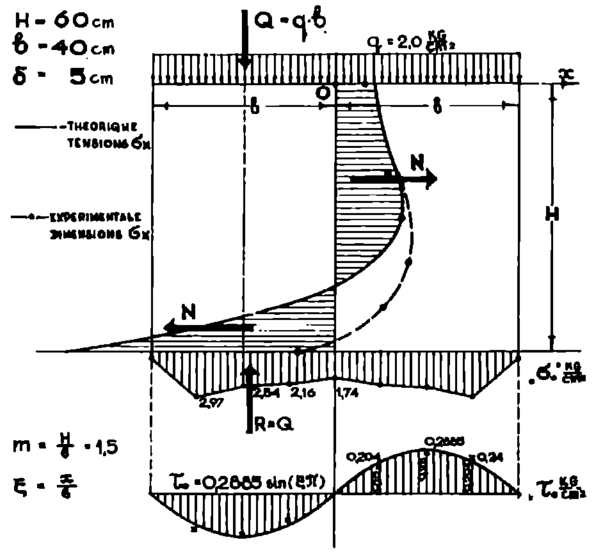

Fig.1. L'influence de la composant peripherique de la resistance reactive du foundement sur les tensions et les deformations des pontres de fondation"

Si on ne tient compte que de la composante verticale de la reaction du fondement σ_0 presentee sur la figure 1 selon les donnees de l'experience, les tensions horizontale σ_x d'apres les conditions de l'equilibre doivent former un couple N dans la section axiale car les charges verticales appliquees a la poutre Q et forment aussi un couple. La calcul theorique des tensions σ_x execute par l'ordinateur electronique dont on voit les donnees sur la figure 1 satisfait a cette condition. Pourtant les tensions reelles mesurees σ_x ne forment aucun couple et par cela meme les conditions de l'equilibre ne sont plus observees. Elles seront satisfaites si on introduit dans le calcul une force horizontale supplementaire qui n'est qu'une resultante des tensions de cisaillement de contact sous la semelle de la poutre. On a au la possibilite de mesurer ces tensions pour le modele d'une haute poutre ayant place dans la zone de contact de la poutre des "rosettes" tensometriques (sous l'angle 45^0). Comme on voit sur la figure 1 on peut considerer ces tensions de cisaillement de contact reparties avec une precision suffisante sur la sinusoide - $\tau_o = A_o \cdot \sin(\xi\pi)$ ou $\xi = \frac{x}{b}$

Alors en partant de la resolution approximative du probleme ils est possible de determiner les tensions supplementaires qui surgissent dans la poutre a condition de tenir compte de l'existence des tensions de cisaillement de contact reparties sous la semelle de la putre suivant la loi sinusoidale. Conformement a cette resolution les tensions horizontales σ_x peuvent etre determinees d'apres la formule suivante:

$$\sigma_x = A_o \frac{1+\cos(\xi\pi)}{m\pi \cdot sh(m\pi)} \cdot \left[m\pi\eta \cdot ch(m\pi\eta) + sh(m\pi\eta)\right]$$

$$\text{ou} \quad \xi = \frac{x}{b}; \; \eta = \frac{y}{H} \quad \text{et} \quad m = \frac{H}{b}$$

La plus grande valeur de cette tension σ_x est sur la semelle de la poutre le long de son axe quand $\xi = 0$ et $\eta = 1$

$$\sigma_x (x=0, y=H) = A_o \cdot 2\left[\frac{ch(m\pi)}{sh(m\pi)} + \frac{1}{m\pi}\right] = A_o \cdot 2 \cdot \alpha_m$$

ou le coefficient α_m dependant de m a les valeurs.

m=2,0	α m=1,159	m=0,5	α m=1,727
m=1,5	α m=1,212	m=0,3	α m=2,419
m=1,0	α m=1,322	m=0,1	α m=6,470

Sur la figure 2 sont presentes les resultats des calculs effectues pour la poutre qui a ete eprouvee par l'ingenieur Kalouguine. Ces resultats confirment une influence importante de la composante peripherique de la resistance reactive du fondement sur les tensions et les deformations des poutres de fondation.

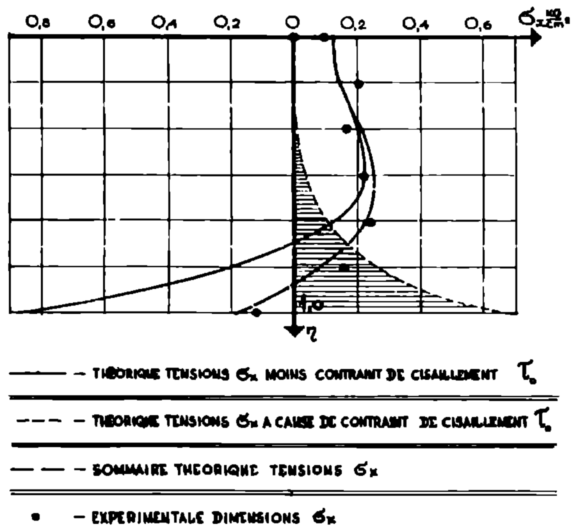

Fig.2. L'influence de la composant peripherique de la resistance reactive du foundement sur les tensions et les deformations des poutres de fondation"

Le materiel experimental presente et son interpretation attestent qu'on ne peut pas negliger la composante peripherique des tensions de contact pour le calcul des constructions sur le sol.

Cette resistance influe non seulement sur les tensions dans la poutre meme mais sur ses deformations et par consequent sur la repartition de la composante verticale de la reaction du fondement.