SPECIALTY SESSION 5

LATERAL PRESSURE OF CLAYEY SOILS ON STRUCTURES

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Chairman Dr.G.P.Tschebotarioff (USA)

Comprehensive reviews of most of this topic have beein given at our last International Conference (Peck,1969) and at Specialty and Regional Conferences since then,e.g., Am.Soc.Civ.Engrs.Conferences at Cornell Univ. (1971), at Purdue Univ.(1972) and at the 5-th European Conf. (1972) at Madrid.

My remarks which follow will concentrate on points related to design problems in practice which still remain controversial and which are debatable in connection with relevant new developments. I will leave to my colleagues, the vice-chairmen of our Session, the outlining of theoretical new contributions in this field.

A. Definitions

1. How should one Define a "Clayey" Soil in Practice? I propose referring this point to the Committee on Terminology of our Society.

2. How should the term "Arching" be used? At the Madrid Conference it was proposed (Bjerrum,Clausen & Duncan,1972) that a 1945 definition be accepted as a point of beginning. It read:-"transfer of pressure from a yielding mass of soil onto adjoining stationary parts is commonly called the arching effect". However,in my opinion,from the very start one should realize that the above definition lumps together two completely different physical phenomena,namely,the shearing resistance of soil to sliding and the wedging of its hard grains (Tschebotarioff,1951,1952, 1973).Differentiation between the two phenomena is essential to permit further more refined theoretical studies and analysis of research measurements. It seems to me that the term "arching" should not be used in connection with plastic clays and should be applied only to cases involving not too loose sandy soils which produce coefficients of lateral pressure K close to,— or even greater than,— unity as a result of the wedging of hard grains into an arch,once its supports are immovable.

In a June 1972 conversation with Dr.Bjerrum I told him of my above coming critique and,at his request, promised to invite him to comment on it at our Specialty Session 5 at Moscow. On learning with sorrow of Dr. Bjerrum's sudden death on 27 Febr.1973 my first impulse was to drop entirely the subject of "arching" from my Introductory Remarks. However,on reflection,knowing his fine character, I formed the conviction that Dr. Bjerrum would never have wanted his untimely departure from our midst to delay clarifying debates of controversial matters in a field to which he has contributed so much in his all-too short lifetime. I am glad that one of his two co-authors, Dr.Duncan,will discuss it for him.

B.Semi-Empirical Design Methods

3. Possible Improvement of the "Strength" Design method for the Bracing of Cuts. The well-known Terzaghi-Peck lateral earth pressure trapezoid was developed in 1943 to serve as an envelope for strut loads measured on the 44 ft. (13.4m) deep excavations for the subways then under construction in Chicago.

A precursor of Eq.(1) was used to relate the trapezoid dimensions to the undrained shearing strength s_u of the clay,employing the Rankine-Resal theoretical concepts as a point of departure.

It was however shown that for smaller depths zero pressures were indicated by that equation, whereas definite pressures were actually measured in the field. It was therefore concluded that the reverse relationship was likely at depths greater than 44 ft (13.4m) where the trapezoid might give too high and hence uneconomical values (Tschebotarioff, 1951). The validity of the first part only of the above critique,referring to small depths, was acknowledged at our last International Conference (Peck,1969).

Since then measurement results in a 70 ft (21.3m) deep cut in Chicago clay have been published (Swatek,Asrow & Seitz,1972),showing that the envelope of pressures producing the measured strut loads nowhere exceeded appr. 60% of values given by the Terzaghi-Peck trapezoid–, see their Fig.12, p.1318.

To bring that trapezoid into agreement with the results of measurements in very deep soft clays of Mexico City and of Oslo,Norway the coefficient "m" was introduced (Peck, 1969) into the original equation,which was modified to read:—

$$K_A = /1-m(4s_u/ \gamma H)/ \qquad (1)$$

on which the maximum lateral trapezoid pressure ($K_A \gamma H$) depended. For Mexico City and Oslo clays the value of that coefficient was to be set at m=0.4 and at m= 1.0 elsewhere.

It now appears that,if design pressures are to be based on Eq.(1),the value of the coefficient "m" should be varied with the excavation depth at any location. Thus, in Chicago clays, if it was originally found that m=1.0 for excavation depths of 44 ft. (13.4m), then one should set m=2.0 at least for a depth of 70 ft. (21.3m), assuming that for both depths γ=120 lbs/ft^3 (1.92 gm$_f$/cm^3) and s_u=700 lbs/ft^2 (3,420 kg$_f$/m^2).

4. Application of the "Neutral Earth Pressure" Method to the Design of Braced Cuts. A method has been proposed (Swatek,Asrow & Seitz,1972) to estimate the actual strut loads from a conventional triangular earth pressure diagram and a neutral value of the active coefficient K=0.5. It is illustrated by their Fig. 11, p.1317 and simulates the construction procedure followed in the field. The cross-hatched pressure areas are added to the strut loads conventionally obtained from the K_o triangular diagram to represent additional loads received by each strut at the time excavation reaches the level of the next strut,but before that strut has actually been installed. It can be seen from their Fig.12, p. 1318 that the strut load pressure envelope computed in this manner closely agreed with the one actually measured in the 70 ft (21.3m) deep Chicago excavation.

The validity of this procedure for cuts in other types of clay has not yet been checked.

The factors which influence the values of the so-called "neutral", "at rest" or "consolidated equilibrium" lateral earth pressure coefficients also are in need of clarification.

A slight expansion of a consolidated clay mass as a result of a lateral yield of its supports will reduce its lateral pressures, which will however gradually build up again almost,but not quite,to their value before the expansion. This is illustrated by Fig.1 (E.H.Ward,1956).The drained test was performed in a cell device described elsewhere (Tschebotarioff,1957) on an 8 in (20cm) diameter and 17 in. (43cm) high sample of Chicago clay which had the following characteristics: w_L=28%, I_p=10%, w=26%, s_u=400 lb/ft^2 (1,960 kg$_f$/m^2). It was taken at El.-6.5' of Chicago City Datum. The K_o values shown on Fig.1 were obtained under an axial load of σ_1=1,300 lb/ft^2 (6,350 kg$_f$/m^2) after completed consolidation, followed by an induced 0.003 radial unit strain.

There is reason to believe that the K_o values will be higher for clays with higher I_p values,for remolded but not yet reconsolidated clays of the same type,and for increased ratios of σ_1 to the preconsolidation pressure of a clay.

The complexity of "cell" tests on undisturbed samples of clay is such that further progress can be expected mainly from field measurements,— see item (7) below.

5.Physical Causes of observed Pressure Redistribution in Braced Cuts.It was shown over 30 years ago (Skempton & W.H.Ward,1952)that consideration of structural continuity effects of the sheeting in a 32 ft (9.8m)deep cut with 3 rows of struts could fully account for measured strut load departures from a triangular distribution of lateral clay pressures. There was therefore no need to assume "arching" in clay to account for observed apparent pressure redistribution, —in this connection see my comments about "arching" above under (2).

To check the effect of sheeting continuity for the case of the 70 ft (21.3m) deep cut in Chicago referred to above and which had 6 rows of struts and inclinometer measured lateral soil displacements reaching a maximum value of 2.3 in. (58.5mm),an unpublished analysis by Professor Joseph J.Gennaro of the Stevens Institute of Technology was made at my suggestion. It did not establish any practically usable relationship between the envelope of measured strut loads and the lateral soil displacements measured in a single inclinometer borehole, largely because continuous sheeting with 7 supports is very sensitive to small differences (of the order of 0.2 in.=5mm) in the lateral displacements of individual struts.

It therefore seems that we will have to continue to rely upon semi-empirical methods for strut design.

6. "Neutral" pressures of Overconsolidated Clays. The following equation has been proposed by Sherif (1971) for the estimation of the coefficient K_o of overconsolidated clays:

$$K_o = \lambda + \alpha(P_r - 1) \qquad (2)$$

where λ and α are functions of the liquid limit, and P_r is the overconsolidation ratio as determined from conventional consolidation tests.

The high values of K_o in overconsolidated clays reflect the "locked in" lateral stresses of such deposits and serve to explain the basic source of much trouble encountered in the Seattle area due to lateral creep of retaining structures and slides of highway cuts (Sherif & Wu,1971). Consideration thereof is also useful in tunnel design (Sherif & Strazer, 1973).

c.Field Measurements for Improvement of Design Methods

7. Direct in-situ Earth Pressure Measurements. Two important relevant developments

have been reported recently, -one in France and the other in Norway.

The first was the design of the "sonde autoforeuse". This is a lateral pressuremeter built into the walls of a borehole casing which can be sunk into the ground with a minimum of disturbance thereof. (see paper 1/3 of our Conference Proceedings by Baguelin, Jezequel & LeMehaute). It appears that the new instrument has overcome a basic flaw of the earlier pressuremeters which permitted some remolding of the clay around the borehole walls as the hole contracted when the casing was pulled up to permit operation of the pressuremeter A similar instrument has been described in the paper 1/75 of our Conference Proceedings by Wroth and Hughes of the United Kingdom. With the new "autoforeuse" type of pressuremeter probe direct in-situ determination of the K_o values of undisturbed natural clay deposits now becomes possible. This has also become possible by a quite different method, that of hydraulic fracturing, which was first developed by geologist for rock studies and then applied in Norway to clays (Bjerrum & Andersen, 1972b). Water is forced under pressure into the bottom of the borehole. Vertical fissures develop in the clay when the fluid pressure exceeds the natural lateral pressure in the deposit. It is possible to determine the K_o value when the water inflow into the hole suddenly decreases as the fluid pressure is decreased and the fissures close.

In a June 1972 conversation with the late Dr.Bjerrum and his aide Dr.DiBiagio, I suggested that the "autoforeuse" method be tested in Norway on the sites already investigated there by the hydraulic fracturing method be similarly tried out in France. We will shortly hear reports on this topic.

I received the following relevant note from Professor Stanley D.Wilson, Chairman of Specialty Section No.1:--"In my Introductory Remarks at Session 1, I explained that I did not propose to include the determination of the in-situ stress of natural soils in that Session. Therefore I agree that this topic is best covered within the scope of Session 5. However, we did include in our Session the subject of pressure cells to record changes in stress resulting from construction operations".

I hope that new devices, or improved old ones will be developed, which could be inserted into the soil mass and left there to record variations of lateral pressures before, during and after construction. It seems to me that only after such studies become possible can we expect significant improvement in our present understanding of the basic factors on which our design procedures should be based.

8. The Use of Adjustable Temperature Compensated Struts in Braced Cuts on a research section of the Lyon subway has been reported by Kerisel et al.(1972). Although the cut was in sand, this important refinement in the measurement techniques of lateral pressures under full-scale field conditions is equally applicable in clays.

D. Lateral Pressures on Piles

9. Use of the Modulus of Subgrade Reaction. A large number of relevant measurements on piles subjected to structural lateral loads applied at the soil surface has been published in recent years, e.g., at the Madrid Conference. However, no comprehensive correlating review of the many results obtained under varying soil, load and pile conditions appears to have been made as yet from which generally valid rules for use in designs could be derived.

10. The Backward Tilting of some Pile-Supported Bridge Abutments has been shown to be caused primarily by lateral pressures of plastic clay layers on the piles under the abutment heel. Movements were found to begin when the weight of the embankment fill behind the abutment exceeded the value of $3s_u$,--where s_u is the shearing strength of the clay layers--- causing plastic shearing deformations of these layers (Tschebotarioff 1970, and Nicu, Antes & Kessler, 1971). These findings were confirmed by a comparative analysis of a number of other similar cases (Marche & Lacroix, 1972).

11. The Design of Piles for Flexure under the Action of Lateral Clay Pressures is necessary in such cases. The conventional procedure of considering only the forces acting on the abutment above the bottom of its footing is then inadequate and should be abandoned (Tschebotarioff, 1970, 1973). Two methods have been proposed for the estimation of clay pressures in such cases,— one by Tschebotarioff (1971 in discussion of Nicu, Antes & Kessler, also in 1973),— and another by DeBeer & Wallays (1972).

A new approach has been developed by Marche (1973). He will present it to us shortly.

REFERENCES

Baguelin, F., Jezequel, J.-F., Le Mee, E. & Le Mehaute, A (1972). Expansion of Cylindrical Probes in Cohesive Soils. J.Soil Mech. Fndt.Div.Am.Soc.Civ.Engrs., 98, SM11, pp.1129-1142.

Bjerrum, L., Clausen, C.J.F. & Duncan, J.M. (1972a) Earth Pressure on Flexible Structures. Proc. 5th Eur.Conf.Soil Mech-Fndt.Engng., Madrid, State-of-the-Art Report.

Bjerrum, L.& Andersen, K.H. (1972b). In-situ Measurements of Lateral Pressures in Clay . Proc.5th Eur.Conf.Soil Mech.Fndt.Engng.Madrid

DeBeer, E.E., & Wallays, M. (1972). Forces induced in Piles by Unsymmetrical Surcharges on the Soil around the Piles. Proc.5th Eur.Conf.Soil Mech.Fndt.Engng.Madrid.

Kerisel, J. et al. (1972). Mesures de Poussee et de Butee faites avec 42 paires de butons asservis. Proc.5th Eur.Conf.Soil MechFndt. Engng.Madrid.

Marche, R.& LaCroix, Y. (1972). Stabilite des Culees de Ponts etablis sur de Pieux traversant une Couche Molle. Canad.Geotech J., vol.9 No.1.

Marche, R. (1973). Pieux sollicites en Flexion par les Couches qu'ils traversent. These de doctorat, Ecole Polytechnique de Lausanne.

Nicu,N.D.,Antes,D.R. & Kessler,R.S.(1971). Field Measurements on Instrumented Piles under an Overpass Abutment. Highway Res.Record 354, pp.90-101.Washington, D.C.

Peck,R.B.(1969) Deep Excavation and Tunneling in Soft Ground,Proc 7th Intern.Con.Soil Mech. Fndt.Engng.,Mexico, State-of-the-Art Volume, pp.225-290.

Sherif,M.E.& Wu,M.J.(1971).Summary and Practical Implications of University of Washington Soil Engineering Research (1965-1970), Univ.Wash.Soil Eng.Res.Rep.7,Seattle.

Sherif,M.A.& Strazer,R.J.(1973).Soil Paremeters for Design of Mt.Baker Ridge Tunnel in Seattle.J.Soil Mech. Fndt.Div.Am.Soc.Civ. Engrs.99, SM1,pp.111-122.

Skempton A.W.& Ward,W.H. (1952).Investigations concerning a Deep Cofferdam in the Thames Estuary Clay at Shellhaven. Geotechnique, p.119-139.

Swatek,E.P.,Asrow,S.P. & Seitz,A.M.(1972). Performance of Bracing for Deep Chicago Excavation. Proc. ASCE Spec.Conf.Perf.Earth & Earth Supp.Struct.,Purdue Univ.,Vol.1, Pt.2,p.1303-1322.

Tschebotarioff,G.P.(1951).Soil Mechanics, Foundations & Earth Structures. McGraw-Hill 655 p.

Tschebotarioff,G.P.(1957).Discussion,Proc. 4th Intern.Conf.Soil Mech.Fndt.Engng.,London vol.III, p.239-241.

Tschebotarioff,G.P.(1970).Bridge Abutments on Piles Driven through Plastic Clay.Proc.Conf. Des.Instal.Pile Found.Cell.Struct.,Lehigh Univ.,p.225-238.

Tschebotarioff,G.P.(1973).Foundations,Retaining & Earth Structures. McGraw-Hill,642 p.

Ward,E.R.(1956). Triaxial 'cell' test No. C-16 on Undisturbed Chicago Clay. Part of Princeton University Progress Report to Office of Naval Research,Washington, D.C.August 2, 1956.

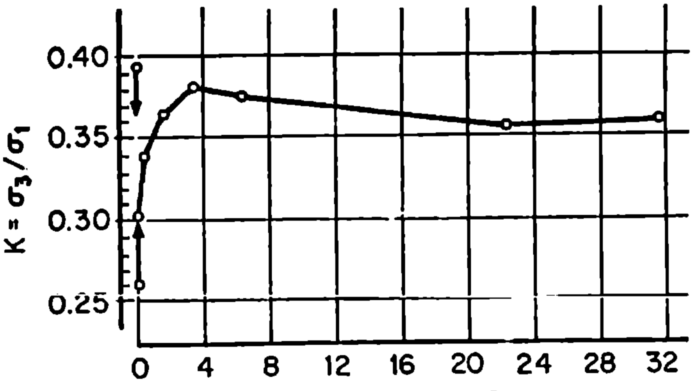


FIG. 1. (From E.R. Ward, 1956).

I would like to invite Prof.Klein (USSR), Vice-Chairman of our Session to make his report. Prof.Klein, will you please.

Prof. Klein G.K. (USSR), Vice-Chairman

Being limited in time, I'd like to attract your attention to the three problems suggested for the discussion.

The first problem deals with soil cohesion and moisture content during the determination of its pressure on structures. This question is of special economic importance during clay fillings since depending on the assumed designed values of internal unit cohesion and furthermore- cohesion of soil with a wall, extremely various results may be obtained for the designed structural loads. The way of cohesion analysis seems to influence the results considerably.

The problem in question needs additional theoretical and experimental analysis.

The second problem deals with the consideration of structure displacements while determining soil pressure on it, i.e.all the necessary considerations of the simultaneous activity of a structure, fill and footing. Experimental and theoretical investigations show quite clearly that it is not enough to calculate retaining walls and other retaining structures under the influence of the so-called active pressure since actual pressure may be much more. Besides,considerations of fill and retaining structures under ultimate balance state makes it possible to calculate the structures only considering stability and strength and gives no opportunity of the calculation according to displacements and crack resistance.

For the time being there are some suggestions on retaining wall design taking into consideration simultaneousness of their displacements and that of a filling and footing (N.K.Snitko, G.A.Dubrova, I.M.Besprozvannaja). These suggestions were published and commented in "Hydrotechnical Construction"journal in 1966-69.

Mechanical properties of real soils were most fully considered in the published solutions of Prof.F.M-Shikheev and his chair colleques "Water ways and ports" of the Odessa Institute of Navy Engineers (V.T.Bugaeva, M.N.Vargina, R.N.Lubeniy, P.I.Yakovlev, etc). The peculiarities of their suggestions in comparison with others are : the suggestion of the determination of critical displacement values to establish the boundaries of the areas with pre-ultimate and ultimate stress states of filling considerations of only effective wall displacements during back filling.

The third problem which is of special attention so far, deals with the usage in soil mechanics and retaining wall design especially of the methods of the theory of reliability which is nowadays greatly applied in structural mechanics and in the solution of other engineering problems.

The usage of this theory in soil mechanics may bring sufficient technical and economic effect especially concerning retaining wall design since the properties of wall materials, filling soils and footings are of random nature and of great dispersion. In this respect, V.N.Karavaev and the author (speaker) considered the problem on retaining wall creeping on clay footing, its strength changing in time. Reological Maslov's model was under investigation, and the creep process was analysed according to Markov.

The influence variations of random parameters typical of soil properties and the structural material upon its reliability and cost was under study. During the investigation as a minimum probability data which may be actually obtained during design, mean values of random parameters were used, i.e. mathematical expectation and mean square deviation methods. As for the laws of soil property distribution, it may be considered normal.

Chairman Dr.Tshebotarioff. G.P. (USA)

Thank you Prof. Klein. Now Prof. Malyshev, will you please.

Prof. Malyshev M.V.(USSR), Vice-Chairman

We have just heard the very interesting and concise speech of the Chairman of the present specialty session, Dr.G.P.Tschebotarioff, in which he dealt with many questions that are of importance to the design and construction of retaining walls.

The pressure of clayey, as well as sandy, soils is closely associated with the possible direction and magnitude of displacement of the retaining wall. Only when a limiting state is reached throughout the fill behind the retaining wall will we obtain either the "active" or "passive" pressure whose magnitude is calculated on the basis of the theory of limiting equilibrium of a loose medium. An intermediate state is one of "complete" immobility of the retaining wall which corresponds, for example, to the walls of locks of the dock type. We do not know, however, how to properly calculate the displacements which correspond to the beginning of a limiting state occuring throughout the fill behind the wall, i.e. when friction and cohesion in the soil are completely mobilized. It is especially important to take the cohesion into account for low retaining walls when the share of the pressure due to cohesion is commensurable with the share due to friction. Moreover, we are concerned with walls of limited height. Consequently, the pressure on them begins to decrease as we approach the base. Especial difficulty is encountered in designing flexible retaining walls. Practical proposals have been made to calculate the pressure on the basis of the Winkler model, using a modulus of subgrade reaction that increases linearly with the depth. Here a translatory displacement of a rigid wall provides for a linear diagram of the pressure acting on the wall. However, the question of determining the modulus of subgrade reaction experimentally is not clear. It is necessary either to standardize these tests, or to give recommendations, based on precise experiments, for determining this modulus. This is necessary because the modulus of subgrade reaction depends on the demensions of the testing plate used to determine it, and is not an invariant quantity.

A model based on a homogeneous medium is used in calculating the pressure on the walls. Actually, the soil of the fill is non-homogeneous and may even be anisotropic. Besides, the manner in which the fill has been compacted, i.e. the way the earth work has been performed, has a substantial effect on the pressure. The internal friction and cohesion in the soil are partially or fully mobilized with an increase in deformation. Moreover, this process develops with time, so that the problem become rheological as well. Theoretically, various design models and schemes can be devised. They may be of the simpler, engineering type or of a more complex type based on the solutions of two-dimensional problems. The results, however, of these theoretical solutions should be compared with the results of experimental observations. In making such observations it is necessary to determine the soil properties carefully so that the experimental data can subsequently be processed in various ways and then used to correct the theoretical solutions. It is also necessary to provide all information on apparatus and techniques used to determine the soil characteristics.

Since, unfortunately, we have very little experimental data available, the questions suggested for discussion in the present session concern the results of field observations on the pressure of clayey soils on retaining structures, and the way the cohesion in the soil is to be taken into account in determining the pressure.

Chairman Dr.Tschebotarioff, G.P. (USA)

Thank you Prof. Malyshev for your report. The next will be Mr. Duncan from USA, University of California, Berkeley.

Prof. J.M.Duncan (USA)

Dr.Tschebotarioff, ladies and gentlemen:

I'm sure we all wish that Dr.Bjerrum was here to carry on this discussion and to explain his concept and opinions concerning the term "arching". His tragic death of course makes that impossible. I will therefore do my best to explain our thoughts about arching when we wrote the General Report on Earth Pressures on Flexible Structures for the Madrid Conference.

Dr.Bjerrum, Mr.Clausen and I included Terzaghi's definition of arching in the Madrid report, so that it would be clear what we meant by arching. This definition is quite general, saying only that arching is the phenomenon by which the earth pressures on the yielding part of a structure are decreased while those on the adjoining nonyielding parts of the structure are increased.

I am sure that Dr.Tschebotarioff is correct when he says that this broad definition of arching includes more than one mechanism for pressure redistribution,namely sliding and wedging. I am sure he is also right when he suggests that there may be important differences between the mechanisms of arching in dense sands,and plastic clays.

However, I do not think that it is essential for progress in research that we should restrict the use of this term "arching" to the redistribution of pressures in sands which are "not too loose". It seems more reasonable to me that we should continue to accept the definition of arching as it is understood by most people in soil mechanics-reduction in pressure on the yielding parts of the structure and increase in pressure on the adjoining parts, without reference to the type of soil, because externally at least, arching is much the same in all types of soil.

There are undoubtedly important differences in the magnitude of the arching effect in different types of soil, in the permanence and stability of the arching effect,and in the relative importance of sliding and wedging in different types of soil. I am sure that Dr.Tschebotarioff is right when he says that we need further research for a better understanding of arching. However, I do not believe that the objectives of this research, which is a clearer understanding of arching, will be promoted by adopting different words for pressure redistribution in dense sand and other types of soil. I think that it will be better to continue to accept a broad definition of the term arching,and to make the distinctions later,when we have the results of the research to guide us.

Chairman Dr.Tschebotarioff,G.P.(USA)

Thank you Prof.Duncan for you discussion. Now I pass the word to Prof. Ralph Peck

Prof. Ralph Peck (USA)

Dr. Tschebotarioff has discussed the possible improvement of the so-called Terzaghi-Peck semi-empirical design method for estimating strut loads in braced cuts in clay. I am the first to agree that improvements are both possible and desirable. When the method was published in 1943, Dr. Tschebotarioff raised the objection that, for any of the cuts, the method predicted pressures that were too small when the cut was shallow, and pressures that agreed with the measurements only at the final depth of the cut. He was quite right, but I do not believe that the introduction of the coefficient "m" is the proper way to account for the discrepancy. After introducing the factor in Mexico City four years ago, I have altered my conclusions somewhat.

The terms "shallow and "deep" are, of course, only relative. The critical quantity is the ratio of overburden pressure to undrained shear strength. For small values, say less than 4, the stresses are in the elastic range and the semi-empirical procedure based on the plastic state should not be expected to apply. This is the case for "shallow" cuts.

For deeper cuts, an oversimplified approach that I now consider reasonable is based on two stability factors. One, $H_c = \gamma H/s_u$, is calculated for the average shearing strength s_u of the clay alongside the cut above excavation level. The other, $N_{cb} = \gamma H/s_{ub}$, which we might call the base stability factor, is calculated for the average shearing strength s_{ub} of the clay below excavation level. Both of these have the form H/s_u, where γH is the weight of the soil above excavation level and s_u is the undrained shear strength within the probable zone of plastic disturbance.

If the base stability factor exceeds 7 or 8, bottom heave is likely to be excessive and the strut loads are likely to exceed appreciably those predicted by application of the semi-empirical procedure. These conditions prevailed, for example, at the cuts cited by Dr. Tschebotarioff in Norway and Mexico. However, if the base stability factor is less than about 6 or 7, the strut loads usually agree reasonably with the predictions even if the stability factor for the soils above the base is as high as 10 or 12.

The measurements reported by Swatek et al, for the 70-ft cut in Chicago led Dr. Tschebotarioff to conclude that, for such a deep cut, the computed pressures should be reduced. Yet the cut on Chicago Subway Contract D8, reported by Wu and Berman, was 68 ft deep and developed strut loads in agreement with the semi-empirical prediction. Hence, great depth does not necessarily lead to an overestimate of strut loads if the semi-empirical procedure is used.

Chairman Dr.Tschebotarioff,G.P.(USA)

Thank you very much Prof.Peck for your comments.Now I want to invite Mr.Klein (Czecho slovakia)

Mr.Karol Klein (ČSSR)

Prof.Bjerrum et al./1972/ extended in Madrid last year an exquisite idea about the arching in vertical direction in soil behind the wall in the case of braced excavations /Fig.1/.He demonstrated that for the magnitude of loading of braced wall was decisive whether the deformations beneath the level of excavation were large or small.

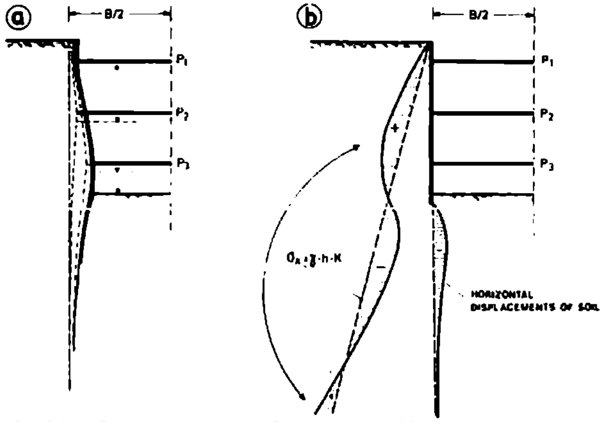


Fig.1.Displacements and the earth pressure redistribution for a braced excavation

For braced walls in the case of compressible soils below the excavation bottom were in fact measured considerable greater loadings than those responding to Rankine's theoretical active earth pressure.

For a braced wall the loading is transferred through the struts to the opposite wall. For an anchored wall the load from the wall is transferred into soil in the region of fixed anchors /Fig.2/ and from them by friction into the soil under the bottom of the excavation.

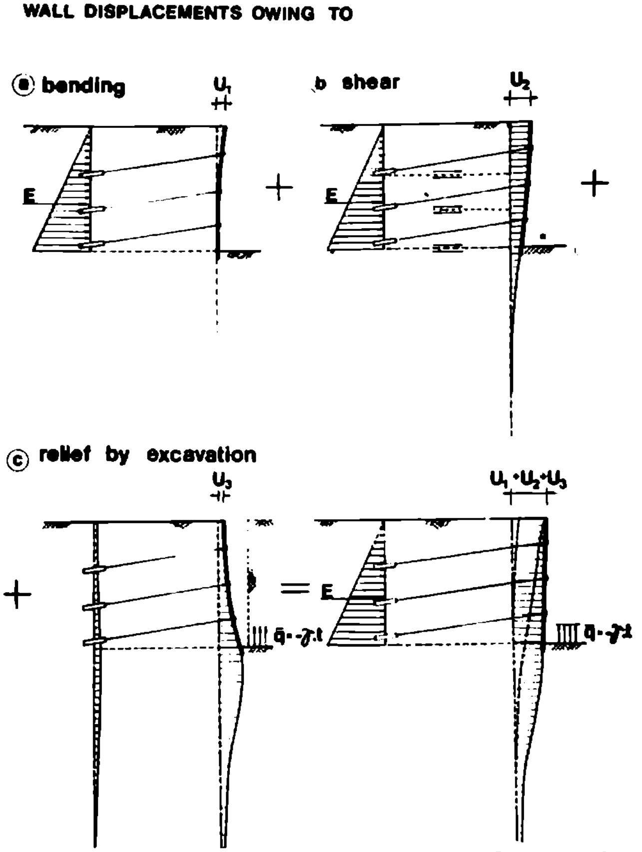


Fig.2. Displacements in the case of an anchored wall

The anchored wall displacement is caused by several factors:

1. by the earth body deformation braced between the wall and the fixed anchors. This body is loaded by prestressing of anchors and by earth pressure. The earth pressure deforms it by bending and especially by shear /Fig.2a,b/.

2. by soil compression beneath the bottom of the excavation owing to its horizontal additional load from the earth pressure which is transferred from the region of fixed anchors into the soil especially beneath the bottom of the excavation /Fig.2b/.

3. by the soil displacements towards the excavation owing to the halfspace relief equal to the excavation weight, which changes the existing equilibrium conditions /Fig.2c/.

The results of the high anchored walls displacement measurements show that the position of the wall free part after the excavation is nearly parallel with its original position and the displacements decreasing only a little towards the excavation bottom.

The displacements of a long wall can be approximately computed from the relation in Fig.3. The analysis design and its comparison with the results of measurements were processed together with Dr.Ing.Nendza /1973/ from Erdbaulaboratorium of Essen.

In the case of prestressed anchors stressed approximately by Rankine's active pressure for long excavations with depth of 15 to 20m the displacements reaching as many as 15cm were measured in Frankfurt and Hamburg, while beneath the bottom of the excavation there were high compressible cohesive soils. For little compressible soils under the bottom of the excavation for inst.in Düsseldorf and Essen /sands, sandstones/ the displacements up to 1,5 cm were measured.

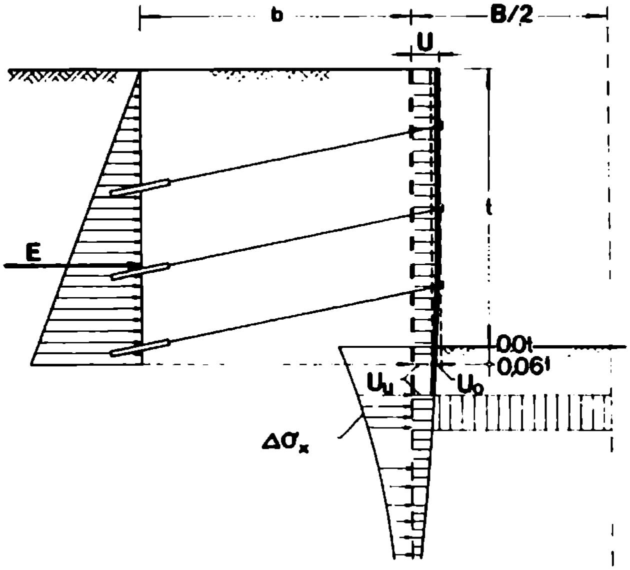
The displacement magnitude . of the anchored wall is in a decisive way influenced by the soil compressibility beneath the bottom of the excavation as well as by the structure size.

The displacement magnitude of the anchored wall is in a decisive way influenced by the soil compressibility beneath the bottom of the excavation as well as by the structure size.

The displacement of an anchored wall inwards of the excavation cannot be prevented when the anchor lengths are nearly equal to that of the excavation depth which prevailingly is sufficient for the wall stability computation. An increased anchor prestressing in these cases can only little reduce the displacements of the wall. In the case of prestress increased concentrates on the wall a load larger than that responding approximately to the earth pressure according to Rankine.

The displacements can be limited in a more important way by considerably skew and long prestressed anchors.

The measurement results show that the behaviour of an anchored structure is to be examined as whole. For the determination of the anchored wall loading it is difficult to apply simply the experience from the measurements of load upon the braced walls especially in cohesive soils.



HORIZONTAL DISPLACEMENT

$$U = U_u + U_o$$

$$\boxed{U = \frac{\Delta\sigma_x \cdot B/2}{E_e} + \frac{2 \cdot E \cdot t}{6 \cdot G_e \cdot b}}$$

WHERE:

E -EARTH PRESSURE

E_e -HORIZONTAL MODULUS OF ELASTICITY AT RELIEF

G_e -SHEARING MODULUS $G_e = \frac{E_e}{2(1+\nu)}$

ν -POISSON'S RATIO

$\Delta\sigma_x$ -CHANGE IN HORIZONTAL STRESSES IN THE WALL PLANE BENEATH THE LEVEL OF THE EXCAVATION

Fig.3. Design of analysis of horizontal wall displacements

REFERENCES:

Bjerrum L., Frimann Clausen C.J., Duncan J.M. (1972) Earth Pressures on Flexible Structures. Proceedings of the 5th European Conference on Soil Mechanics and Foundation Engineering, vol.II, Madrid 1972

Nendza H., Klein K.(1973) Bodenverformungen beim Aushub tiefer Baugruben. Conference No 81-73, Tunnelbau in offener und geschlossener Bauweise,Haus der Technik, Essen, 1973.

Chairman Dr.Tschebotarioff G.P.

Thankyou Mr.Klein for your discussion. The next will be Mr. Foss (Norway)
Mr.Foss will you, please.

Mr. Foss Ivar (Norway)

My discussion concerns the design of braced cuts in soft clay, particularly the use of semi-empirical design methods.

The design of braced excavations consists of a geotechnical and a structural design,but the complete design procedure is rarely considered in geotechnical literature. I have therefore, on behalf of the Norwegian consulting engineers Noteby, Norsk teknisk byggekontroll A/S,tried to establish a rational approach to this problem.

Fig.1 shows a breakdown of the problem into a geotechnical and a structural branch.The first step of the structural design is to calculate the total earth pressure acting on the structure. Then the distribution of the earth pressure on the wales should be evaluated,and finally variations from the average load should be considered in order to arrive at design loads for bracing,wales and shelting. Using allowable stresses for the actual construction material,or a system of load factors and materials coefficients when the design is done for the ultimate limit state, the necessary dimentions of struts, wales and sheetpiles may be found through structural calculations.These dimentions represent the result of the analysis.

In the structural design,any soil strength entering the calculations should be taken as a most realistic estimate of the strength, as the safety of the structure is ensured through application of allowable stresses (or load factors/materials coefficients). The geotechnical problem is to ensure satisfactory safety against failure of the soil, e.g.a total failure of a retaining wall,or a bottom heave failure for a braced cut,see Fig.2. In this analysis,the strength of the retaining structure is usually not of any importance and the main output of the analysis is a safety factor against total failure. It should be checked, however that the capacity of the supporting system at least corresponds to the loads which may be computed from the geotechnical calculations.This is usually the case,but may not be so in special cases such as retaining walls at the foot of slopes with low safety factors. The methods used for the geotechnical analysis is otherwise outside the scope of this discussion.

The principles discussed above may be used regardless of the type of soil. The following design procedure, however, has been developed for cuts in the normally consolidated, soft to medium stiff clays of medium plasticity which are typical for Oslo and the areas around.

The calculation of total earth pressure is based on principles described by Bjerrum, Frimann Clausen and Duncan (1972) in their report to the Madrid Conference: Here, they introduced the concept "depth of influence" D_o which is defined as "the depth of the lower boundary of the volume of soil which has taken part in the straining caused by the change in loads during the excavation". In the case of soft clay resting on rock or a layer of firm soil at reasonable depth below the bottom of excavation, this depth is easily identified, Fig.3A. If this is not the case, however, little guidance is given as to how D_o should be determined. I therefore propose that D_o as a maximum is reckoned to the depth where active and passive earth pressures become equal, Fig.3B. The value of D_o may be reduced in the case of small excavations.

The total net earth pressure is now the difference between the active pressure P_A and the passive pressure P_P summarized down to the depth D_o, corresponding to the hatched areas in Fig.3. It should be noted that D_o is used irrespectively of whether the wall extends to this depth or not.

In cases where the shear stress which can be mobilized along the wall is small, the active and passive pressures may be computed according to the classical Rankine theory. If D_o extends far below the foot of the sheet-pile and considerable shear stresses may therefore be mobilized on a vertical plane, a Prandtl type sliding surface may be more appropriate.

The shear strength of the soil has to be estimated in the best possible way. In the case of soft clays, anisotropy and rate effects are important factors which may be considered for instance as explained by Bjerrum (1973) in his general report to session 4 of this conference. For the Oslo clays rate effects are small, but the anisotropy has been taken into account in the study of field cases referred to below.

In cases where the retaining structure reaches rock or firm ground, the influence of stress-strain properties of the soil and the supporting system should be considered. For Oslo clay, the shear strength on the active side is mobilized even at very small strains, but on the passive side the shear strength may only be partly mobilized. The effect is illustrated in principle in Fig.3c and examined more closely be Palmer, La Verne and Kenney (1972).

A comparison of earth pressures calculated as described above, and actually observed strut loads and earth pressures, are shown in Fig.4. In the calculations, the full shear strength is assumed to be mobilized on both sides of the wall, and the anisotrophy of the clay has been taken into account. The observed values are from Technical report no.1-9 from Norwegian Geotechnical Institute, which give data from instrumented cuts at the subway and other construction sites in Oslo. As indicated by the figure, the agreement between observed and calculated values is satisfactory.

The distribution of the net earth pressure depend on the deformations in the soil and the wall during construction, construction procedures etc. It is therefore difficult by traditional analytical means to calculate the distribution of the earth pressure, and certainly the distribution does not correspond to the theoretical diagrams in Fig.3. The finite element method may give an acceptable solution to this problem, but is too cumbersome for routine work. Further calculations are therefore based on empirical methods.

The average distribution of strut loads from the cuts in Oslo are shown in table 1 for systems with different number of wales. There are, however, two sources of scatter in the loads which makes a design based on the average loads unconservative.

Firt, all strut loads are not equal for the struts along one wale. Fig.5 illustrates the scatter for one cut in Chicago (Wu & Berman 1953) and the cuts in Oslo. In Oslo the number of struts in each section was 3-4, in Chicago 8. The load on the most heavily loaded struts may be more than 50% higher than the average strut load.

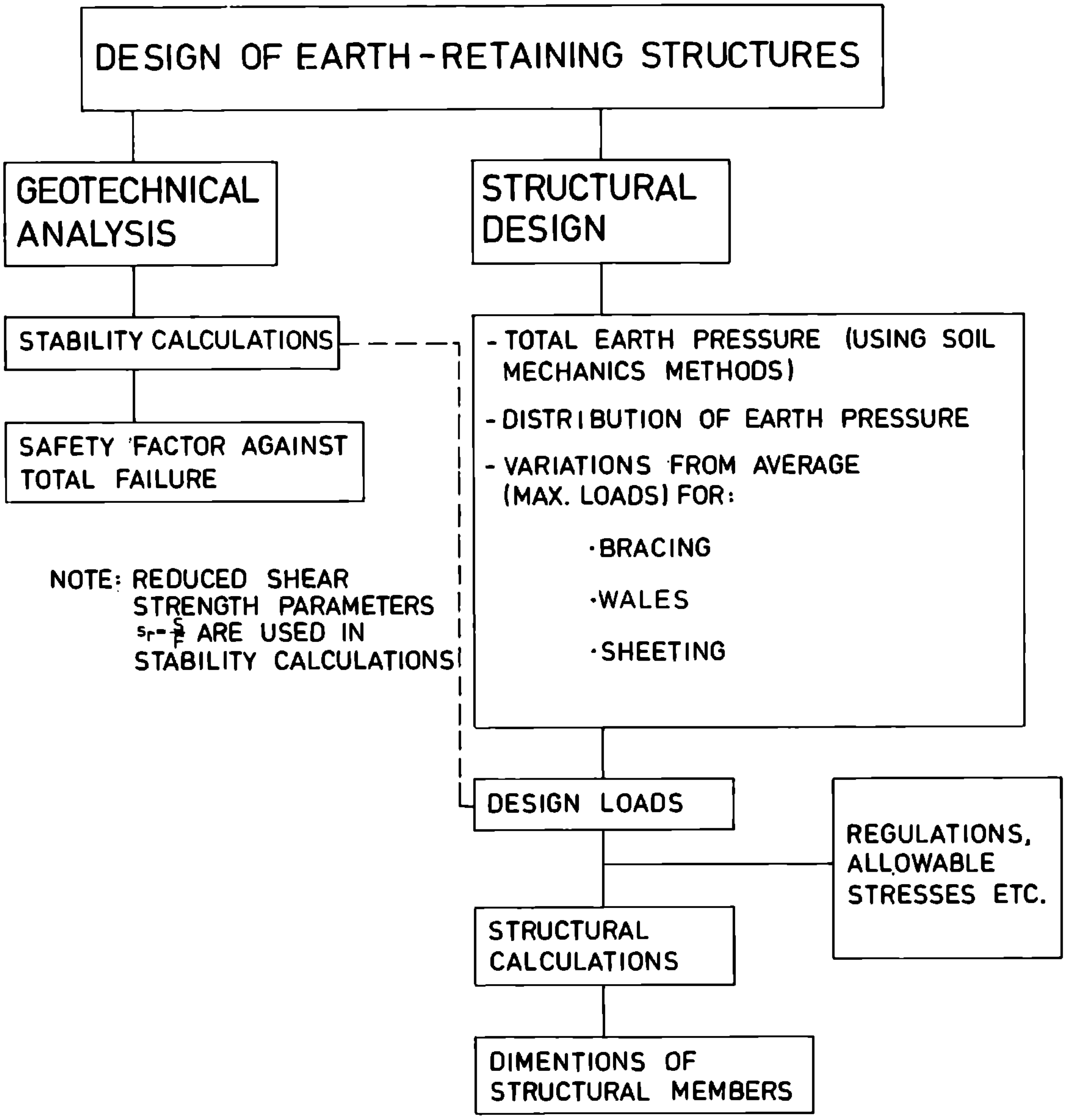
Second, the distribution of average strut loads on the various wales will vary from one cut to another. This is particularly important in cuts with two wales, where the construction procedure may entirely govern the distribution of the loads. For the cuts in Oslo, the upper or the lower layer of struts could each carry up to 80% of the total strut load.

If one heavily stressed strut in a braced cut fails, it may lead to a progressive collapse of the entire bracing system. The struts should therefore be designed for the expected maximum load and not for an average load, considering the scatter described above. For cuts is similar soils and with similar construction procedures to those in Oslo it was concluded that design loads for struts should be 60-80% higher than average loads, see table 2. These figures have been selected from statistical considerations which indicate that about 90% of all struts will in this case have loads not exceeding the design load.

For wales the design load may be chosen slightly lower than for struts, as the scatter, and also consequences of a failure, are both smaller.

For sheetpiles, local overstressing will result in redistribution of loads accompanied by deformations. The sources of scatter mentioned above are therefore not so important in this case, but the effect of sheetpile stiffness on the deformations should be seriously considered.

The main advantage of the design procedure outlined above is that the designer will



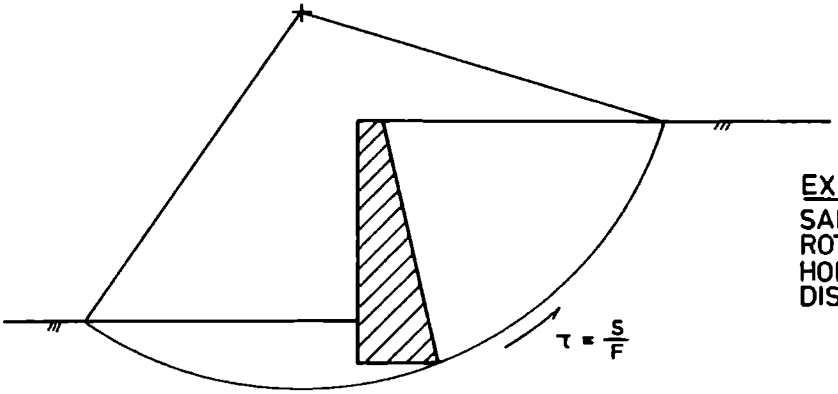
NOTE: REDUCED SHEAR STRENGTH PARAMETERS $s_r = \frac{s}{F}$ ARE USED IN STABILITY CALCULATIONS

NOTE: THE FULL SHEAR STRENGTH IS USED IN EARTH PRESSURE CALCULATIONS

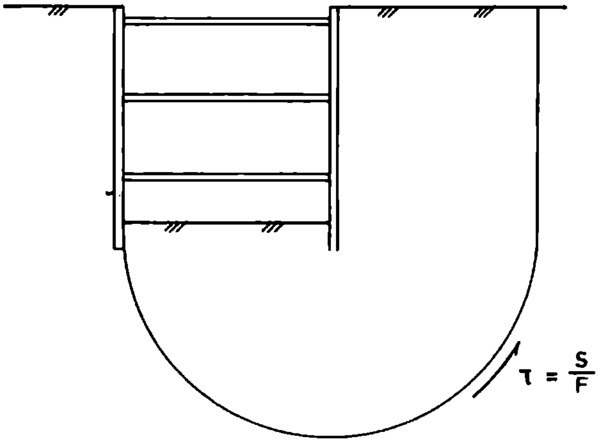
FIGURE 1:
BREAKDOWN OF DESIGN PROBLEM

FIGURE 2:

GEOTECHNICAL STABILITY CALCULATIONS



EXAMPLE NO.1
SAFETY AGAINST
ROTATION OR
HORIZONTAL
DISPLACEMENT

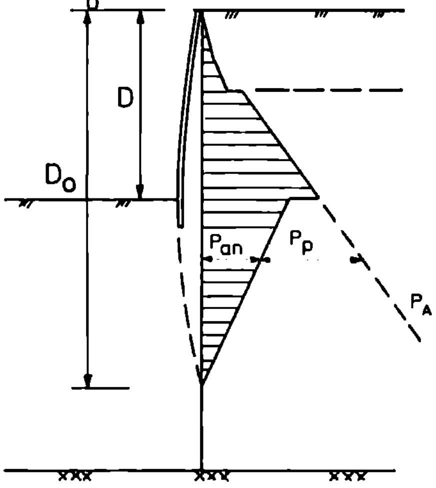


EXAMPLE NO. 2:
SAFETY AGAINST BOTTOM
HEAVE FAILURE.

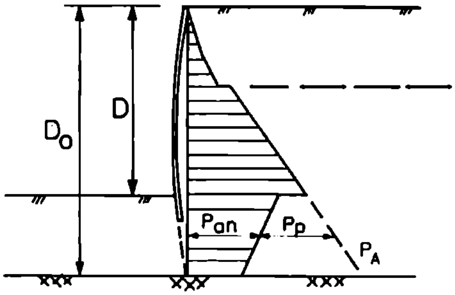
FIGURE 3:

CALCULATION OF TOTAL EARTH PRESSURE

D_0: DEPTH OF INFLUENCE

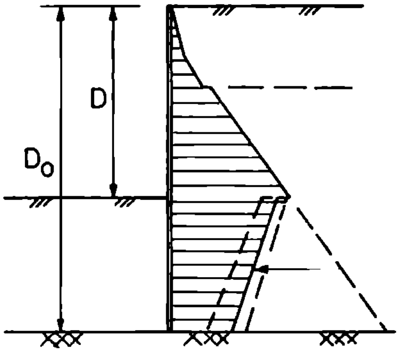


B) $D_0 = D_{0MAX}$



A) D_0 = DEPTH TO ROCK OR LAYER OF HIGH STRENGTH

A) & B) SHEET PILES ARE FLEXIBLE COMPARED TO STIFFNESS OF EARTH. SHEAR STRENGTH IS FULLY MOBILIZED ON ACTIVE AND PASSIVE SIDE.



P_{AN} DEPENDS ON SHEAR STRESSES MOBILIZED ON PASSIVE SIDE

D_0 =DEPTH TO ROCK OR LAYER OF HIGH STRENGTH

C) SHEET PILES ARE RIGID COMPARED TO STIFFNESS OF EARTH. SHEAR STRENGTH IS FULLY MOBILIZED ON ACTIVE SIDE BUT NOT ON PASSIVE.

FIGURE 4:

OBSERVED AND CALCULATED TOTAL EARTH PRESSURE

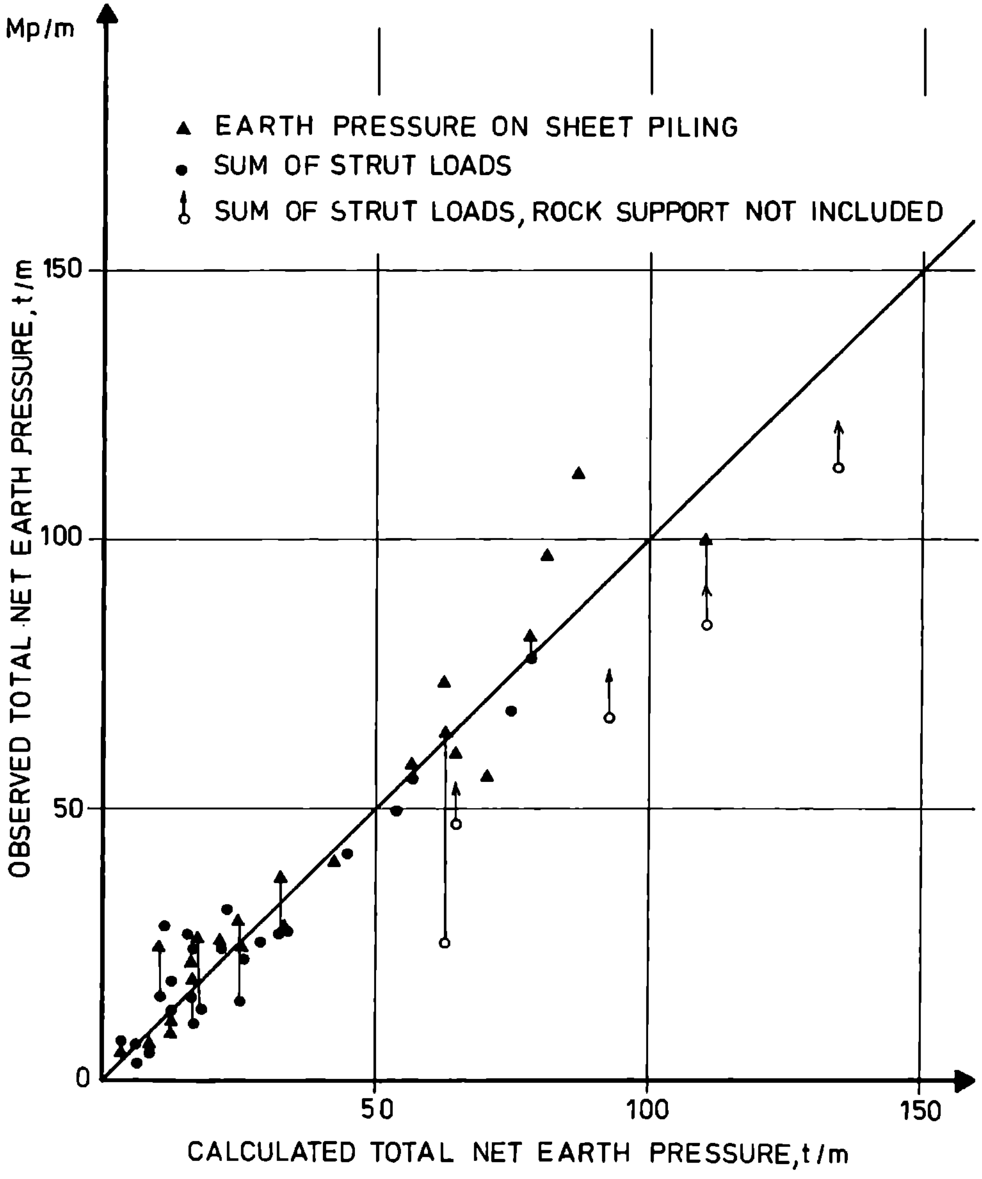


FIGURE 5:

SCATTER OF STRUT LOADS

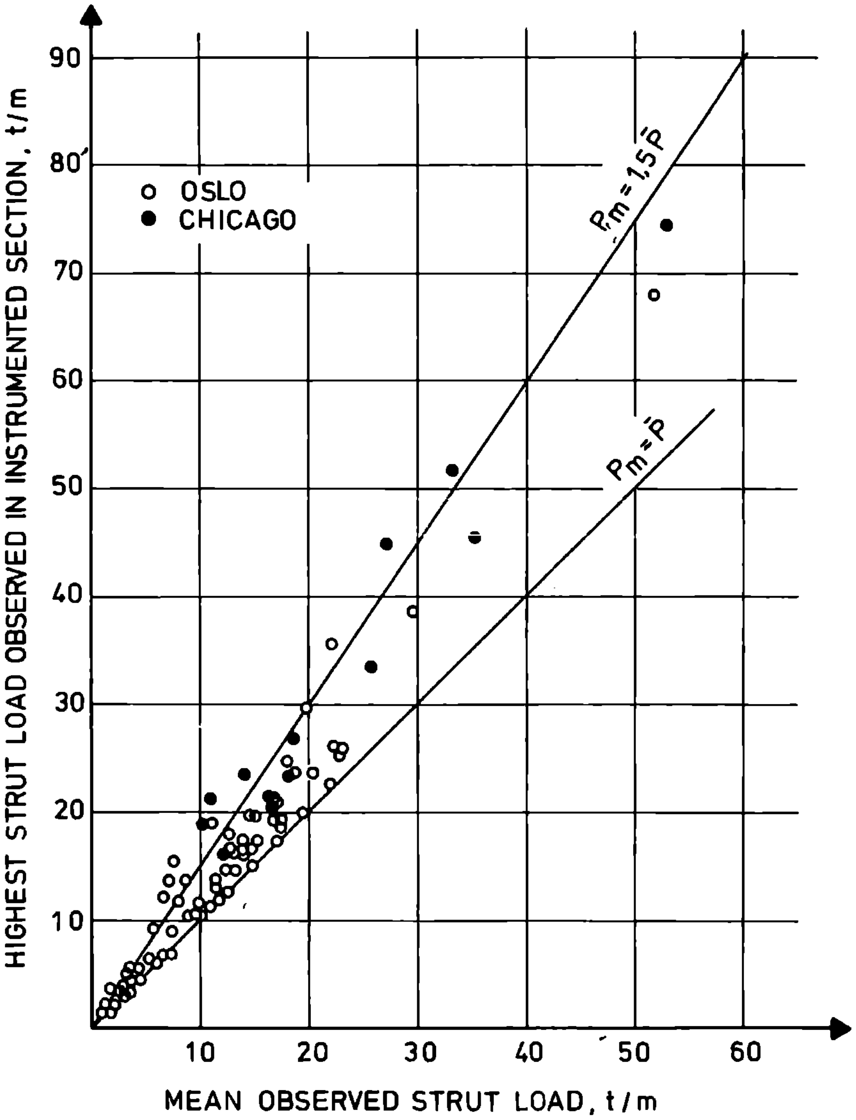


TABLE 1:

DISTRIBUTION OF STRUT LOADS (AVERAGE)

NUMBER OF LAYERS	LOAD ON EACH LAYER (IN PERCENT OF TOTAL)					
	A	B	C	D	E	SUM
2	50	50				100
3	20	40	40			100
4	0	33	33	33		100
5	0	25	25	25	25	100

TABLE 2:

DISTRIBUTION OF STRUT LOADS (DESIGN LOADS).

NUMBER OF LAYERS	LOAD ON EACH LAYER (IN PERCENT OF TOTAL)					
	A	B	C	D	E	SUM
2	90	90				180
3	30	65	65			160
4	0	55	55	55		165
5	0	40	40	40	40	160

NOTE: THE SUM EXCEEDS 100% IN ORDER TO ACCOUNT
FOR SCATTER IN STRUT LOADS.

always know what he is doing. Therefore, he is also able to alter the procedure in cases when basic assumptions are not met.

Consider for instance and anchored sheet-pile wall through soft clay to rock. Due to prestressing of the tie-backs, movements of the wall are very small. Earth pressure on both sides of the wall may be approximately equal to the earth pressure at rest, and the total net earth pressure will be considerably larger than if the full shear strength is mobilized on both sides of the wall. On the other side the scatter of anchor loads from one anchorage to the other may be considerably less for an anchored excavation than a braced, since the anchorages have all been prestressed to the same load and are not subject to temperature differences. The influence of such departures from the basic procedure may all be accounted for in the various steps of the design, and a more rational design is obtained.

REFERENCES

BJERRUM, L. (1973) Problems of Soil Mechanics and Construction of Soft Clay. Proc. 8th Int. Conference on Soil Mechanics and Foundation Engineering, Moskva, vol.3.

BJERRUM, L., FRIMANN CLAUSEN, C.-J. and DUNCAN, J.M. (1972): Earth Pressures on Flexible Structures. (A State-of-the-Art Report) Proc. 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid

Norges geotekniske institutt (1962-1966): Technical Report no.1-9, Oslo

PALMER, J.H., LA VERNE and KENNEY, T.C. (1972) Analytical Study of a Braced Excavation in Weak Clay. Canadian Geotechnical Journal, vol.9, p.145-164

WU, T.H., and BERMAN S. (1953) Earth pressure measurements in open cut contract D8, Chicago subway. Geotechnique vol.3, 1952/53, No.6, p.248-258.

Chairman Dr. G.P.Tschebotarioff (USA)

Thank you Mr.Foss for your report. The next will be Mr.DiBiagio from Norway

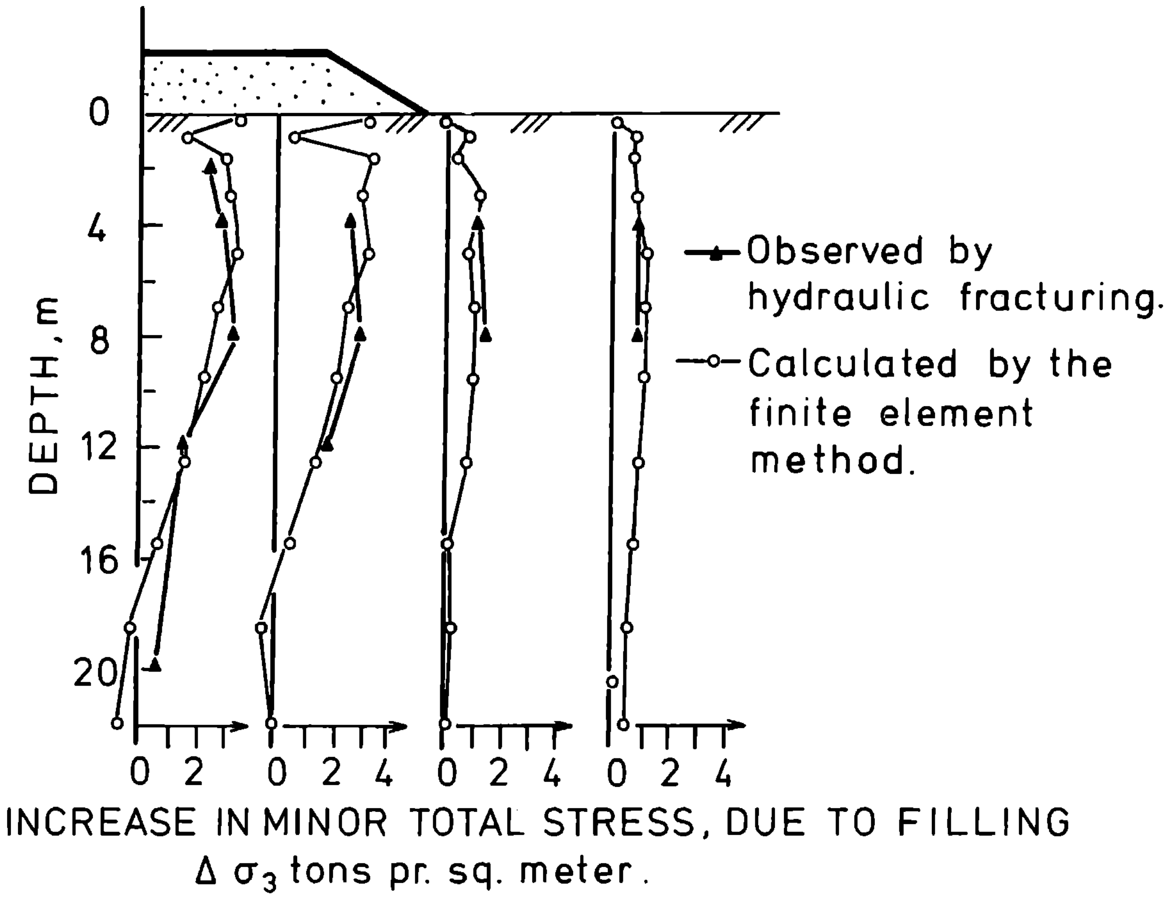
Mr.DiBiagio E. (Norway)

The hydraulic fracturing method of estimating in situ stresses in clay has been used in Norway for several years now. This method and the technique used in Norway has been described by Bjerrum and Andersen at the European regional conference in Madrid in 1972. Although we believe that this method works satisfactorily in our soft, homogeneous, normally consolidated clays, we are of course anxious to compare the results obtained by the hydraulic fracturing technique to results derived from other methods of investigation.

To date at the Norwegian Geotechnical Institute we have been trying to ascertain the validity of the hydraulic fracturing method by comparing such data to the results of laboratory tests and to the results of other types of field measurements. The borehole pressometer, however, which Professor Tschebotarioff mentioned in his introductory remarks has not been used in Norway for measurement of lateral stresses in the ground.

In the final volume of proceedings of the Madrid conference, we have included a typical set of data which compares the K_o-values obtained by hydraulic fracturing to those obtained by means of a specially instrumented pipe pile that has been used for in situ horizontal stress measurements in Norway. The agreement for this site, which is one of our most thoroughly investigated test sites, was exceptionally good.

Another example of an attempt to make on independent check of the results of hydraulic fracturing is shown in Fig.1. These measurements were recently carried out by Mr. Berre at the Institute in conjunction with a large 2-dimensional test fill on a homogeneous deposit of soft plastic clay of low sensitivity. Before the start of filling he used the hydraulic fracturing technique to estimate the K_o-value for this deposit and he arrived at a value of about 0.6 which is reasonable. Of more interest, however, are some tests he did later. At the time of completion of the placement of the fill, he carried out hydraulic fracturing tests in 12 piezometers that were part of the general instrumentation system for the test fill. The minimum principal stresses due to the weight of the fill as determined by the hydraulic fracturing method are compared in Fig.1 to the values obtained by means of finite element computations. The undrained finite element analysis was based on a bilinear stress-strain curve estimated on the basis of the lab data available to date and it is a plane-strain solution. In the figure the curves drawn indicate the change in minor principal stresses resulting from the applied sand fill. The comparison is remarkable perhaps almost unbelievable-and from it we can conclude either of 2 things, defending whether you believe in computers or field measurements: If the hydraulic fracturing method did give the correct stress values, then we can conclude that we selected the correct parameters for the finite element analysis; or, on the other hand, if we know parameters in solution are correct and if we assume that the output of the computer is absolutely correct (as we often tend to do) then we can conclude that the hydraulic fracturing method did give very reasonable results when used in this particular deposit under the prevailing site conditions. Either way-regardless of what conclusion you accept this data is encouraging and I believe that comparisons of this kind are of great value to us in order to establish the usefulness of the hydraulic fracturing method and of equal importance to help us establish the limitations of this method.



Chairman Dr. G.P.Tschebotarioff (USA)

Thank you Mr.DiBiagio. Now I would like to invite Mr. Jezequel (France) to make his contribution

Mr.Jean-François Jezequel (France)

Nous voulons vous presenter quelques resultats de six annees d'experimentation avec la Pressiometre Autoforeur. Cet appareil, dont la description et les applications ont ete donnees en detail anterieurement (BAGUELIN et autres,1972, a,b,c,1973,a,b; JEZEQUEL,1972) a pour caracteristique originale de forer son trou par lui-meme sand modifier le sol, en particulier sand relachement de la pression horizontale.

Il comporte trois elements essentiels (figure 1):

- la sonde,cylindrique creuse,qui est l'element de mesure
- la trousse coupante
- l'outil desagregateur,rotatif,par lequel passe le fluide d'injection.

La penetration dans le sol s'effectue par verinage: le sol decoupe par le trousse,est broye par l'outil desagregateur et remonte a travers le corps de l'appareil jusqu'a la surface par le fluide d'injection. Pendant cette operation,la sonde est maintenue a volume constant, la pression est suivie a tout, moment et permet de controler la qualite de la mise en place (figure 2). La mesure de la pression horizontale des terres au repos se fait de la maniere suivants:

La sonde est arretes a la profondeur voulue. On enregistre alors l'evolution dans le temps de la pression laterale totale,et eventuellement de la pression interstitielle. La valeur limits atteinte represente la valeur cherchee.

Une campagne de mesures a ete effectuee sur 7 sites de sols fins de nature assez diverse: argiles,silts et tourbes (fig.3). Sur chaque profil de 10 metres environ,on a porte,outre la pression laterale poh cherchee,le pression verticale totale pov resultant du poids des terres,et la pression hydrostatique u_e.

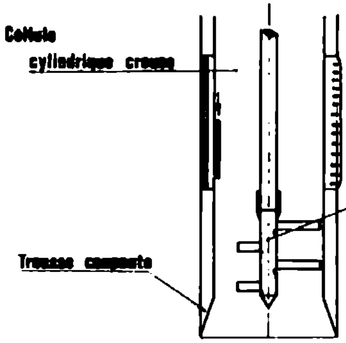
Les valeurs du coefficient k_o correspondantes presentent les particularites suivantes:

- valeurs tres elevees dans les couches superficielles surron-solidees,de l'ordre

de 4 a 5 en general;
- au-dessous,la valeur de K_0 reste constante avec la profondeur dans les cas des argiles et des silts (0,46 pour l'argile de CRAN et 0,40 pour la silt de PLANCOËT).

Par contre elle diminue avec la profondeur dans la cas des argiles organiques et des tourbes pour atteindre des valeurs parfois tres faibles, par exemple 0,25 a 10 metres de profondeur dans le cas de PROVINS.

Dans le site de CRAN en particulier (argile peu consistante),on a compare diverses methodes de determination de la pression horizontale effective au repos p'oh (fig.4).



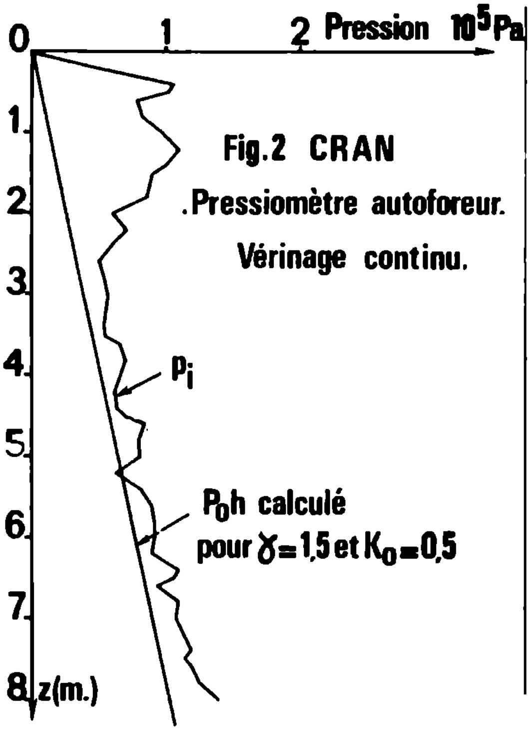


Fig.2 CRAN

.Pressiomètre autoforeur.

Vérinage continu.

Les ecarts relatifs sont importants: ils peuvent attaindre 50%.
- Les essais triaxiaux de typedraine a deformation laterale nulle donnent dans ce cas les valeurs les plus faibles.
- La formule de JAKY ($K_0=1-\sin\varphi'$)donne des valeurs comparables a celles du Pressiometre Autoforeur pour la partie normalement consolidee,mais cette concordance n'a pas ete retrouvee sur les autres sites.
- L'autre methode de determination in-situ de P'oh, la methode de fracturation hydraulique de BJERRUM et ANDERSEN,donne des valeurs plus elevees.

On notera que la methode de fracturation hydraulique a ete appliquee avec un piezometre special mis au point dans les Laboratoires des Ponts et Chaussees: le Piezometre Autoforeur,La mise en place de cet appareil est identique a celle du Pressiometre Autoforeur.

L'element de mesure (fig.5),au lieu d'etre une sonde dilatable,est constitue par une cellule filtrante bordee par deux cellules de garde dilatables en caoutchouc. Elles ont pour role:
- de permettre le desaerage de la partie filtrante,operation qui s'effectue hors sol a l'interieur d'un tube,
- de controler la qualite de la mise en place,
- d'empecher les ecoulements parasites pendant la mesure proprement dite.

Cet appareil permet de mesurer en particulier le coefficient de permeabilite K et le coefficient de consolidation c des sols intacts.

Pour se rendre compte de la validite de la mesure de poh au Pressiometre Autoforeur, on peut comparer un profil de mesures sur le sol vierge de CRAN et un profil sous un remblai stabilise (figure 6) de la maniere suivante: on determine la valeur du coefficient K_0 au point A,sur sol vierge; puis, combinant cette valeur avec la valeur de poh mesuree en B, sous le remblai, on calcule la pression verticale pov, d'ou la masse volumique moyenne du remblai. La valeur trouvee: γ =1,89 t/m^3 est tres proche de celle determinee a partir de carottage (γ=1,80 t/m^3).

Une autre verification de la methode consiste a etudier l'influence de la qualite de la mise en ouvre: la mise en place par autoforage,avec des valeurs de la pression instantanee pi plus ou moins fortes,donne des valeurs reproductibles pour la pression de stabilisation, consideree comme poh. Par contre, la mise en place de la sonde par verinage simple, sans autoforage, c'est-a-dire avec refoulement total,donne une pression laterale qui reste notablement plus forte que poh, meme apres 3 mois de relaxation (fig.7).

CONCLUSION:

La determination des pressions horizontales des terres au repos a l'aide du Pressiometre Autoforeur constitue une methode directe et efficace. Elle est tres rapide: un profil de 10 metres de sols meubles demande un a deux jours de travail.

On s'emplois actuellement a etandre son champ d'aplication a des sols plus raides:

- par example par mise en place de cellules Glötzl par autoforage carre (fig.7).
- et utilisation de moteurs incorpores a la sonde pour atteindre de grandes profondeurs (jusqu'a cent metres).

Fig.8 Autoforage carré

Mise en place de cellules Glötzl.

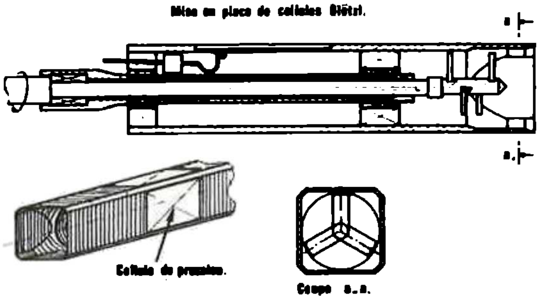
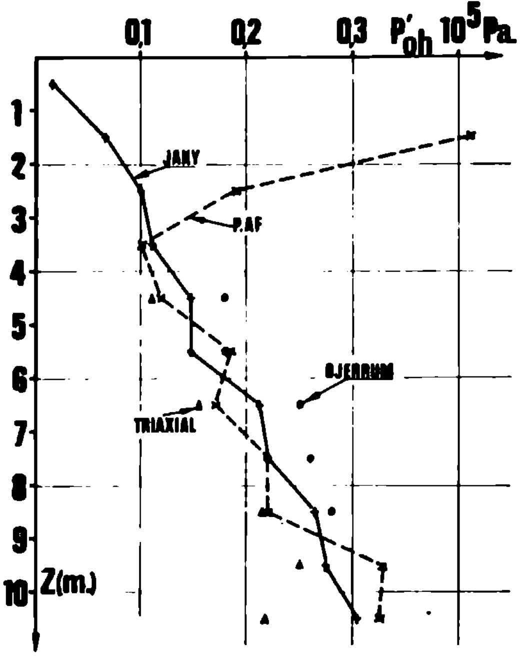


Fig.4 _Site de CRAN_

_Comparaison des mesures de P'oh-



Pressiomètre autoforeur. P_0h sur différents sites. Fig.3

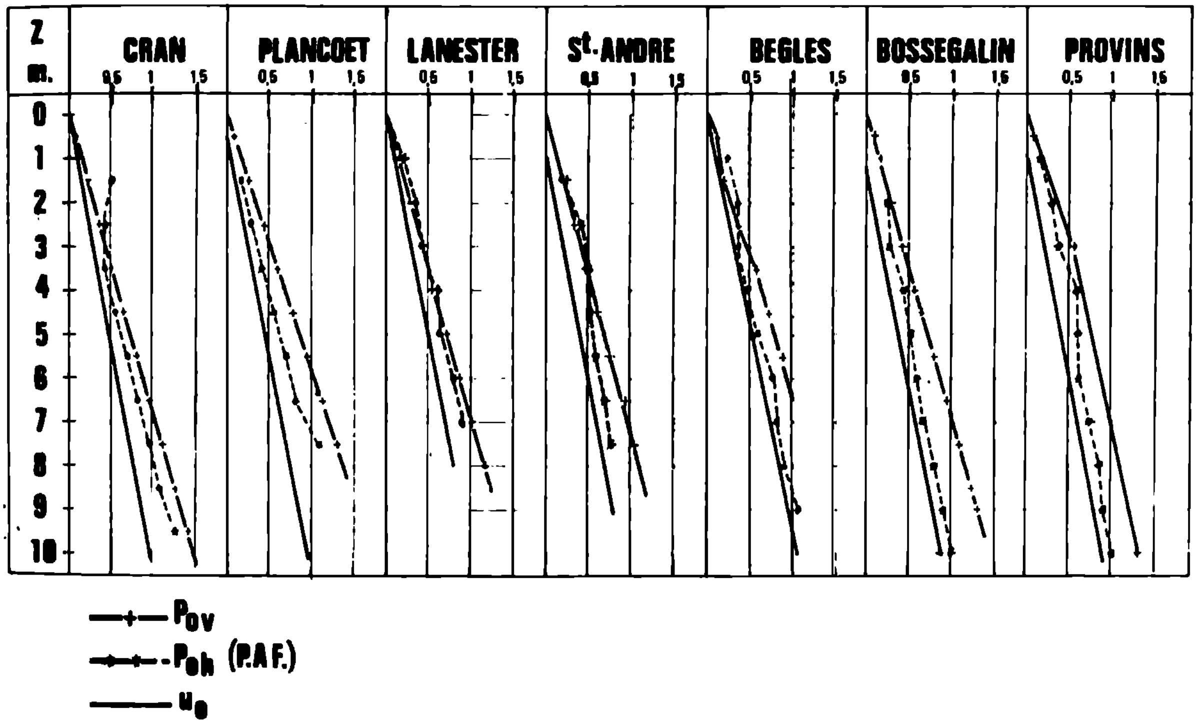


Fig.5 _Piézomètre autoforeur_
(Mesure de K et C_V)

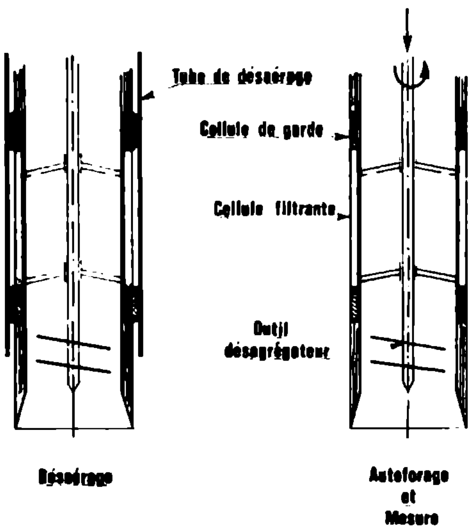


Fig.6 _CRAN_
Poids spécifique du remblai.

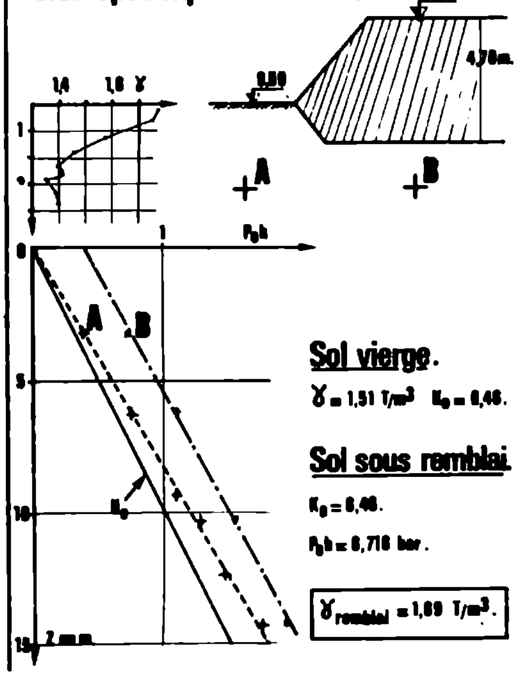
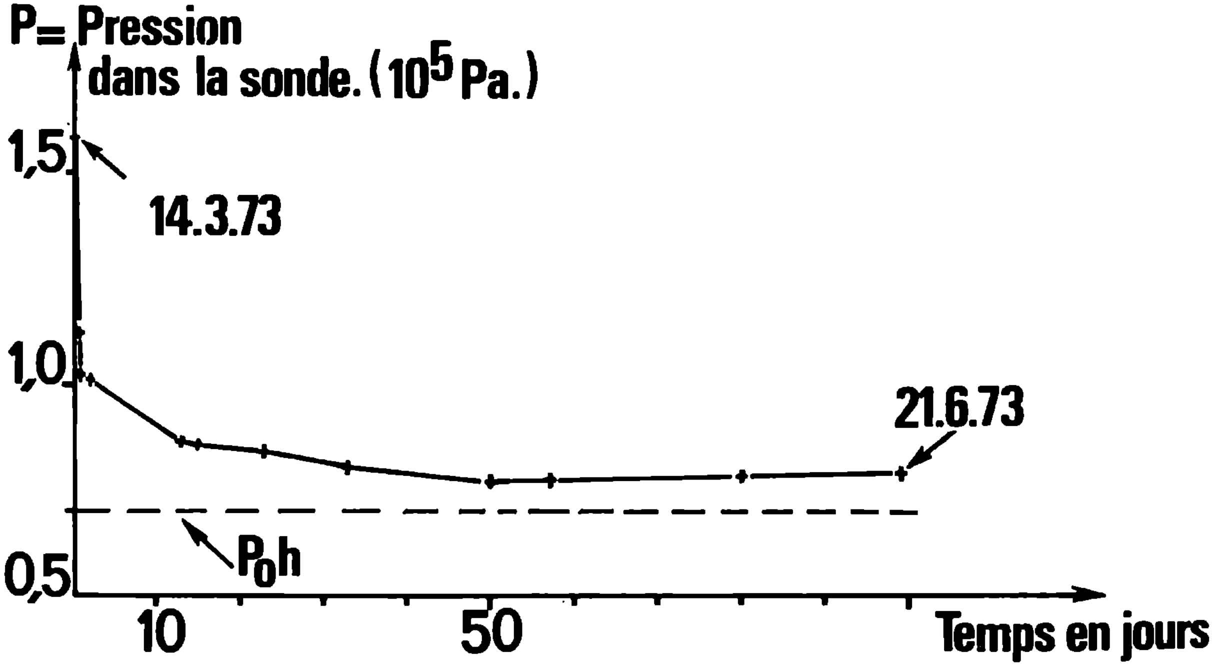


Fig. 7 CRAN Z=5,50m.
.Mesure de P_0h après refoulement total.



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REFERENCES:

Baguelin F., Jezequel J.F., Lemee E., Le Mehaute A. (1972)

a- "L'essai d'expansion cylindrique et le loi effort-deformation des sols purement coherents"
VIe Congres International de Rheologie. Lyon 4-8 septembre 1972, Toms IV, p.239.

b- "Expansion de sondes cylindriques dans les sols coherents", Bulletin des Laboratoires des Ponts et Chaussees no.61, septembre-octobre 1972, p.189-202.

c- "Expansion of cylindrical probes in cohesive soils", Journal of the Soil Mechanics and Foundations Division.
Proceedings of the American Society of Civil Engineers, vol.98, no.SM 11. Proc.Paper 9377, novembre 1972, p.1129-1142.

Baguelin F., Jezequel J.F.(1972,a)
"Undrained plane-strain expansion of a cylindrical cavity in clay: a simple interpretation of the pressuremeter test"
Geotechnique, volume XXIII, Number 2 June 73- Discussion-pages 287,292.

Baguelin F., Jezequel J.F., Le Mehaute A. (1973,b)
"Etudes des pressions interstitielles developpees lors de l'essai pressiometrique. Comptes rendus du VIII^{e} Congres International de Mecanique des Sols et des Travaux de Fondations, Moscou, 1973, Tome II, p.19 a 24.

Jezequel J.F.(1972)
"Mesure du coefficient K_0 en place". Comptes rendus du Ve Congres Europeen de Mecanique des Sols et de Travaux de Fondations, Madrid 1972, Volume II, Discussions pages 286-287.

Chairman Dr. G.P. Tshebotarioff (USA)

Thank you Mr.Jezequel. I would like now to pass the word to Mr. R.Marche (France).

Mr. Rene Marche (France)

When a surcharge has to be placed on part of the surface close to a pile foundation, the piles are subjected not only to the forces due to the structure and eventually to the negative skin friction, but also to lateral pressure caused by the surcharge.

When the surcharged is placed on the surface, horizontal displacements take place in the foundation layers. Along a vertical line at edge of the surcharge, the horizontal displacements can have the shape given figure 1. The pile is subjected to lateral pressure from the soil because it has its own rigidity.

The head and the tip of the pile are restrained and consequently the pile tends to limit the displacement of the soil. The lateral pressure can be large enough to initiate structural rupture of the pile or to cause excessive displacements of the structure supported by the piles.

In order to estimate the lateral pressure of the soil on the pile, the equilibrium position of the pile after it has been deformed must be found.

For the conditions shown, figure 1, the surcharge induces displacements γ_z of the soil along the vertical A A when no pile is present. At equilibrium the displacements of pile are δ_z. To compute the lateral pressure p_z of the soil on the pile, the displacements γ_z and δ_z are computed at each depth as shown figure 2.

The lateral pressure is given by

$$p_z=(\gamma_z-\delta_z)k_{hB}$$

in which k_{hB} is the modulus of subgrade reaction of the soil at the depth considered.

If the pile is devided in n-1 elements of length h limited by n stations the pressure p_z and the forces of reaction R_z can be computed at each station as a function of δ_z. The pile is considered as a beam loaded with concentrated forces R_z whose amplitude are a function of δ_z.

If the pile is in equilibrium, all along the pile and particularly at each station the bending moment due to the external forces R_z must be equal to the bending moment corresponding to the curvature of the pile. In equating the two expressions of the bending moments, a system of n-1 equations with n-1 unknowns δ_z is formed. The system is solved to obtain the values of δ_z and the position and shape of the deformed pile. The bending moments in pile are then computed.

In order to judge the validity of the method the results of three different full scale loading tests were reviewed. The bending moments measured in four instrumented piles were compared with the bending moment computed with the method described previously.

Because of the time available, I will review the results of only one field loading test. In addition, the influence of the values attributed to the modulus of subgrade reaction on the computed moments will not be examined. However, it can be said that the values attributed to the modulus of subgrade reaction have no significant influence on the computed bending moments if (1) reasonable estimates for the modulus of subgrade reaction are made and (2) the variation of the modulus of subgrade reaction with depth translates correctly the differences of rigidity of the layers that the pile goes through.

The results of the loading test which will be examined were reported by Heymann in 1961. The load test set up is given in figure 3.

Three test piles each in steel with an open rectangular cross-section and a length of 12.5 m were driven on 5m centers into a subsoil of peat, clay and sandy clay, the whole being covered with a sand embankment 2m thick. The toes of the piles were driven into a firm stratum of sand. The displacement of the head of the pile was prevented by means of struts propped up at ground level against a concrete beam founded on 8 batter piles. On one side of the pile row, an embankment was extended in the direction of the piles by steps of 5 m.

The stresses in piles were measured my means of strain gages. The maximum bending moment measured in the piles as a function of the distance from the piles to the toe of the road embankment is shown in fig.4.

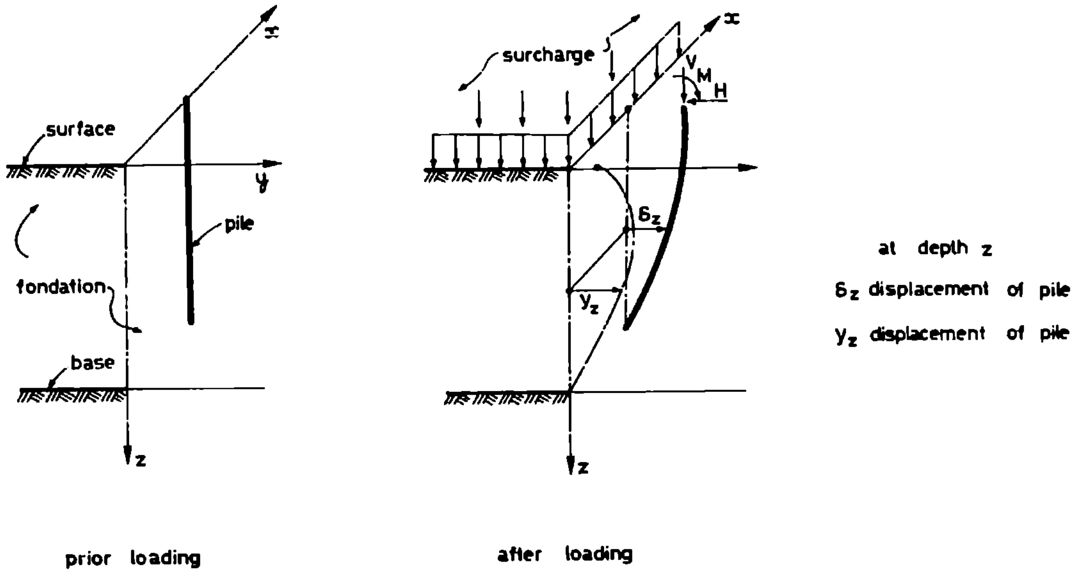
Figure 5 gives the displacements in the subsoil measured after each consecutive extension of the embankment.

Using the horizontal displacements of the soil measured in place and the method given previously, the bending moments in the piles were computed. They are given in figure 6. The computed bending moments increases as the distance between the toe of the embankment and the pile decreases.

This has been observed during the test. The depth of the maximum bending moment which was computed corresponded with the depth at which the maximum bending moment was measured.

The measured and computed maximum bending moments are compared in figure 7. The black point and the open circles give the measured and the computed maximum bending moments respectively. The values of the two are for this loading case quite similar. The computed displacement of the pile are compared to the measured displacement of the soil in figure 8. The displacement of the pile and of the soil are not significantly different. Consequently the reaction frame has not prevented, at least theorically, the displacement of the head of the piles. This also must have been the case during the loading test. If the reaction frame had been efficient, the curvature of the pile would have been much higher than that corresponding to the maximum bending moment which was measured. In addition, it can be shown in reviewing the measurement of the horizontal displacement of soil, that the batter piles or the trestle must have had a displacement of the order of at least 20 cm.

In conclusion, the method seems to permit a reasonable estimate of the bending moments in piles based on this loading test and on the two others. The computed bending moments are not sensitive to the values attributed to the modulus of subgrade reaction but they are sensitive to the considered displacements of soil when no pile are present. This is the most important drawback of the method because the displacement of the soil can not presently be easily obtained with a satisfactory precision and certainly. At least, the range of variation of the probable bending moment in piles can be obtained.



prior loading

after loading

Figure 1 Relative displacements of pile and soil

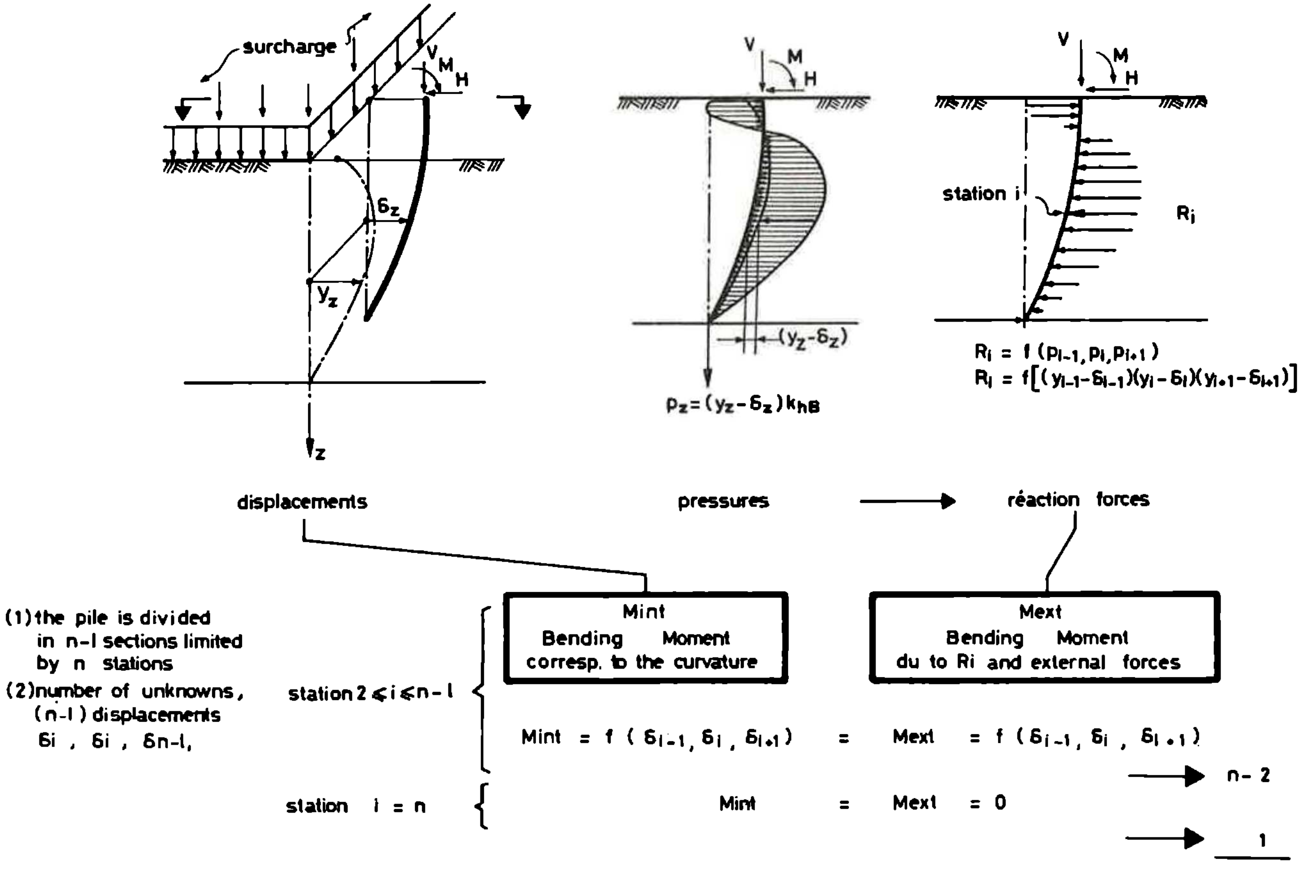


Figure 2 Principle of the méthod.

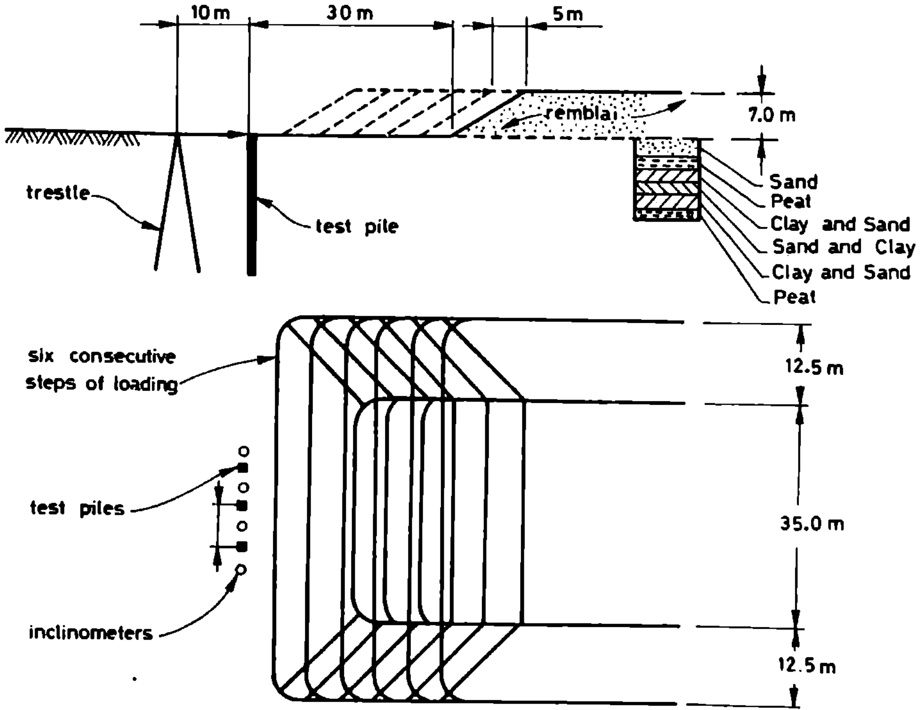


Figure 3 Loading test at Amsterdam

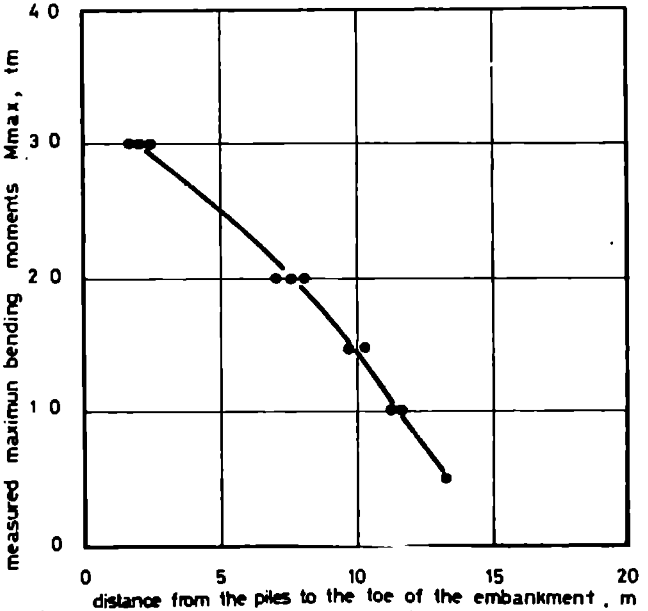


Figure 4 Measured maximun bending moments in piles

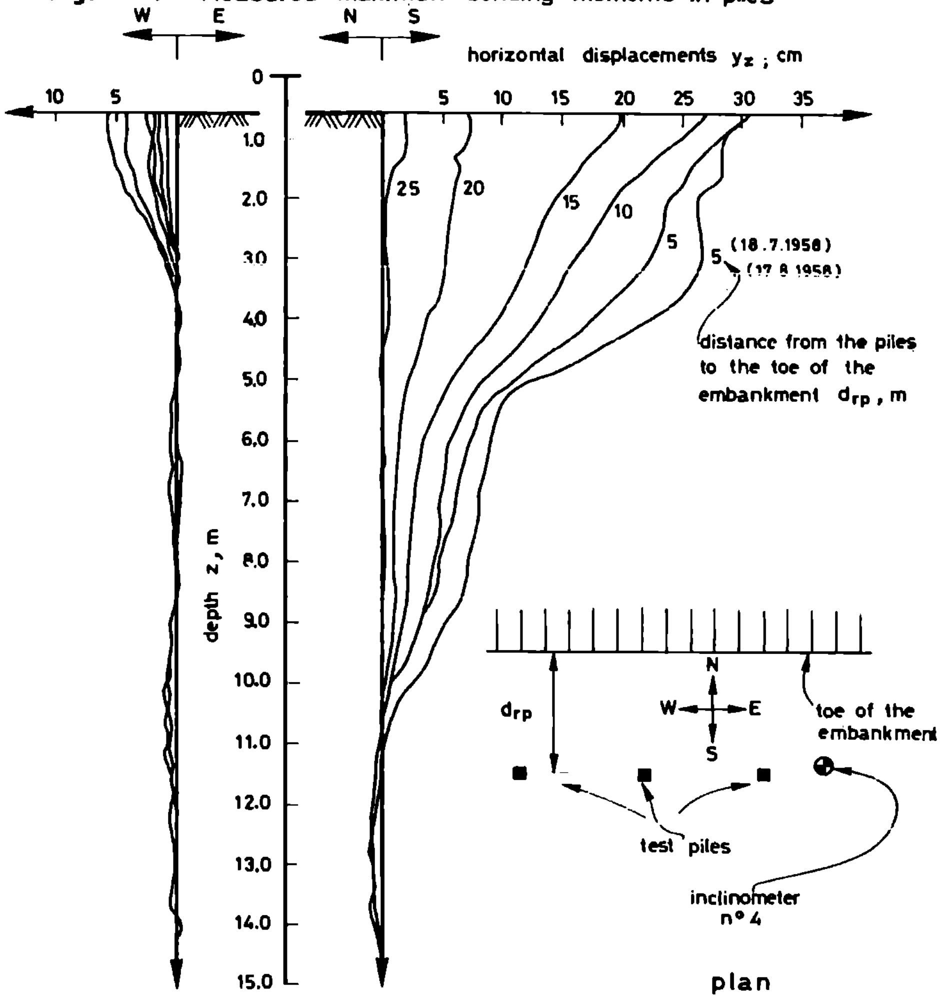


Figure 5 Horizontal displacements in foundation as measured in inclinometer n° 4

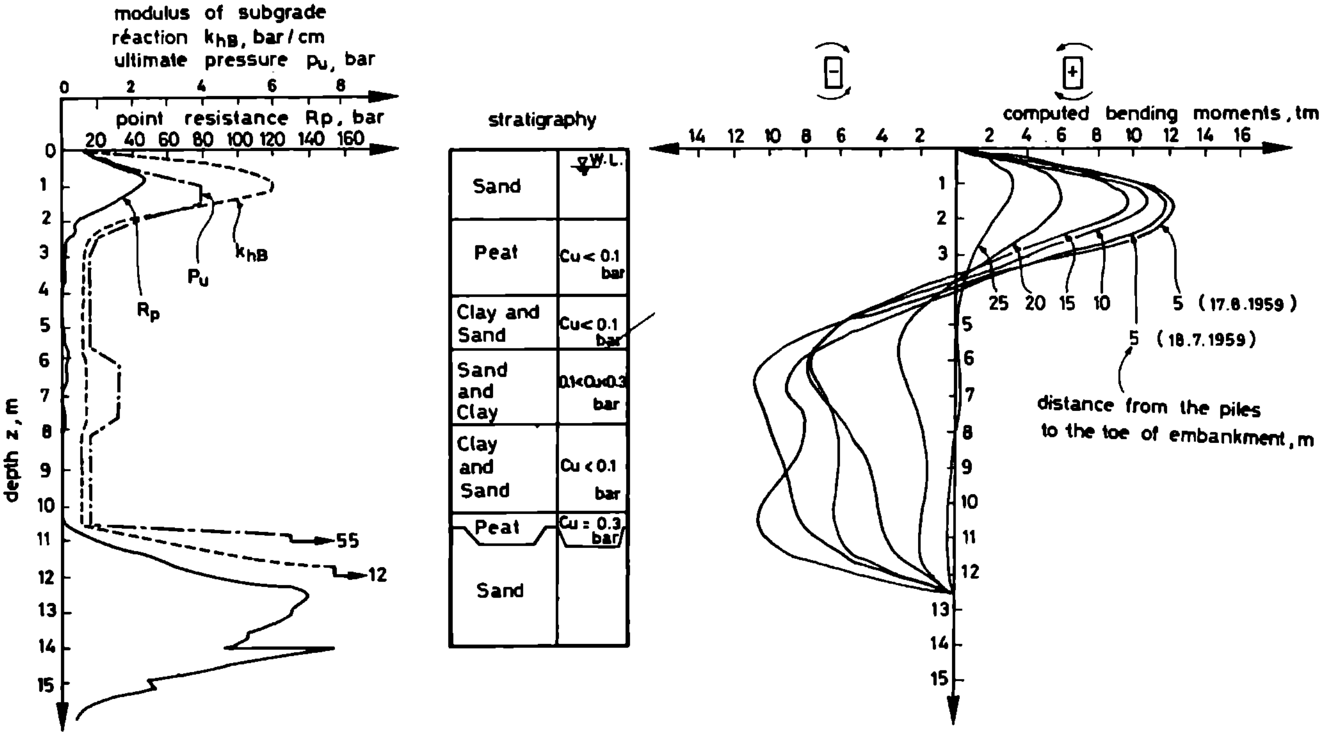


Figure 6 Loading test at Amsterdam, computed bending moments in. test piles

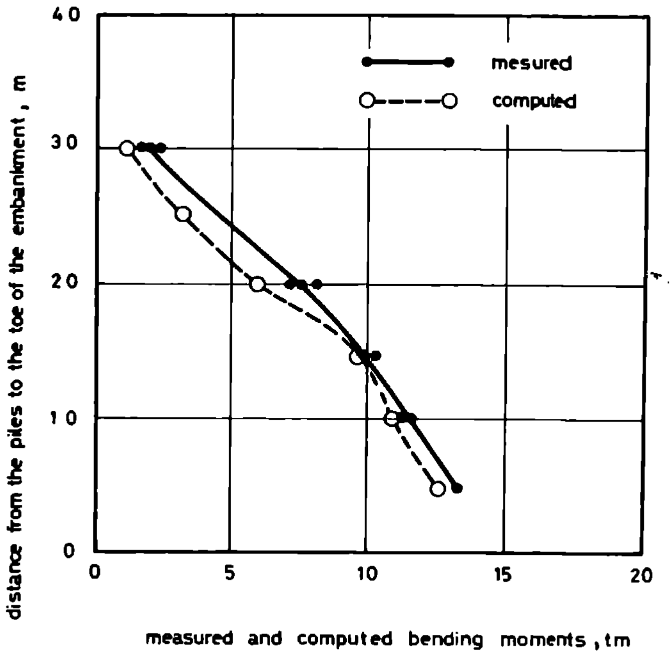


Figure 7 Measured and computed maximun bending moments

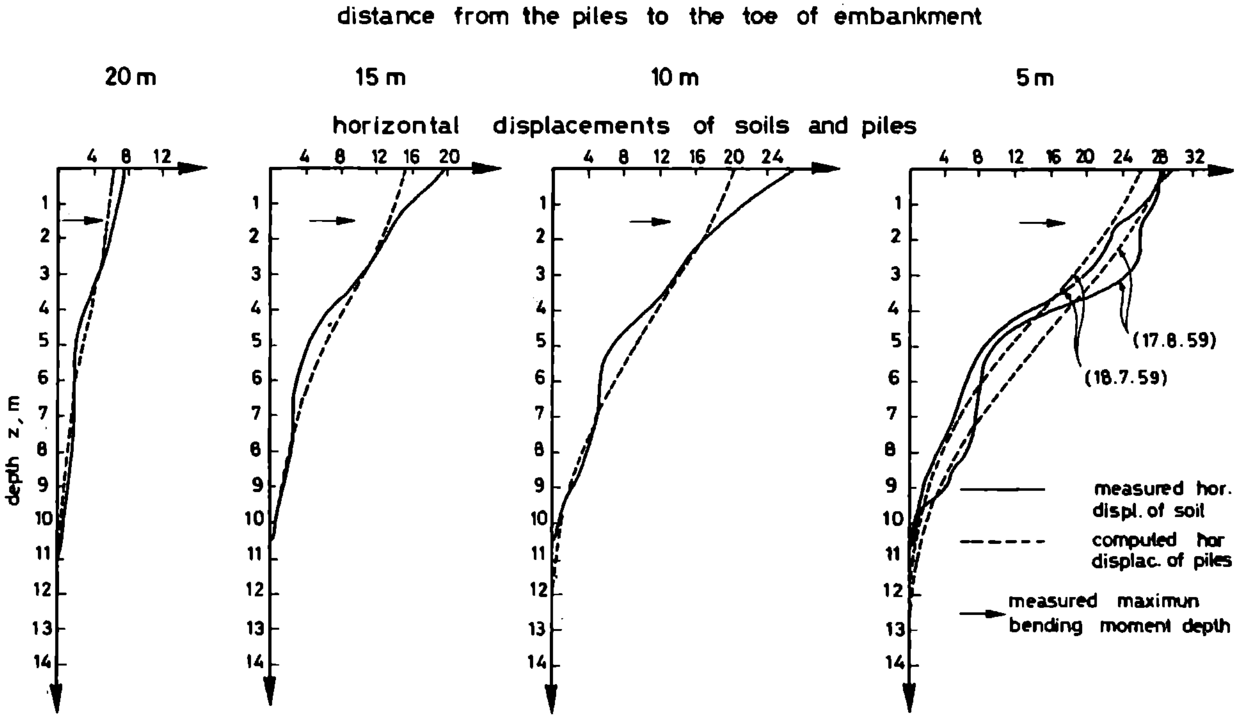


Figure 8 Computed displacements of piles

Chairman Dr.G.P.Tschebotarioff (USA)

Thank you very much Mr.Maroho. Mr.Moser, will you please.

Mr.M.A.Moser (West Germany)

A soft cohesive soil as shown in Fig.1 requires that big buildings must be founded on piles in a deeper stratum. If in the vicinity of the building an embankment has to be placed, the foundation elements of the building will be strained because of horizontal movements of the soft subsoil.

For a typical soft cohesive soil, an investigation has been made on the mutual influence of two piles situated along a line in the direction of subsoil movement. Various values of consistency index of the soil and different geometric conditions have been considered. The soil for the example reported here is a silty clay with a liquid limit of w_f=60%, a plasticity limit of w_a=20%, and a consistency index of k_w=0.72. A section in the middle of the soft subsoil at the first two piles of a row as shown in figure 1 has been analytically investigated with a finite element model.

For a section area, the lateral margins of which correspond to the centerlines (axes of symmetry) between the pile rows, a mesh was chosen with 540 elements and with 589 nodal points. The centerlines have been realized by roller supports. The displacement of the subsoil was accomplished by moving the first row of nodalpoints (front of the area in question) to the right, while the piles have been represented by fixed nodal points and rigid elements.

The dimensions of the piles and the distance between them can be varied within the limits of this net. The reaction of the subsoil under deformation has been analysed for the considered soil by means of undrained plane strain shear tests. A significantly nonlinear stress-strain behaviour was observed. As shown at the bottom of Fig.2, a multilinear characteristic was used for an approximation. To allow for this soil behaviour the continuum has been deformed in small steps.

Fig.3. shows the deformation of the subsoil after a horizontal movement of about 25% of the pile width for two different arrangements.

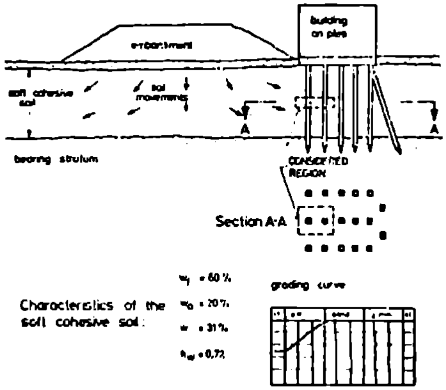


Fig. 1: Foundations on a soft cohesive soil

The drawing at the top shows the movements of the subsoil around a pile of a single row in true relative scale. The drawing at the bottom presents the movement around two piles of a double row with a distance of d=6.b (where b is the pile width). The distance of the two piles in Fig.3b is large enough to let the subsoil form nearly full flow patterns around each single pile.

One important question in such a situation is which mutual influence may occur as to the lateral pile loads. Therefore, in the next figure the load-displacement curves for the three piles are presented.

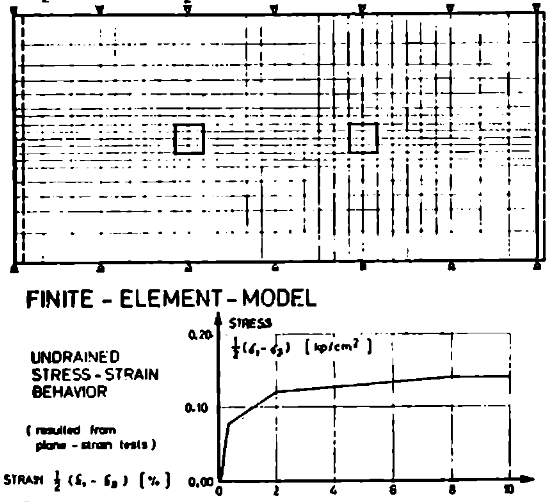


Fig. 2: Finite-Element mesh and soil characteristics

As can be seen by these lines, a steep increase of the lateral load can be observed for the first percents of soil displacement, according to the elastic range of the soil characteristic. In the course of the further displacement, a plastified zone around the piles is developing that gradually becomes greater. As a consequence, no further essential increase of the lateral load occurs after a displacement of about 10% of the pile width.

From this example one may conclude that the lateral load of the first pile in a double row is only 10% less, that of the second pile of the double row about 20% less than that of a similar pile in a single row. (while for the second pile a reduction to about 80% occurs).

If the distance of the two piles becomes greater these reductions are even less. On the other hand, if the distance of the two piles decreases such that the space between the piles is-as an exemple-equal to the pile width, the reduction for the first pile will be 30% and for the second pile as much as 50%.

The presented results show that a reasonable reduction of the lateral loads on pile groups in soft soil in motion can only be expected if the space between the piles is very small.

Chairman Dr.G.P.Tschebotarioff (USA)

Thankyou Mr. Moser for your discussion.
The next will be Mr.Rengautch from the USSR

V.N.Rengautch (USSR)

L'action reciproque des murs de defenses durs et flexibles avec les terrains remblayes stables et instables sous les charges statiques represente toujours un probleme complique.

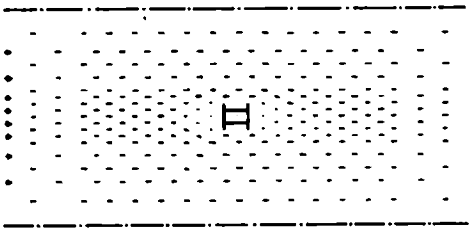
Ce probleme devient encore plus complique sous les charges dynamiques et surtout seismiques.

D'apres l'experience, les experminents des terrains stables ne lerminant que des pressions de contacte sur la superficie interieure laterale du mur ne donnent pas d'effet et sout souvent contredisants.

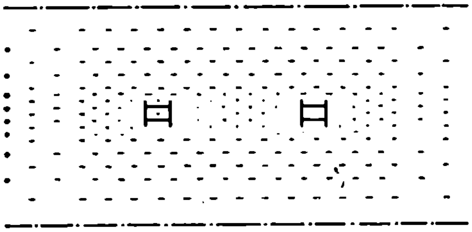
Les dernieres annees nous avons accompli toute une serie des essais complexes concernant le travail des murs durs et flexibles et des murs de defenses sur les modeles de grands gebarits. Ces essais faits surtout dans les conditions in situ de la construction et de l'exploitation des ouvrages de quai et des batardeaux (pour la defense des tranchees de 10 m de profondeur) sous l'action des charges statiques, dynamiques et seismiques donnent les contours caracteristiques (proches aux paraboliques) des zones de brouillage dans le massif remblaye (zones de l'etat limite) et des zones de la densite.

Nous avons accompli ici les dimentions des tensions dans le terrain emblaye et la fixation des zones de brouillage le long de tout le massif remblays.

Quand l'humidite des terrains argileux augmente, l'epure de la pression en contraste des terrains instables dans les conditions limitees est proche aux contours du triangle dont la valeur maximum est un peu plus que la pression de Coulomb.



a) SINGLE PILE



b.) TWO PILES IN A DISTANCE OF d = 6·b

Fig. 3: Soil movements around piles (scale of movements = scale of plan)

M.A.Moser

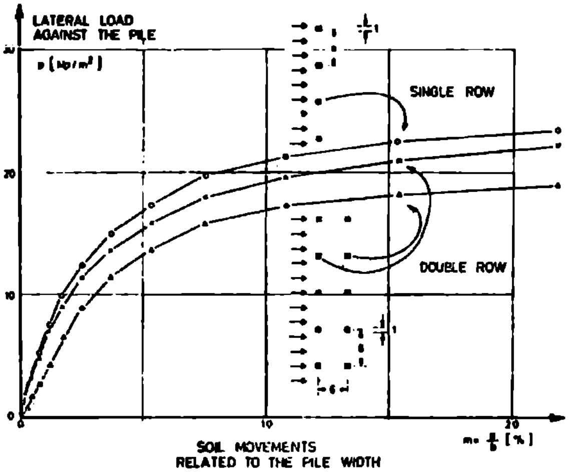


Fig. 4: Load-displacement curves for the piles

M.A.Moser

Pour reçevoir des decisions optima et pour comprarer des resultats de calcul avec ceux des parametres reels dans les conditions in situ la chaire de "Fondations" de l'Institut des ingenieurs des chamins de fer de Leningrad cree une serie de laboratoires in situ au nord et au sud du pays.

On fait cela pour obtenir des precisions de l'application de la theorie aux taches pratiques dans le domaine de projection et construction des murs de defenses.

Ici nous donnons les photos de quelques etapes de nos experiments.

Le tableau N.1 montre l'appareil d'observation de grands modeles des murs de defenses.

Nous appelons cet appreil "stand sur les ressorts"

A l'aide de ce stand (qui est represente ici pendant des essais avec la charge statique) nous avons examine toute une serie de grands modeles des murs de defenses durs et flexibles dans les conditions des actions de charges statiques, dynamiques et seismiques.

Le stand est equipe d'un caisse 130 m^3 de volume, poids maximum de la partie sur les ressorts 320 t.

Sous la charge statique creee en ce cas a l'aide des dalles en beton arme les ressorts du stand assurent la protection contre les vibrations et forment les conditions ideales de l'equilibre limite.

Le tableau N.2 nous montre l'installation des tensimetres exterieurs sur la surface du mur dans les points determines.

Le tableau N.3 montre le moment des essais dynamiques (c'est la dalle de vibration fondee sur la surface du remblayage qui travaille).

Le tableau N.4 montre les essais du quai en charge statique.

Le tableau N.5 montre les essais du batardeau de deux rangs: au premier plan - a l'aide de la charge dynamique, au deuxieme plan - a l'aide de la charge dynamique.

Nous allons continuer nos recherches dans ce domaine.

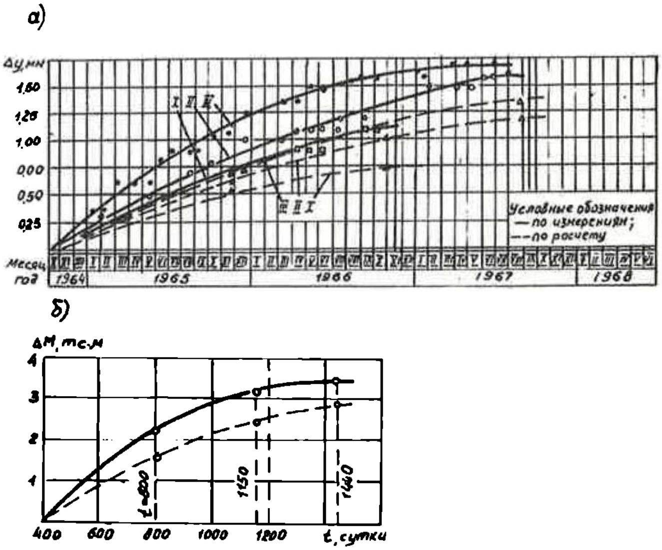
Chairman Dr.G.P.Tschebotarioff (USA)

Thank you Mr. Rengautch. Now I would like to invite Mr. Budin from the USSR.

Mr. Budin, would you please.

Budin A.Y. (USSR)

Thin brest-walls are widely used building constructions. During exploitation it became clear that strained state of thin brest walls on the clayey soils as a rule changes essencially in time.To reveal the reason of the circumstance mentioned above the vast field investigations that lasted for many years under different engineer-geological conditions were undertaken at some units. The investigations,the ground pressure, the walls' strain and deformation were being measured, clearly showed that the cause of nonstability of strained state of the walls is in the change of the reactive pressure of clayey ground upon them as a result of the display of its reological properties, In most cases pressure intensity relaxes gradually which increases efforts in the parts of the structures. Fig.1-a shows the measured and calculated increments while bending of anchored wall which has the free height of 7,3m, the submerged section of which cuts the layer of softplastic-belt clay,the capacity of which is 5,3m and which cuts the underlying morainal loam to 1,95m. The diagram 1-b gives the increments of the bending moment at the most submerged cross section of the wall.



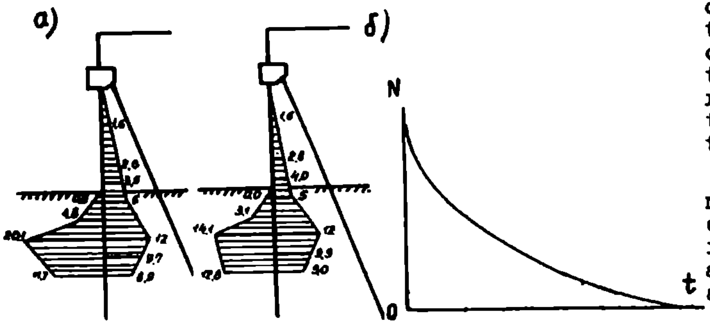
The process of relaxation of contact reactive pressure of the cripping clayey soil is explained by using of all the elastic forces of the thin brest walls while increasing their bendings. Epuras of the soil pressure upon the concrete sheetpiling wall, anchored by the inclined piles and erected at the same engineer-geological conditions as well as the construction mentioned above are given in the Fig.2-a and illustrate it. The period of time between measurements, the results of which are given in Fig.2-a, lasted 14 months.

From the above mentioned, it becomes clear that it is necessary to provide the durable strength of thin brest-walls submerged into clayey soils. For this it is necessary to be able to build an epura of the reactive pressure soil upon walls corresponding to any age of the structure. As a result of the theoretical works the defferential equations were obtained, illustrating the regularity of the walls' interaction with clayey soils,which possess reological properties.

The solutions found give us the oppurtunity to calculate the intensity of clayey soils of lateral pressure upon non-anchored and anchored walls as the function of time t depending on the wall rigidity EI and the coefficient of the soil viscosity η/1,2/. The great influence upon relaxation of reactive pressure of the clayey soil is produced by the kind of work of the wall section submerged into the underlying soil. At the calculated dependences the coefficient of the pliability of the bed of the underlying soil K and the length of the wall section l submerged into it are given. If we know the reactive pressure p(x,o), we can define the coordinates of the pressure epura from the ratio:

$$p(x,t)=p(x,o)f(EI,\eta,k,l),$$

where f depends upon the constructive peculiarities of the walls and upon the correlations of geometrical measurements of their elements.



On the base of the calculations made we can make the diagram of the dependence of the ability of the thin-wall structure on the clayey foundation upon time $N=\phi(t)$ (fig.2-b)

The calculations according to the methods worked out correspond to the results, obtained from the field investigations.

Literature:

1. Budin A.Y. Calculations of the sheet-piling wall on the cripping foundations. Calculations of the stability of the transport hydrotechnical structures. "Energy", 1967.
2. Budin A.Y. Exploitation and durability of the hydrotechnical structures in the port. "Transport", 1971.
3. Budin A.Y. Analysis of the work of the thin breast-walls on the cripping grounds. "Base, foundations and mechanics of soils", N 6, 1969.

Chairman Dr.G.P.Tschebotarioff.

Thank you Mr.Budin. Miss Karaulova, will you please.

Karaulova Z.M. (USSR)

1. The observations show, that in many cases the displacement of retaining walls right up to their destruction depends on the presence, in the wall foundation or in the strata, supported by these walls, of the clay soils which tend to the creep deformation.

2. When the creeping process is going on, the strength of clay soils, under certain conditions, due to decrease of the cohesion value characteristic for clays, may decrease with time.

3. The laboratory tests conducted in Moscow Automobile and Road Construction Institute show, that the value of cohesion in clay soils of solid and semi-solid consistency, depending on the deformation, may decrease by about three times. In this case the limiting state is in compliance with the angle of deformation nearing 0.20.

4. Because of the above said, when determining the active pressure of soil acting on the retaining walls, the value of cohesion is often excluded from the calculations at all, or without a reason, it is assumed as a part of it.

5. For the evaluation of the shearing strength of clays it seems efficient to proceed from the relationship suggested by N.N.Maslov (1941), in which cohesion represents the combination of acting in soil binding forces (Σ_w) of water-colloidal nature and forces of hard-structural cohesion (C_c) of irreversible character.

6. Proceeding from the above said, when determining the active pressure of hard highly jointed clayey bedrocks in which the binding forces (Σ_w) and forces of structural cohesion (C_c) for the strata may actually be assumed equal to zero, the rated cohesion is also equal to zero ($C_{rated}=0$).

7. If in the filling and in the foundation of the retaining walls the plastic clay occurs, the rated value of cohesion (C_{rated}) in the active pressure value, is determined according to:

a) the possibility of deformation of displacement of the retaining walls proper as a result of the creep deformation of clay soils in their foundation;

b) the permissibility of such a deformation for the proposed retaining wall.

8. If such a deformation is present and permissible, the rated value of cohesion, when determining the value of active pressure, may be assumed equal to the binding forces, i.e. $C_{rated}=\Sigma_w$

9. If the prolonged displacement of a retaining wall is impermissible, the rated value of cohesion is assumed to be equal to zero ($C_{rated}=0$).

10. In case the deformation of displacement is absent and the clay soils are slightly jointed, the rated value of cohesion may be assumed equal to the value of binding forces (Σ_w) and to that of the forces of structural cohesion, i.e. $C_{rated}=\Sigma_w+C_c$.

11. Proceeding from the above conceptions and criteria of the creeping deformation, described in the Report at the Conference in Madrid (1972), the value of soil active pressure acting on the retaining walls, may vary, depending on the character of soil and on the conditions of deformation of displacement of the retaining wall proper.

REFERENCES

MASLOV N.N. Karaulova Z.M. "Problem of general stability of rigid retaining structures on clay" 5 European Conference on Soil Mechanics and Foundation Engineering, vol.1, Madrid, 1972.

Chairman G.P.Tschebotarioff (USA)

Thank you Miss Karaulova for your discussion. The next will be Mr Yakovlev (USSR) Mr.Yakovlev will you please.

Mr.P.I.Yakovlev (USSR)

As it is known, Coulomb's theory neither has been successfully applied for many practically important cases nor there is found any finite analytical solution for determining cohesive soil pressure on a retaining wall.

At calculation it should be considered also that the solutions based on Coulomb's theory contain sometimes significant errors which may reach its highest in the case of cohesive soils.

That is why the engineering solution based on a theory of limiting state of stress which was completed in the works of professors V.V.Sokolovsky and S.S.Golushkevich (USSR) is of practical importance.

In the USSR the engineering solution of the problem is suggested to be based on Golushkevich's theory. The solution is applied for the general case of inclined rough wall, inclined surface of back-fill and external uniformly distributed vertical load acting on a back-fill surface. The solution for the case of inclined external load acting on a back-fill surface can be obtained without any special difficulties. The finite solution is obtained in a more appropriate analytical form (Yakovlev, 1973).

Slip surfaces are determined on assumption of weightless medium but at calculations to follow the forces of interaction acting between different zones of limiting state of stress were obtained making account of unit weight of soil. As it was proved by professor S.S.Golushkevich and V.S.Khristophorov such assumption is quite appropriate while developing practical methods of determining soil pressure on a retaining wall.

In this case the slip surface comprises two plane lines limiting the zones of minimum and maximum state of stress and a conjugate arc of logarithmic spiral in Prandtl's zone.

The solution obtained can serve as a basis for calculation and compilement of the Table of lateral pressure coefficients for cohesive soil.

Summing up it should be stressed that on a basis of the general theory developed by professors Sokolovsky and Golushkevich simple engineering solutions can be obtained for all cases where Coulomb's method is now applied which can bring significant gain of accuracy of calculations (Jakovlev, Lubenov, Vargin, 1968, 1972, 1973).

BIBLIOGRAPHY

Yakovlev P.I.(1973) O predelnom naprjazhennom sostoyanii sypuchego klina (Safe Stress State of Granular Wedge Soil). Stroitelnaya mekhanika u raschyot sooruzheniy, N 1.

Lubenov R.V., Yakovlev P.I.(1968). Morskie prichalnie sooruzheniya gravitatsionnogo tipa (Marine Gravitational Mooring Structures). OIIMF, Odessa

Yakovlev P.I.(1972). Opredelenie davleniya sypuchego grunta na podpornuyu stenky s naklonnoy zadney granju (Determination of Granular Soil Pressure on a Retaining Wall with Inclined Back Side). Gidrotechnicheskoe stroitelstvo, N 10.

Yakovlev P.I., Vargin M.N.(1973) Opredelenie davlenija grunta ot polosovoy nagruzki, raspolozhennoy perpendikuljarno stenke (Determination of Soil Pressure under Stretched Load Acting Normally to Retaining Wall). Izvestiya vuzov, stroitelstvo i architektura, N 5.

Yakovlev P.I. (1968). K opredeleniyu passivnogo davleniya grunta na podpornye stenki (On Determining of Passive Soil Pressure on Retaining Walls). Stroitelnaya mekhanika i raschyoty sooruzheniy, N 4.

Chairman Dr.G.P.Tschebotarioff (USA)
Thank you Mr.Yakovlev.
The next will be Mr. Vidmar from Yugoslavia.

Mr.S.Vidmar (Yugoslavia)

In two previous papers we have presented model test results proving that, after stopping the movement of the retaining plate, the active pressure of cohesive soils increases and the passive pressure decreases, both approaching the value of the earth pressure at rest (Šuklje and Vidmar 1961, Vidmar 1963). Similar relaxation effects have been observed in tests with sandy backfill, yet the active pressures were found to increase toward values smaller than the initial at rest pressures, and the passive pressures to decrease toward ultimate values overpassing the initial at rest pressures (Vidmar 1968). Recently (Vidmar 1973) the tests of this kind were repeated in an improved model diminishing the effect of the lateral wall friction. These tests have confirmed previous statements. Moreover, special test series have been performed in order to establish kinematical conditions for sustaining certain values of earth pressures corresponding to various degrees of the mobilization of the soil strength in either active or passive direction.

By two translatory displacement-earth pressure diagrams Fig.1 presents the sequence of two series of such tests. The silty backfill (w_L=46%, I_p=18%, w_o=33%, γ=1.70 t/m^3) 24 cms in height has been subjected to a surface load of 1 t/m^2 in the first and of 2.5 t/m^2 in the second series. As proved by the stability analyses, only in the last phase (ΣP=0) of the second series the strength of the backfill (c=0,45 t/m^2, φ=34°) was fully mobilized; a constant speed of the movement of the retaining plate was necessary to maintain the limiting value of the active earth pressure. At the degrees of the strength mobilization smaller than 0.50 (Fig.2: 0.435 at q= 1 t/m^2, ΣP=7 kp; 0.48

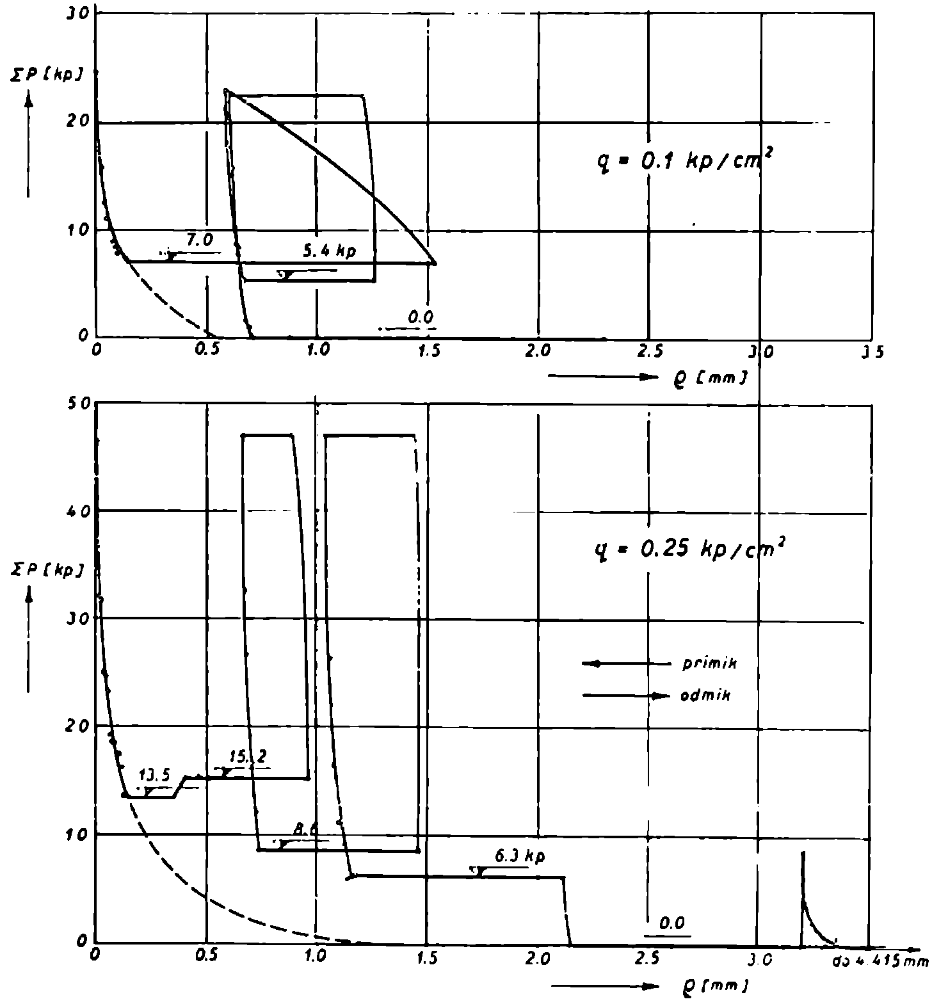


Fig. 1

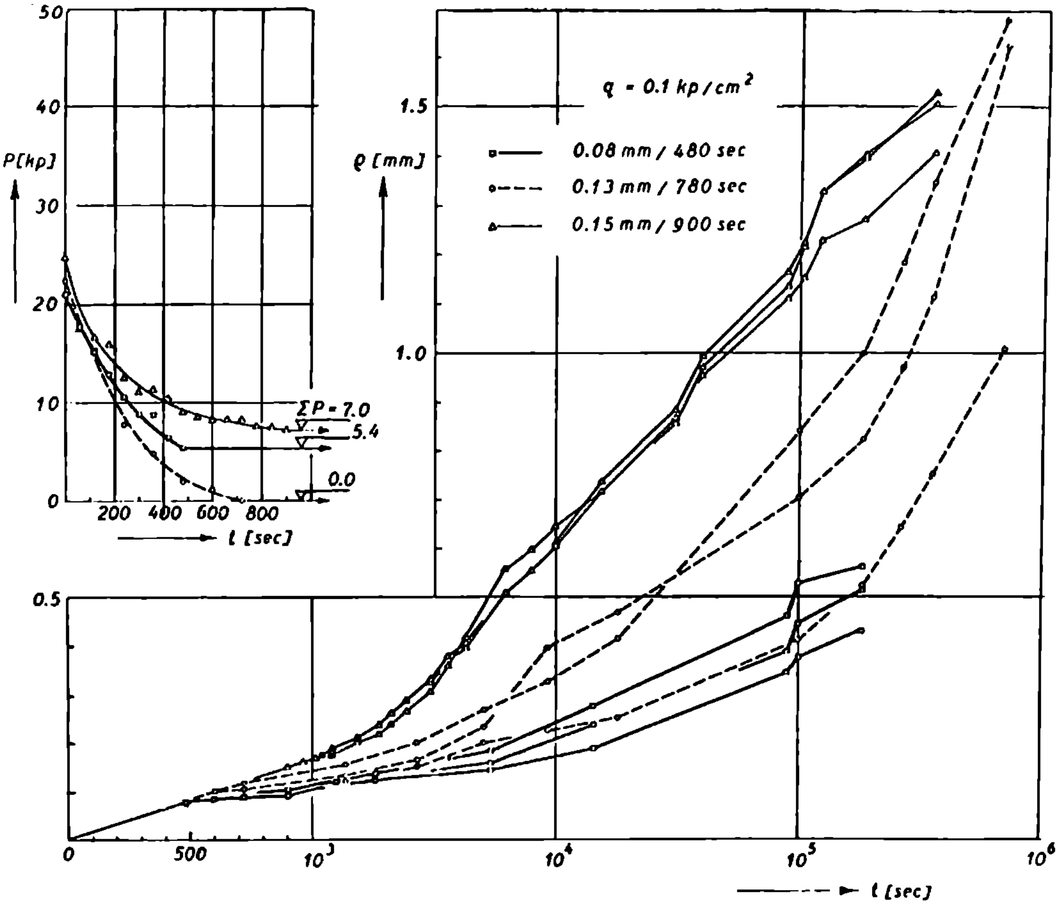

50
40
30
20
10
0
ΣP[kp]
200
400
600
800
t [sec]
ΣP = 7.0
5.4
0.0
q = 0.1 kp/cm²
ϱ [mm]
1.5
1.0
0.5
0.08 mm / 480 sec
0.13 mm / 780 sec
0.15 mm / 900 sec
0
500
10³
10⁴
10⁵
10⁶
t [sec]

Fig. 2

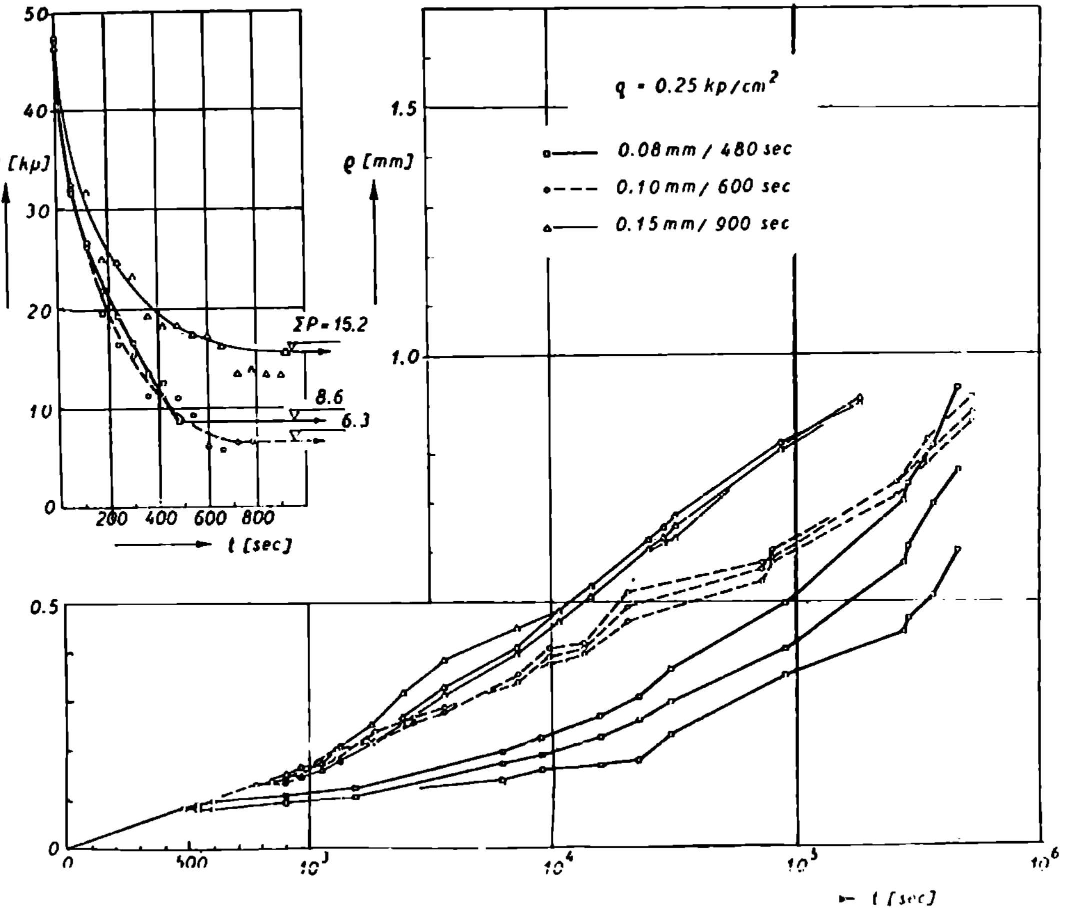


Fig. 3

$q = 1\ t/m^2$, $\Sigma P = 5{,}4$ kp) and 0,70 respectively (Fig.3: 0,7 at $q = 2.5\ t/m^2$, $\Sigma P = 15.2$ kp) the displacement needed to maintain the mobilized earth pressure increased linearly with the logarithm of time. The negative acceleration became smaller at higher mobilization degrees, yet the speed still decreased even at the mobilization degree as high as 0.91 ($q = 2.5\ t/m2$, $\Sigma P = 6.3$ kp).

Returning to the diagrams of Fig.1 it can be noticed that, after reversal movement in the passive direction, smaller relative displacements are sufficient to mobilize a certain active earth pressure value; the envelope of the summarized pressure-displacement diagram seems to approach the extrapolated virgin diagram.

The above ascertained kinematical conditions for maintaining a certain effective-stress state agree with the experience of either ring-shear (Šuklje-Vidmar 1961) or triaxial tests (Bishop 1966, and Lovenbury 1969, Šuklje 1967). The secondary settlements of the soil bases of retaining structures are able to secure the displacements needed to maintain earth pressures corresponding to a certain degree of the strength mobilization of the backfill. The determination of the quantitative relation between the deformation of the subsoil and the backfill is a complex problem which, at the actual state of our knowledge, cannot be solved without simplifying assumptions. The procedure applied by Šuklje, Marczal and Podržajeva (1968) for the Rankine's case of earth pressures gives some indications for the existing possibilities.

REFERENCES:

Bishop, A.W., 1966. The strength of soils as engineering materials, Sixth Rankine Lecture Geotechnique 16: 91-130.

Bishop, A.W., and H.T. Lovenbury, 1969. Creep characteristics of two undistrubed clays. Proc.7th Int.Conf.Soil Mech.Found.Eng., Mexico, 1: 29-37

Šuklje, L., 1967. Common factors controlling the consolidation and the failure of soil. Proc.Geotechn.Conf.Oslo 1967, 1:153-158.

Šuklje, L. and S. Vidmar, 1961-a. Essais sur les effects provoques par la retenue du fluage des sols. Proc.5th Int.Conf.Soil Mech.Found Eng., Paris, 2: 485-492.

Šuklje, L., and S. Vidmar, 1961-b. A landslide due to long term creep. Proc. 5th Int. Conf. Soil Mech. Found. Eng., Paris, 2: 727-736.

Šuklje, L., L. Marczal and J. Podržajeva, 1968. Dependence of earth pressures on strains in Rankine's case. Acta Geotechnica, University of Ljubljana, No 22: 1-19

Vidmar, S., 1963. Relaxation effects on the earth pressure of cohesive soils. Hung. Acad. of Sciences, Proc. Int. Conf. Soil Mech. Found. Eng., Budapest, 103-118.

Vidmar, S., 1968. Relaxation effects on earth pressures of non-cohesive soils. Acta Geotechnica, University of Ljubljana, No. 21: 1-25.

Chairman Dr. G. P. Tschebotarioff (USA)

Thank you very much Mr. Vidmar.

Now we shall listen Mr. Hanrahan from India.

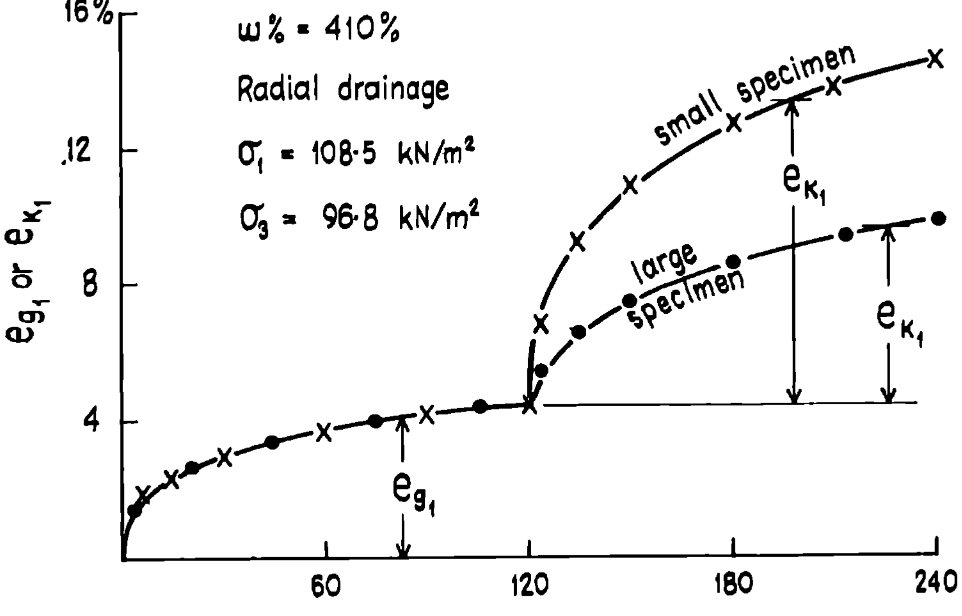
Mr. E. T. Hanrahan (Ireland)

In order to solve the problem of the lateral pressure exerted by clay soils on structures it is first necessary to establish the appropriate relationship between stress, strain and time. It is essential to bear in mind that this relationship is not a material property. It varies with size of specimen and for every element in a deposit or stratum.

This may be illustrated by referring to Fig. 1. In the upper diagram two separate curves of principal strain versus time are shown for a large and a small specimen. The material properties and conditions of test are the same for both with the single exception of the dimension. Now suppose that the measurements have been made on a small specimen, an adjustment for scale must be made to obtain the correct relationship which pertains to the large specimen. The method usually employed for making this correction is inadequate; as may be seen from the slide. Here it can readily be shown that each of the curves in the upper diagram is compounded from two curves shown separately in the lower diagram. One of these curves is quite independant of scale effects, i.e. the curve which describes the principal strain caused by principal stress-difference.

The other curve is highly dependent on dimension and length of drainage path. It follows that basic measurements of the two types of strain must be made, and separate scaling factors applied to each, before a relationship derived from one size is applicable to another size.

The three quantities, stress, strain and time, are uniquely inter-related for each size of specimen. Thus, if two of the three are specified, say for example, stress and time, the third quantity strain, is then automatically defined.



Observed e_{k1} and e_{g1} Strain-components versus Time for two sizes of specimen, under constant stress, during undrained phase followed drained phase

If the strain, thus determined,is not compatible with the deformation of the support an alteration of stress must,of necessity, take place. However,if the laboratory measurements have been made over the appropriate range of stress,it is a simple matter to predict the correct value of stress.

The upper curves in Slide 1 show the major principal strain.Similar curves showing minor principal strain can be drawn. In this instance the two components of strain are of opposite sign.A revised definition of zero lateral strain is that one component of lateral strain is exactly equal the other.The condition of uniaxial consolidation is that the two components of lateral strain exactly balance each other at all times throughout the loading period.The continuously changing stress required to bring about this condition may be readily estimated from the e_g and e_k parameters. From this estimation the earth pressure against a rigid support or a support of known deformation may be obtained.

Chairman Dr.G.P.Tschebotarioff (USA)

Thank you very much Mr. Hanrahan for your contribution. Now I want to pass the word to Mr. Pruška (ČSGR)

Mr.Lumir Pruška (ČSSR)

Mister Chairman,Ladies and Gentleman!

In the short discussion time I like to turn your attention to one practical application of results presented in the written contribution to this session.

Firstly let us look at the known fact that the relationship σ_3' versus σ_1' forms a hysteresis loop when the pressure is at rest and when the leading principal stress σ_1' varies from zero to a certain maximum $\sigma_{1\,max}'$ and again back to zero. This loop has been ascertained and proved experimentally. Fig.1 shows the first hysteresis loop measured by Kjellman in 1936 and calculated by author (Pruška 1973) for the identical boundary conditions.

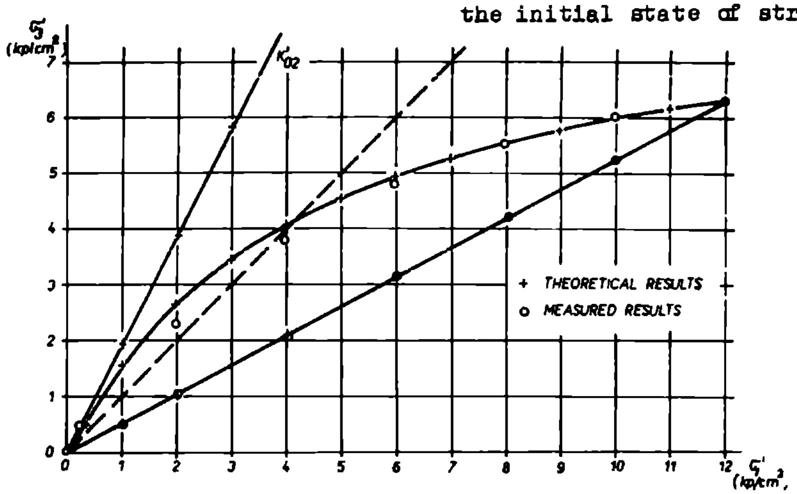
Under other boundary condition,e.g.for cylindrical specimens,the hydteresis loop, though not the same,is of a similar shape. An example of such hysteresis loops is in Fig.2. The presented hysteresis loops were measured by three authors in different contries and in different years (Fjodorov,Malyshev 1959, Plehm 1965, Mach 1970) on samples of sand having almost the same mechanical properties. The specimens,too,were similar, cylindrical,with a 2:1 height-to-diameter ratio.

From the example goes out that if we continue in measurement of hysteresis loops for different max.stresses σ_1' and if these max. stresses σ_1' increase up to infinitum,the hysteresis loops will fullfill a whole zone of pressure at rest (Pruška 1972).This zone is bordered by the lower and upper limits described in the written contribution to this section.

Now let us turn our mind to pressures on retaining structures.The dependence of the actual pressure on wall movements and its relation to the active and passive pressures is frequently stated by help of the well known curve (Fig.3a) indicating that in the modelling of soil pressures on a vertical wall,the active pressure arises at a support displacement from the original position of about 0.002 H. The passive pressure originates at a displacement of about 0.010 H.

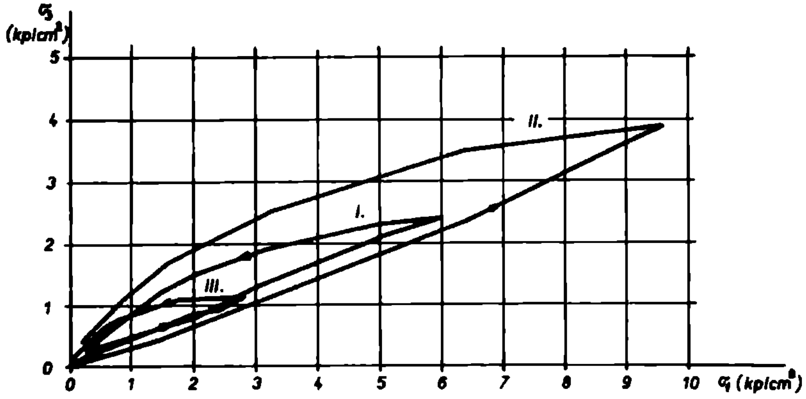
Measurements made at "Centre Experimental de Recherches et d'Etudes des Batiment et des Travaux Public-Paris" (Tchong,Iseux 1972) have proved this dependence to be not unique, for the passive pressure arises also under other deformations and the displacements mobilizing the passive pressure are not merely a function of the supporting wall height.

Let us introduce the relationships set forth in the beginning of this contribution into the problem of pressures on retaining structures in accordance with Fig.3a in which the initial state of stress at zero strains



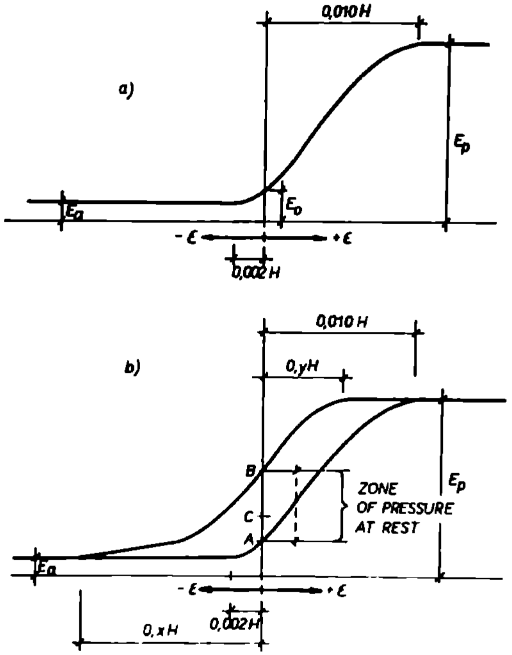
HYSTERESIS LOOP OF SAND IN A CUBIC SAMPLE AS MEASURED BY KJELLMAN 1936(o) AND CALCULATED BY AUTHOR(+)

Fig.I



HYSTERESIS LOOPS OF SAND AS MEASURED BY FJODOROV AND MALYSHEV (I.), PLEHM (II.) AND MACH (III.)

Fig. 2



RELATION OF PRESSURE AT REST TO THE ACTIVE AND PASSIVE EARTH PRESSURE. FIG. a THE TRADITIONAL DEFINITION, FIG b THE NEW EXTENDED DEFINITION

FIG 3

denotes the pressure at rest defined by a single value, E_o. Since the pressure at rest is not a constant, it is necessary to consider - rather than a single value of the pressure at rest - a whole set of pressures at rest lying in the interval bounded by the lower and the upper coefficient of pressure at rest, K_{01} and K_{02}.

What the introduction of this procedure into the schematic diagram in Fig.3a means, is that for zero strain there applies the line segment AB (Fig.3b) expressing the various stresses from which one can set out when examining the origin of the active or passive pressure. It then follows from the continuity of strains that there exist various ways - given by the initial conditions - of reaching the limit stress and therefore, that it is necessary to introduce in place of a single curve according to Fig.3a, some unknown, not closely defined zone according to Fig.3b. What we know of this zone are points A and B and value of active and passive pressures.

The limit values of the pressure at rest are functions of the angle of internal friction and cohesion. For $\phi'=0^\circ$ and $C'=0$ the two values become identical and that is why they are represented by a single point C in Fig.3b. It follows from this statement that actual pressures and the limit strains belonging to the active and passive pressures depend equally on the angle of internal friction, on cohesion and on the stress history.

The aim of this contribution is to point out the necessity of following the initial state of stress and the stress history in soil tests. Respecting this reality leads to surprising conclusions as demonstrated by Broms (1972) in his study of soil pressures on supporting walls.

REFERENCES:

Broms B.(1971), "Lateral Earth Pressures due to Compaction of Cohesionless Soils", Proceedings 4th Budapest Conference on SMFE, str. 373-384.

Fjodorov I.V., Malyšev M.V.(1959), "O bokovom davleniji v pesčanych gruntach". Gidrotechničeskoje strojitelstvo, roč.23, č.6. str.18-22.

Kjellman W.(1936), "Report on an Apparatus for Consumate Investigation of the Mechanical Properties of Soils", Proceedings ICSMFE, Cambridge, Mass. Vol.II, str.16-20.

Mach V. (1960), "Laboratorni zkoumani vlivu opakovaneho zatežovani sypkych zemin na hodnotu jejich součinitele bočniho tlaku v klidu". Stavebnicky časopis,roč.XVIII, č.5 str.361-375.

Plehm H. (1965), "Rohrdruckversuche mit sandigen und kiesigen Erdstoffen".Mitteilungen der Forschungsanstalt für Schiffahrt,Wasser-und Grundbau,Berlin,Schriftenreihe Wasser-und Grundbau,Heft 14,s.104-137.

Pruška L. (1972b), "Basic Equations of Pressure at Rest of Granula Materials".Proceedings 5th European CSMFE,Madrid,vol.I, p.69-76.

Pruška L.(1973) "Hysteresis of Stresses in Granular Media if Pressures are at Rest" (in Czech).Stavebnicky časopis (in print).

Tcheng Y.,Iseux J.(1972, "Essais de Butee en Vraie Grandeur et Contraintes Engendrees par une Surcharge Rectangulaire sur un Mur Vertical" Proceedings 5th European CSMFE, Madrid, vol.I, p.207-214.

Chairman Dr.G.P.Tschebotarioff (USA)

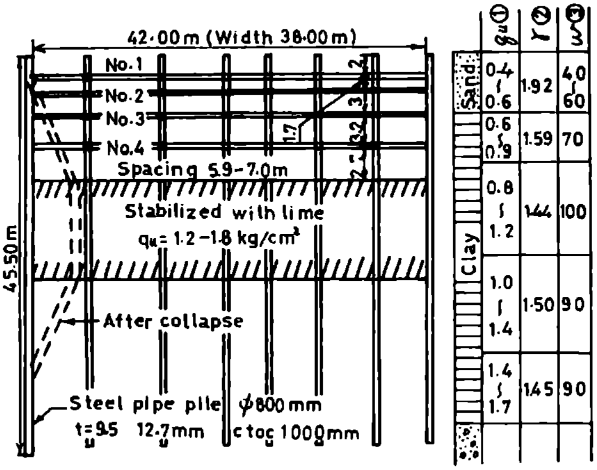
Thank your very much Mr. Pruška. Now we shall listen to Mr. Fukuoka from Japan. Mr. Fukuoka, will you please.

Mr.M.Fukuoka (Japan)

Mr.Chairman,ladies and gentlemen:

There are many field observations concerning braced cuts in Japan. The design criteria has been made on the basis of field data. Nevertheless,there have been some failures of the open cuts which were designed carefully according to these design criteria.Collapse of the braced cuts are characterized by buckling of struts and large deformation of sheet piles with practically no heaving at the bottom of excavation. The cause of the catastrophes are mainly explained as follows. Sheet piles begin to rotate,because of deformation characteristics of the soil below the bottom of excavation, and as a result the lowest row of struts is subjected to an extremely large load. The backfill pressure becomes very large,and the inside pressure at the bottom of excavation becomes very small because of pore water pressure in clayey soil,which probably contains sand seams.This may also result in an extremely large load on the lowest row of struts. It is quite difficult to incorporate the collapse mechanism into a practical design. Therefore,it seems best at present to proceed cautiously with the construction work while the forces in the struts,the deformation of the walls, and the pore water pressure are measured at frequent intervals.

I would like to show you some figures concerning to two examples of collapse.



q_u ① : Unconfined compressive strength q_u(kg/cm²)

γ ② : Unit weight γ(t/m³)

w ③ : Water content w(%)

Fig.I

Fig.2

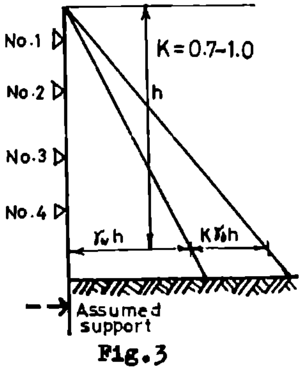


Fig.3

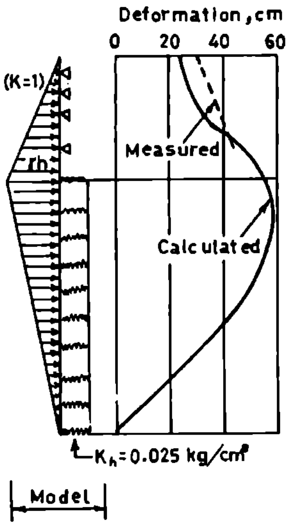


Fig.4

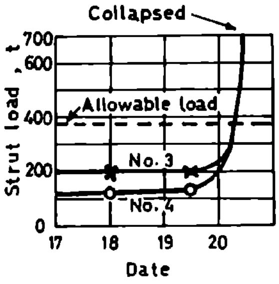


Fig.5

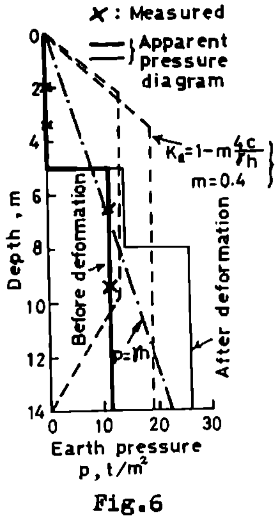


Fig.6

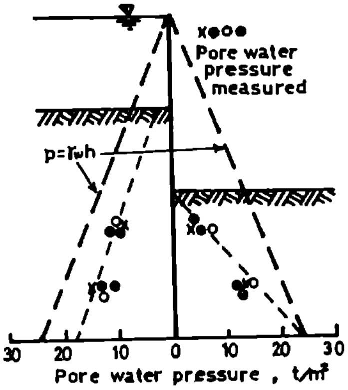


Fig.7

Chairman Dr.G.P.Tschebotarioff (USA)

Thank you Mr.Fukuoka for your discussion. The time has come for me to make my concluding remarks. I was very glad that Professor Peck agreed with Dr.Duncan's statement accepting the need to differentiate between sliding phenomena and wedging phenomena in the case of different types of arching. As to just what terminology should be used, I think that the best thing will be to refer a decision on this matter to the Committee on Nomenclature of this Society.

We have also had very valuable discussions on the matter of braced cuts and I am sure that from the remarks of Professor Peck, Mr.Karol Klein and Mr.Ivar Foss new ideas will come forward to improve our present methods which definitely are not entirely satisfactory.

I still believe that our future progress will greatly depend on the development of instrumentation which will permit us to measure in-situ the variations in the lateral pressures of clay soils over long periods of time.

I think that the studies reported from Norway and from France are a step in the right direction and will be very useful in clarifying our thoughts on the matter.

Very interesting contributions were also made on the matter of lateral pressures agains piles. Both Professor Marche and Mr. Moser have spoken on relevant but slightly different topics, but both will be very important.

There have been quite a number of theoretical contributions which I will not attempt to analyze,- this is not my strong point,- but I am sure that theoreticians will have enjoyed all the presentations they have heard.

I would like to thank all the contributors to this Session, especially our Deputy Chairmen and our Recording Secretary.

In conclusion, I would like to say a few personal words. At my age, 74, it is only reasonable to assume that this is the last time I appear in an active capacity at an international engineering gathering and I am particularly happy that I was given the opportunity to do so on the soil of my native land. I would like to cordially thank my colleagues on the Soviet Organizing Committee for having invited me to do so.

May I repeat this in my native language.

Т.к.мне 74 года, только разумно ожидать,что это мое последнее выступление в активной роли на на международном инженерном собрании.Поэтому я попросил разрешения повторить на моем родном языке мою сердечную благодарность всем моим Советским коллегам Организационного Комитета за их любезное приглашение меня председательствовать на этом заседании на родной для меня земле.

The Specialty Session 0 No.5 is closed.

Thank you.

WRITTEN CONTRIBUTIONS

EARTH PRESSURES ON RETAINING WALLS IN EXPANSIVE CLAY.

G.D. Aitchison, M. Kurzeme, B.G. Richards (Australia).

1. METHODS OF ESTIMATING LATERAL PRESSURES

The conventional methods of estimating lateral pressures on retaining walls can be broadly classified into three groups - elastic methods, limit equilibrium methods and empirical methods.

Elastic methods are applicable where small deformations are expected and estimates of displacements or load-deformation response are required. Limit equilibrium methods are more applicable where displacements are not critical, provided they are tolerable and estimates of maximum lateral pressures are required. Empirical methods find application where large deformations occur and are generally restricted to particular classes of structures (e.g. sheet pile walls).

When expansive clays are encountered the problem of estimating lateral pressures is further compounded by the moisture reactive nature of the soil. The introduction or removal of moisture may cause a marked increase or decrease in lateral pressure as compared to an inert soil under the same conditions. Further, the presence of major planar discontinuities within the soil mass may dominate the behaviour of the soil-wall system, irrespective of other factors.

2. MODES OF BEHAVIOUR OF WALLS IN EXPANSIVE CLAY

A number of distinctly different modes of behaviour of retaining walls in expansive clays have been observed in Australia. These appear to have resulted from the introduction of the structure causing a decrease in soil moisture, or an increase in soil moisture, or having mobilised movement along a planar discontinuity.

Removal of moisture from the soil mass may result from the introduction of an air-conditioned basement. The moisture, transported through the basement retaining wall, causes the clay to shrink away from the wall, resulting in zero lateral pressure, or perhaps an active pressure condition.

Alternatively, water may collect in the inevitable gap between the structure and the clay face, causing swelling and a rise in pressure. Walls free to move a moderate amount respond to this pressure but walls with restricted movement may suffer a rise in pressure until the wall fails or the soil fails in a passive mode.

The presence of an unfavourable planar discontinuity may give rise to a modified active (or passive) pressure situation with a defined failure plane. The appropriate shear resistance, mobilized in such a case, would be the residual shear strength of the joint material.

3. MEASUREMENTS OF LATERAL PRESSURES IN EXPANSIVE CLAY

In Adelaide, Australia, a section of a rigid basement retaining wall, 25 m long and 7.5 m high has been instrumented to observe lateral pressures (Kurzeme and Richards, 1973). The structure is located in moisture sensitive Hindmarsh clay. The instrumentation comprises of 24 earth pressure cells (four rows by six columns of cells) at the wall/clay interface, and 31 psychrometers in the clay mass behind the wall.

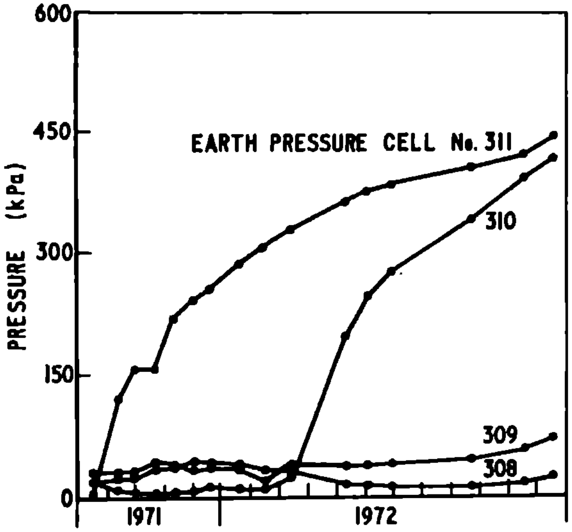


Fig. 1 - Indicated total earth pressures, cells 308-311.

Over the eighteen months of observation the bottom row of cells (e.g. cell No. 311, depth 7 m) has shown a progressive increase in pressure from the time of installation (Fig. 1). The present readings indicate a lateral pressure of the order of four times the overburden pressure at this depth. The second bottom row (depth 6 m) commenced a progressive increase in pressure five months after installation, with present indicated pressures approaching those of the bottom row. The two top rows of cells (depth 4.1 m and 1.8 m) have shown no significant change. The psychrometers in the clay mass have not indicated any systematic change in the moisture environment of the clay mass. The nearest row of psychrometers is 2 m from the wall.

The mechanism operating in this case appears to be that indicated in 2(b). During construction free water was encountered at the surface of the clay (depth 3 m) and it is postulated that this water is collecting against the wall, causing local swelling and a rise in lateral pressure. The swelling appears to be progressing upwards, as the gap is progressively sealed off to further access of the water.

4. CHOICE OF DESIGN METHOD

It is seen that the choice of design method to use in a particular situation is not easy. Such factors as soil type and properties (expansive or non-expansive), and wall type (movable or immovable) are relatively easy to define. It is more difficult to assess soil structure (planar discontinuities) and the existence of free water. However, the two factors of least control at the design stage and being probably most critical in expansive clays, are construction procedures and subsequent use.

REFERENCES:

KURZEME, M. and RICHARDS, B.G. (1973), "Earth Pressure Observations on a Retaining Wall in Expansive Clay - Gouger Street Mail Exchange, Adelaide". CSIRO, Division of Applied Geomechanics, Technical Report No. 17.

ON A COMMON WORK OF PILING WALLS AND CREEPING FOUNDATIONS. A. Ya. Budin (U.S.S.R.)

The work of bulkhead piling walls on creeping foundations is distinguished by distinctly expressed features, the integral effect of which is the increase of efforts acting in them as a result, in certain cases the structures calculated on a method which does not take account of foundations creep, eventually come into wrench state, and in other cases they do not lose their operational characteristics due their overrated safety factors. More often, as shown in Fig. 1, when a driven part of the wall cuts a layer of creeping soil and by its lower end enters sublaying uncreeping soil, a mechanism of growing efforts in the structure consists in that a reaction pressure of creeping soil on the wall relaxes. When disigning piling bulkhead walls on the creeping foundations, it is necessary to evaluate their durability. For this purpose it is necessary to construct an epure of soil reaction pressure corresponding to an age of the structure, at which its carrying capacity can be exhausted. The dependence for evaluating an intensity of a reaction pressure of creeping soil on the piling wall as a function of time, rheological and durable characteristics of soils and parameters of structures are found by means of a common decision of equation describing the creep of a soil semizone consisting of a "plastic" soil (according to N. N. Maslov) and a differential equation of a wall elastic line. The relaxation regularity of reaction contact soil on the wall, rigidly pinched in sublaying uncreeping soil, is described by the equation of paradolic type

$$\frac{\partial^2 p(x,t)}{\partial x^2} = a^2 \frac{dp(x,t)}{\partial t} , \qquad (1)$$

where $a^2 = H\eta/EI$,

H is a depth of creeping soil layer,

η is a coefficient of coil viscosity during the secondary creep,

EI is a rigidity of piling wall.

The decision of Equation (1) depends on the nature of initial reaction load $p(x,0)$. At $p(x,0)=\gamma\lambda(H-x)$ where λ is a coefficient of soil reaction pressure, we obtain the following equation:

$$p(x,t)=\frac{2\gamma\lambda H}{\pi}\sum_{n=1}^{\infty}\frac{1}{n}\exp\left\{-\frac{n^2\pi^2 EI}{H^2\eta}t\right\}\sin\frac{n\pi}{H}x . \qquad (2)$$

As will be seen from Fig. 1, a scheme of anchored wall, the lower part of which, having a length and driven into uncreeping soil, possesses an elastic pliability, offers the greatest interest for practical purposes. In this case a relaxation regularity is described by equation.

$$\frac{EI}{H\eta}\int_0^x dx\int_x^H dx\int_0^t p(x,t)\,dt = \int_0^x dx\int_0^x \left[M(x,0)-M(x,t)\right]dx -$$

$$-\int_0^x dx\int_0^x \Delta R(t)\ell(x)\,dx + \frac{EI}{K\ell^2}\Big\{\big[M(0,0)-M(0,t)-\Delta R(t)$$

$$(L-h)\big]f_1(x)+\left[Q(0,0)-Q(0,t)-\Delta R(t)\right]f_2(x)\Big\} , \qquad (3)$$

where M and Q are bending moment and cutting force in the wall, correspondingly, expressed bv $p(x,t)$ and $p(x,0)$, $\Delta R(t)$ is an increase of anchor effort during a period of time t, K is a coefficient of bed pliability of uncreeping soil $L, h, \ell(x)$ are geometrical dimensions, shown in Fig. 1, $f_1(x)$ $f_2(x)$ are functions depending in ratios of structure geometrical dimensions. We obtain the following decision of Equation (3) corresponding to a linear outline of initial reaction load epures:

$$p(x,t)=p(x,0)\exp\left\{\frac{-EIK\ell^2 t}{H\eta\left\{\left[F(x)-B(x)\right]K\ell^2+\Phi(x)EI\right.}\right\} , \qquad (4)$$

where $F(x)$, $B(x)$, $\Phi(x)$ are functions depending on the nature of initial reaction load $p(x,0)$ and parameters of structures the values of which are determined from nomogramms. The natural studies of work of piling bulkhead walls on creeping foundations are carried out on three units. The units of studies (berthing quays) are reinforced concrete gantry bollwerk, reinforced concrete and steel single anchors bollwerks. Duration of observations for these units is 9.7 and 5 years, correspondingly. In the process of studies we measured structures displacements, bending flexures of walls elastic line, pressure of soil on walls, bending moments, axial efforts of anchor tods and piles. The obtained natural satisfactorily coincide with the results of formula (4).

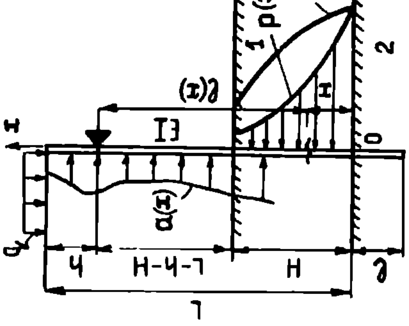


Fig. 1

1 - creep ground, 2 - uncreep ground

THE EFFECT OF STRESS RELAXATION ON THE LATERAL EARTH PRESSURE OF COHESIVE SOILS.

Hiroshi Fujimoto /Japan/

A change with time elapsed of lateral earth pressure on a retaining wall is a very important problem. However, there are only the suggestive explanation by D.W.Taylor (1956) and the experimental research by S.Vidmar (1963) about the problem, and a report that discussed theoretically about the mechanism of the change with time elapsed of lateral earth pressure has not yet been published. In this summary, a result of the authorial consideration on the abovementioned mechanism will be presented.

Now, let us consider the case that a wall is displaced in the outside direction from back-fill soil caused by a creep of cohesive soil or an earthquake etc. Then, a shearing strain ε_{rs} occurs at stage before reaching the critical state of yielding on a plane inclined with angle α_c to horizontal plane in the back-fill soil. Thus, if the displacement of wall is once fixed, a shearing stress τ corresponded to ε_{rs} must undergoes slow relaxation.

That is, let t=0 be the time when the displacement of wall is fixed, the shear stress $\tau_{(t=0)}$ at that time is represented by following equation.

$$\tau_{(t=0)}=\frac{a_1(const)-a_3(t=0)}{2}\sin 2\alpha_c=\frac{P_d(t=0)}{2}\cdot\sin 2\alpha_c \ldots \qquad (1)$$

where $a_1=\gamma Z_c$ is the major principal stress at the certain depth Z_c and $a_{3(t=0)}$ is the minor principal stress, i.e., the intensity of lateral earth pressure at time t=0. Therefore, the shear strain ε_{rs} is kept constant since the wall is fixed, then, $\tau_{(t=0)}$ undergoes slow relaxation from time t=0. The shear stress (t) at any time t=t is

$$\tau(t)=\frac{a_1(const)-a_3(t)}{2}\sin 2\alpha_c=\frac{P_d(t)}{2}\sin 2\alpha_c \qquad (2)$$

Consequently, since $a_1(const)$ and α_c are constant values, the intensity of active lateral earth pressure $a_{3(t)}$ must gradually increases to maintain the equilibrium state expressed by the eq.(2) with relaxation of $\tau(t)$

For the passive state, it is satisfactory to consider us as follows. That is, supposing the passive state as wall is moved against the back-fill soil, $a_3(t)$ becomes greater than $a_1(const)$. Now, if the displacement of wall is once fixed, since ε_{rs} is kept constant the shear stress $\tau_{(t)}$ undergoes slow relaxation. In this case, since $a_{3(t)}$ becomes greater than $a_{1(const)}$ and $\tau_{(t)}$ must be positive, the eq.(1) and (2) are respectively as follows.

$$\tau(t=0)=\frac{a_3(t=0)-a_1(const)}{2}\sin 2\alpha_c \qquad (3)$$

and

$$\tau(t)=\frac{a_3(t)-a_1(const)}{2}\sin 2\alpha_c \qquad (4)$$

According to the eq.(4), $\tau_{(t)}$ decreases so far as $a_1(const)$ and α_c are kept constant. Consequently, the intensity of the lateral passive earth pressure $a_3(t)$ must gradually decreases from value of $a_3(t=0)$ to maintain the equilibrium of the eq.(4). In other words, the lateral passive earth pressure with time elapsed in contrast with active state.

In case of triaxial stress relaxation test, if the stress measured is the deviator tress $P_d(t)=a_{1(t)}-a_3$ and a_3 is kept constant, the principal stress ratio $a_3/a_{1(t)}$ under condition of $a_{1(t)}>a_3$ is corresponding to the apparent coefficient of active earth pressure. Thus, let $K_{a(t)}$ be the above principal stress ratio,

$$K_a(t)\equiv\frac{a_3(const)}{a_1(t)}=\frac{a_3(const)}{P_d(t)+a_3(const)} \qquad (5)$$

Therefore, if $P_{d(t)}$ decreases with time elapsed in the above equation, $K_{a(t)}$ increases with time from $K_{a(t=0)}$ at the initial time of relaxation.

The data of variation with time elapsed of $K_{a(t)}$ obtained from the results of triaxial stress relaxation tests on the compacted cohesive soils by the author (1969) are presented in Fig.1.

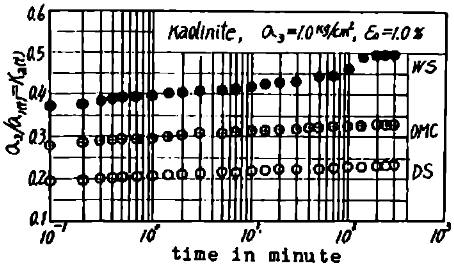


Fig.1 Variation of $K_{a(t)}$ with time elapsed, obtained by meanse of tri-axial stress relaxation test.

From these results, it was found that the increasing ratio of $K_{a(t)}$ of WS-specimen (Compacted in wet-side of compaction curve of soil sample) is highest and that of DS-specimen (Compacted in dry-side) is lowest.

In case of passive state, since $K_{p(t)}$ under condition of $a_3>a_1(t)$, it is represented by following equation.

$$K_{p(t)}=\frac{1}{K_{a(t)}}=\frac{P_{d(t)}+a_3(const)}{a_3(const)} \qquad (6)$$

Accordingly, if a_3 is kept constant, $K_{p(t)}$ decreases with relaxation of $P_{d(t)}$.

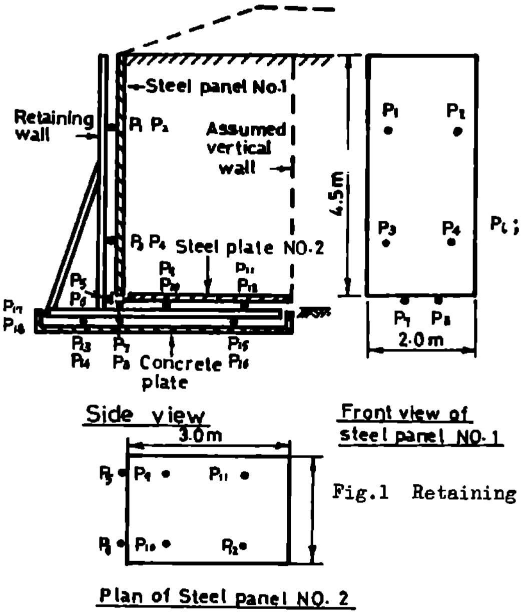
REFERENCES:

1. D.W.Taylor(1956),"Fundamentals of Soil Mechanics",pp.522-524.
2. S.Vidmar (1963),"Relaxation Effect on the Earth Pressure of Cohesive Soils",Proc. of TCSM & FE,Hungarian Academy of Science, pp.103-117.
3. H.Fujimoto (1969),"Studies on the Stress Relaxation of Compacted Cohesive Soils and Its Application to Engineering Practice", Memoirs of Faculty of Engineering,Miyazaki University,No,6, pp.53-147.

MEASUREMENTS OF EARTH PRESSURE ON A RETAINING WALL WITH CLAYEY BACKFILL M.FUKUOKA (JAPAN)

The retaining wall was constructed firmly using the H-shaped steel beams (Fig.1). A cohesive soil classified as the group 3 by Terzaghi and Peck was used as the backfill. The steel panel was fixed on the vertical wall, and the horizontal pressure and the wall friction were measured with the load cells. The second steel panel was fixed on the base plate. The soil reaction under the retaining wall was taken by the concrete slab, and it was measured with the load cells, also. P. 1, 2, 3, 4, 9, 10, 11, 12, 13, 14, 15 and 16 in Fig.1 show the load cells for vertical pressures, and p. 5, 6, 7, 8, 17 and 18 those for frictional forces. The 16-ton bulldozer was used to move and compact the backfill 30 cm thick layer by layer. Table 1 shows the result of the tests on the backfill and foundation. The horizontal earth pressure on the vertical wall should be zero according to the Coulomb's formula. Nevertheless, the fairly large pressure acting upon the vertical wall was observed (Fig.2). The total vertical pressure on the base plate is less than the weight of the soil mass on it. The point of application of the resultant pressure is situated backward from the center of the panel. The wall friction is acting on the vertical wall in spite of the level surface of the backfill. The wall friction is acting on the base plate, too. The reaction from the base is almost balanced with the backfill pressure. The difference between the two forces seems to be due to the difficulty in keeping the two dimensionality, and the error involved in the measuring apparatus. The retaining wall moved as the construction work went on, and finally amounted to 9 mm horizontally and 4 mm vertically. The coefficients of subgrade reaction are Kv=1.187 kg/cm^3 vertically and Kh=0.232 kg/cm^3 horizontally. The coefficient of subgrade reaction using the 30 cm in diameter plate is K_{30}=6 kg/cm^3, as shown in Table 1. The ratio K_{30} to Kv is 5, while the ratio Kh to Kv is 0.2. Therefore, the horizontal displacement is large. In spite of the surcharge as high as 1 m as shown in Fig.2, very little increase was observed in the pressure and wall friction of the vertical wall. The horizontal displacement of the wall was also very little. When artificial rainfall was applied over the surface of surcharge, the horizontal pressure on the vertical wall decreased very little. Earth pressure changed with time, but its magnitude was not large. The bending moments of vertical wall and horizontal base plate are almost the same irrespective of the distribution of pressure. Therefore, the test of this kind, which gives the resultant forces instead of the pressure distribution, is useful for design purposes. The force on the vertical wall, which is assumed for the stability analysis, is easily obtained by this method.

The large model retaining wall 6 m high (with no base plate) was constructed in the laboratory at the Public Works Research Institute, Ministry of Construction. The test has been performed for many years. The test results with cohesive soils gave the similar results as stated above. The point of application of the resultant force on the vertical wall is about one third of the wall height, while the point of application for the cohesive soil is much higher.



Side view

Front view of steel panel NO-1

Plan of Steel panel NO. 2

Fig.1 Retaining Wall

Table 1 Result of soil testing

Backfill	Triaxial test c= 3.0 t/m^2, ϕ = 12 degrees (mean value) Unconfined compression test q_u = 0.3~0.7 kg/cm^2 W_L = 32.2%, W_P = 24.8%, W = 22.7%, CBR = 3 γ=1.867 t/m^3 G = 2.61 Plate bearing test, diameter of bearing plate 30cm K_{30}=9 kg/cm^3
Foundation	Sand, Ground water Level -1.0 m K_{30} = 6 kg/cm^3

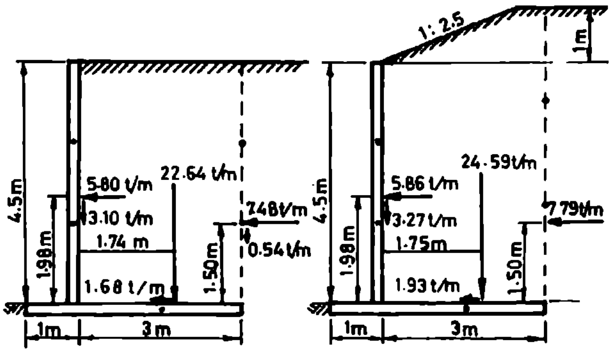


Fig.2 Resultant pressures

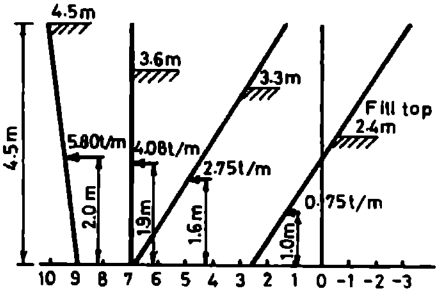


Fig.3 Displacement of vertical wall(mm)

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DISPLACEMENT MEASUREMENTS OF A MULTI-ROW ANCHORED BORED PILE RETAINING WALL.

E.T.HAWS, R.TABB /ENGLAND/

The construction of the foundations for the National Westminster Bank's new Head Office in London necessitated that an excavation, through Thames Gravel into the underlying London Clay, be taken some 12m below the foundations of adjacent 9 storey buildings. The heavy surcharge, together with restrictions on the allowable wall lateral deflection, required that a rigorous design procedure be followed. The wall and ground anchor system were modelled mathematically using a beam on elastic foundation approach and a stage by stage analysis was performed to simulate the excavation sequences. Wall bending moments, anchor loads and predicted movements were output by the computer. High values for the lateral earth pressure coefficients were deliberately chosen since the expected displacements were thought insufficient to allow relaxation to Ka conditions. Values of 0.4 and 0.6 were therefore chosen for K values in the gravel and overconsolidated clay.

The analysis showed that spring parameter changes have little effect on bending moment, shear force and anchor force whilst the wall displacements showed near linear response.

The final design made use of a 600mm diameter heavily reinforced bored pile retaining wall toed into the London Clay some 5m below dredge level. Three rows of ground anchors supplied a horizontal force component of approximately 500 kN/m run/row, transmitted through heavy reinforced concrete waling beams.

During the analysis and design it became apparent that there are insufficient case histories for this type of construction. It was therefore decided to monitor the wall behaviour during construction with readings being taken at frequent intervals. The results of this instrumentation were used during construction to assess the validity of design assumptions and they showed that alterations to construction procedure were unnecessary.

The wall instrumentation took three forms; two types of displacement measurements and anchor load measurement. A total of eight 800 kN vibrating wire ground anchor load cells were installed at a total of four vertical sections, two of these being in wall sections with three rows of anchors and two being in wall sections adjacent to a shallow excavation with only one row of anchors. An extensometer line consisting of 17 No.3m bays was installed in the basement of an adjoining building along a line approximately normal to the bored pile wall. Displacement measurements taken to an accuracy of ±1/2mm along this line were transferred to pile head movements at five positions, four of which corresponded to anchor load cell sections. During piling construction, a total of five deflection tubes were concreted into selected piles by first welding the tube to the centre of the reinforcement cage over the full construction depth. The patented displacement tubes were read by sighting onto pins welded to the walls of the 100mm x 100mm box sections. For this purpose a frame was fabricated which attached to each head position and on to which was mounted, in an inverted position, a 1 sec of arc vertical circle theodolite. Repeatability of readings for the upper pins was to within 1/50th mm whilst the lower pins were recorded to within 1/10mm.

The deflection and load cell measurements have illustrated the effectiveness of this method of construction in limiting wall deflection, with only nominal movements occurring at anchor positions after stressing. The importance of providing adequate toe resistance to enable the lower anchors to be installed, without initial large lateral movements, cannot be over emphasised. The final wall deflection curves for the multianchored wall show little or no movement to have occurred at the head position and linear increase in deflection magnitude with depth up to a maximum value averaging 10mm into the excavation at the toe.

Since little or no movement has occurred at anchor positions since stressing the anchor free lengths and thus load should have remained constant. However, almost all the anchors have shown a tendency to lose load at a decreasing rate with time and to this date a loss of some 10% has occurred since stressing. This can probably be attributed to creep within the anchorage and retained ground.

A Finite Element back analysis is at present being performed with sequential modelling of the excavation from Ko conditions and with the effects of anchor stressing included.

A total of 1000 elements are being used in this plane strain analysis to predict lateral pressures at each excavation stage. The first analysis will be carried out using existing published modulus values and the displacements calculated will be compared with those observed in the field. Subsequent analyses will be made with modified moduli aimed at matching the recalculated displacements as closely as possible to field values.

CALCULATION OF COFFERS CONSIDERING THE SOIL REPULSE. V.A.Ivahnuk (USSR)

Coffers for buried chambers are calculated at present both in the USSR and in other countries considering the reactive lateral pressure of soil along the circular sections. Non-uniform lateral pressure of soil due to warpings, heterogeneous depth of soil and other factors are taken into account using the coefficient of irregularity. As a result, the coffers calculated according to this method have in certain cases surplus safety margins or get intolerable deformations (cracks, wrecking states), in other cases.

The analysis of deformations and damages of some coffers, field experiments made during some past years show inadequacy of real working conditions of the coffers and their calculations. The values of lateral pressures of soil measured in situ when warpings take place exceed the design ones (active pressure) by 30-60%, while coefficients of irregularity calculated according to measurements are much more than the normative ones (up to 1.8 against 1.25).

Again, taking into account the reactive lateral pressure of soil is necessary because of the introduction, in the USSR during some last years, of thin-walled prefabricated constructions of coffers from hollow blocks and panels of solid section submerged in thixotropic jackets design of which demands more precise definition of loads.

For the case when soil properties are described by means of the model of bed coefficient and a coffer is the body rigidity of which exceeds that of the surrounding soil, a method of calculating coffers, is developed considering the reactive lateral soil pressure which appears when warping.

Analytical dependences are obtained for estimating the displacement and the value of maximum reactive pressures (IVAHNUK,V.A.,1973).

Using methods of calculation of orthotropic circular cylindrical casing as for transverse load suggested by A.G. Immermann, formulas are obtained for calculating the coffer walls as per circular sections considering the reactive lateral pressure of soil.

Calculations made using the suggested method show a better agreement of the results with field data than the existing one.

However, it is to be noted that many coffers possess final rigidity and calculation as per circular sections does not reflex the operation of structure in the vertical plane. That is why we try to develop a method of calculation of coffers possessing final rigidity considering the reactive lateral pressure of soil. Taking into account complicated stress and strain state of a hollow cylinder (such as f. inst. a coffer), complexity of conditions of his operation in a heterogeneous soil medium, the investigation is being continued in the direction of using the method of final elements for this purpose.

REFERENCE:

IVAHNUK,V.A. (1973), "Coffers in the construction of mining enterprises," The Nedra Publishers, Moscow.

VARIATION IN LATERAL PRESSURE ACTING ON CAST IN SITE DIAPHRAGM WALL AND DISPLACEMENT OF WALL. T.KAWASAKI,Y.IKUTA, Y.MENDE (JAPAN)

In the major cities of Japan,there are many cases when multi-leveled basements and underground structures are constructed at places with poor conditions of the ground. The methods of construction adopted on such occasions incorporate retaining walls of soldier piles,sheet piles,piling walls and cast in site diaphragm walls and caissons. Of these, cases of use of diaphragm walls have increased greatly in recent years. Their methods of application are not limited merely to temporary retaining of earth and the walls have come to be used structurally as permanent elements such as earthquak resisting walls and bearing walls. Therefore,the stresses and deformations of the walls during the periods they are being used as temporary works prior to becoming permanent walls become of importance.

Regarding these problems, earth pressure and water pressure cells on wall surfaces and stress meters inside walls,and at times, inclinometers, are installed for studies through measurements of lateral pressures acting on the walls and deformation-stress.

This report is concerned with the relation between lateral pressures acting on diaphragm walls and the displacement of the walls. The scope of the study covers the stage of Construction of walls underground, the period they are being utilized as retaining walls, and after they ultimately have become basement walls. For ground of cohesive soil, there are detailed measurement data in hand, for projects at five places. The feature of the method of measurement is that earth pressure cells and water pressure cells were installed as wall-surface gages in a state that they were balanced with the flow pressure of concrete on the trench wall surfaces.

From these measurements, explanations may be made of: (1) the fact that the equilibrium state of active lateral pressures and passive lateral pressures during excavation is influenced at the time of construction of the wall, (2) the rate of·change of the lateral pressure coefficient has a relation with the displacement quantity of the wall, and (3) the displacement quantity of the wall is related to the modulus of elasticity below the bottom of the excavation.

DETERMINING LONG-TERM DEFORMATIONS OF QUAYS FOUNDATIONS TAKING ACCOUNT OF SOILS RHEOLOGICAL CHARACTERISTICS.

V. M. Karpov (U.S.S.R.)

The report states a method of evaluating two types of quays: gravity quay and rigid bollwerk working on the scheme of a free support. Two schemes of displacement are examined. According to the first scheme it is assumed that the displacement of quays of the two types occurs along a circular cylindrical surface of radius R with the least stability factor. The soil in the zone of deformation is subject to Schvedov-Bingam's law with a variable viscosity factor according to N. N. Maslov. The epures of shear stress τ and "threshold of creep" τ_{lim} are constructed on the slide arch. We determine the mean value $\Delta\tau$ of exceeding the mean value τ over the mean value τ_{lim}. The power of the deformed zone D is evaluated from expression $D = R_0 - R$ where R_0 is a radius of circle for which $\Delta\tau = 0$ and concentric to the circle of radius R. The value of displacement of points along a slide circle is determined by formula:

$$S_t = \frac{\Delta\tau D}{\mu \eta_\kappa}\left[\ln\left(e^{\mu t} - \frac{\Delta\eta}{\eta_\kappa}\right) - \ln\left(1 - \frac{\Delta\eta}{\eta_\kappa}\right)\right] \qquad (1)$$

where $\Delta\eta = \eta_\kappa - \eta_0$; η_κ; η_0; μ are an initial and final coefficient of viscosity and a creep parameter according to N. N. Maslov. According to the second scheme we determine the displacement of a rigid bollwerk owing to a rotation of the wall around a point of fastening the anchor. The determination is carried out by two ways: in the first way it is supposed that the displacement of the wall occurs owing to soil compaction in the prism of heaving and displacement of the latter in the direction of a slide line. The soil is approximated by the model adopted in the first scheme. The displacement of wall owing to soil compaction in the prism of heaving is defined by formula:

$$S_{max.} = \frac{1}{2} \cdot \frac{m_v \gamma h_0^2}{1 + e^0} \cdot \frac{1 + K_0}{1 + 2K_0} \cdot \frac{K_p \cdot K_a}{tg\psi} \qquad (2)$$

where m_v is a coefficient of volume change of soil; γ is a unit of soil weight; K_0 is a coefficient of earth pressure at rest; K_p and K_a are coefficients of passive and active earth pressure, correspondingly; e is a void ratio; $\psi = 45 - \frac{\varphi}{2}$; φ is an angle of internal friction; h_0 is a depth of wall immersion. The displacement of prism of heaving in the direction of a slide line is defined by formula:

$$S_t = \frac{1}{\mu\eta_\kappa}\left[\ln\left(e^{\mu t} - \frac{\Delta\eta}{\eta_\kappa}\right) - \ln\left(1 - \frac{\Delta\eta}{\eta_\kappa}\right)\right]\left\{\frac{2Q}{\pi}\left[\cos\psi + \sin\psi\, tg\varphi\right]\left(\frac{D}{b}\, arctg\frac{b}{D} + \frac{1}{2}\ln\frac{D^2 + b^2}{b^2}\right) + \frac{2QD^2}{\pi b^2}\cos\psi \ln D - \gamma\cos\psi\, tg\varphi\left(\frac{D^2}{2}\cos\psi + h_0 D\right)\right\} \qquad (3)$$

where Q is a summary (total) epure of wall reaction pressure on the soil; b is a length of the prism heaving base; D is a distance from the bottom of wall to the limit of zone where $\tau = \tau_{lim}$. In the second way the soil is approximated by the model of linear-deformed medium and "hereditary" medium according to G. N. Maslov- N. H. Aroutunjan with the extent of creep proposed by V. A. Florin for the soils. The determination of wall slide is carried out by the way of integration along the height of wall h_0 from the decision of M. I. Gorbounov - Possadov for a concentrated force inside of an elastic semiplane. The constant of integration is defined from a condition that on the depth equal to double depth of wall, crank efforts and therefore, relative deformations of distortion along the axis oz are highly small and can be neglected. To take account of continuity rupture we introduce with some reserve the coefficient 2.0 into the final expression of displacements. The displacement value of wall bottom is defined by formula:

$$S = \frac{Q}{\pi E_1}\left(\alpha + \beta \ln h_0\right) \qquad (4)$$

where Q is a summary (total) epure of wall pressure on the soil; E_1 is a modulus of soil deformation for a plane

$$\alpha = 4 + (1 - K_0)^2; \quad \beta = (1 - K_0)^2$$

According to V. A. Florin, for the soil having rheological characteristics, the value of creep extent in the form of

$$c(t,\tau) = \frac{1}{E_0}\left(1 - e^{-\lambda t}\right)$$

is substituted for the value $\frac{1}{E_1}$ into expression (4) Thus

$$S_t = \frac{Q}{\pi E_0}\left(1 - e^{-\lambda t}\right)\left(\alpha + \beta \ln h_0\right) \qquad (5)$$

where E_0; λ are creep parameters; t is a time. In the work we give an example of determining quay wall displacements of a rigid bollwerk type at the base of which there are clayish silts, having:

η_0 = 550 kg·day/sqcm; η_κ = 7250 kg·day/sqcm;
μ = 0.024 1/day; E_0 = 19.4 kg/sqcm;
λ = 0.0022 1/day;

The curve determined from formula (5) has the best coincidence with observed values. However the determinations of other structures displacements showed that in some cases the curves drawn according to formula (1) or (2 and 3) have the best coincidence with results of observations. This is explained that one of considered soil models and a scheme of structure displacements corresponds better to each specific structure in the different soil conditions. Therefore in the present stage of studying problem, the forecast of wall displacements must be determined by all three methods, and it is necessary to take the greatest displacement for the calculated one. In the future the perfection of the above methods will permit to determine quays foundations according to the second limit state (by deformations) which is especially important for soft clay soils.

THE CONTRIBUTION TO THE DETERMINATION OF THE INFLUENCE OF COHESION ON THE LATERAL SOIL PRESSURE. L.Pruška (Czechoslovakia)

It is suitable to start the study of the influence of cohesion on the lateral soil pressure with the basic case and to identify both the lateral and vertical stresses with principal stresses at conditions of the pressure at rest. The basic equations for this case by cohesionless soils and for cubic soil samples have been published by author (1972a). It has been stated that the coefficient of pressure at rest isn't a constant, but that it varies in dependence on the stress history in a zone bordered by the lower resp. upper limit

$$\sigma_3' = K_{01} \cdot \sigma_1' \qquad (1)$$

resp.

$$\sigma_3' = K_{02} \cdot \sigma_1' \qquad (2)$$

where

$$K_{01} = tn\left(\frac{\pi}{4} - \frac{\phi'}{2}\right) \qquad (3)$$

$$K_{02} = tn\left(\frac{\pi}{4} + \frac{\phi'}{2}\right) \qquad (4)$$

The equ.3 and 4 are valid for $\phi' =$ const. The principal stress directions are const., therefore at the upper limit $\sigma_1' < \sigma_3'$. The lower limit has been reached when the pressure increases from zero. The upper limit has been reached when the pressure decreases from infinitum. The actual coefficient of pressure at rest is given, when the pressure decreased from $\sigma_{1\,max}'$ on the lower limit to (the preconsolidation ratio $\sigma_{1\,max}'/\sigma_1' = Pr$) by (Pruška 1972b)

$$K_0 = \frac{K_{01} \cdot Pr}{1 - K_{01}^2 (1 - Pr)} \qquad (5)$$

2. The method used for cohesionless soils leads for purely cohesive soils under the equal boundary conditions to the determination of the zone of pressure at rest bordered by the lower resp. upper limit

$$\sigma_3' = \sigma_1' - c'\sqrt{2} \qquad (6)$$

resp.

$$\sigma_3' = \sigma_1' - c'\sqrt{2} \qquad (7)$$

The equ.6 and 7 define the max.possible influence of cohesion on the pressure at rest. We see that for the principal stress σ_1' 0 the principal stress σ_3' reaches to values depending on the cohesion. This statement corresponds with laboratory test results on five soils with different cohesion (Brooker and Ireland 1965).

3. Combining both the results valid for cohesionless and purely cohesive soils we find that the pressure at rest in cohesive soils ($\phi' =$ const. and $c' =$ const.) varies in a zone bordered by the lower resp. upper limit

$$\sigma_3' = K_{01}\left(\sigma_1' - c'\sqrt{2}\right) \qquad (8)$$

resp.

$$\sigma_3' = K_{02}\,\sigma_1' + c'\sqrt{2} \qquad (9)$$

The zones of the pressure at rest for the above dealt three soil types are illustrated in the attached figgure.

4. It has been stated (Pruška 1972a) that values belonging to the zone borders are reachable when the increasing resp.decreasing stress σ_1' starts from zero resp. infinitum in cohesionless soils. When it starts from an other value, the relation σ_3' versus σ_1' isn't a strait-line.
Similar results are valid for cohesive soils. The lower strait-line limit may be reached when the loading starts from the pressure

$$\sigma_1' = \sigma_3' = -\frac{c'\sqrt{2}}{1 - K_{01}} \quad K_{01} = \frac{c'\sqrt{2}}{1 - K_{02}} \qquad (10)$$

Starting from other values e.g. from $\sigma_1' = \sigma_3' = 0$, the measured relation σ_3' versus σ_1' isn't the lower limit and therefore the strait-line as well. It is a convex curve generally and has been measured by Bishop (1965). The lower limit as a strait-line has been measured by Gersevanov (1936) and analysed by Kézdi (1962). The results lead to the equation of type /8/.

REFERENCES:

BISHOP A.W. et al.(1965), "Triaxial Tests on Soil at Elevated Cell Pressures." Proc. 6th ICSMFE,Montreal, Vol.I, p. 170-174.

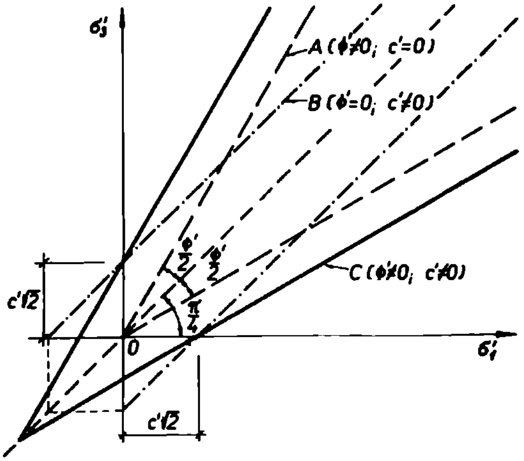
BROOKER E.W., IRELAND H.O. (1965), "Earth Pressure at Rest Related to Stress History." Canad. Geotech.Journal, Vol. II, No 1,p.1-15.

GERSEVANOV N. (1936), "Improved Methods of Consolidation Test and of the Determination of Capillary Pressure in Soils." Proc.,1st ICSMFE, Cambridge, Mass., Vol.I, p.47-50.

KÉZDI A. (1962), "Erddrucktheorien", Springer-Verlag Berlin/Göttingen/Heidelberg, p. 35-36.

PRUŠKA L. (1972a), "Basic Equations of Pressure at Rest of Granular Materials." Proc. 5th Eur.C. SMFE, Madrid, Vol. 1, p.69-76.

PRUŠKA L. (1972b), "Coefficient of Pressure at Rest in a Granular Media at Decreasing Pressures" (in Czech). Research Report ÚTAM-ČSAV, Praha.



BORDERS OF ZONES OF PRESSURE AT REST FOR COHESIONLESS (A), PURELY COHESIVE (B) AND COHESIVE SOILS (C).

LABORATORY AND FIELD DETERMINATION OF SOIL PARAMETERS FOR THE DESIGN OF Mt. BAKER RIDGE TUNNEL. M.A. Sherif , R.J. Strazer (USA)

ABSTRACT

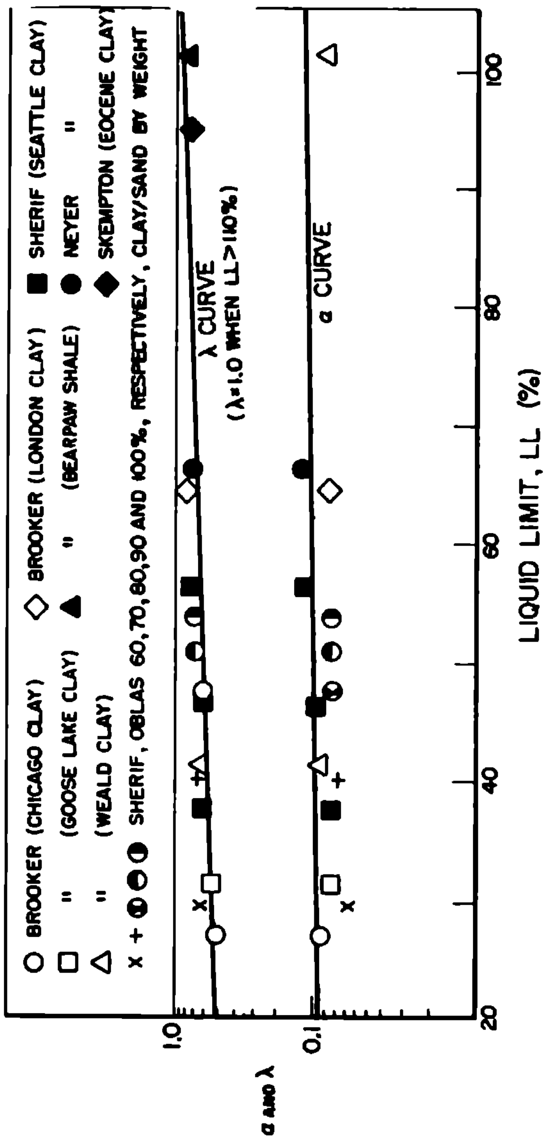
Soil investigations were initiated by the Washington State Highway Department on the subsurface soils encountered along the proposed 61.5 feet diameter and 1500 feet long Mt. Baker Tunnel in Seattle. Field experiments for the determination of the modules of horizontal subgrade reaction K_{hl} and the passive soil failure strength Q_o were conducted at various depths in two 6 feet diameter test shafts. Table I summarizes the K_{hl} data obtained during the tests. The Q_o is the maximum passive soil strength that can be mobilized on the excavated side of the structure below the grade level; the K_{hl} on the other hand is a measure of the elasticity of the soil and it is defined as the stress on a one foot wide or a one foot square plate which will produce one inch of plate deflection. Two block soil samples were obtained from one of the test shafts at approximately the same depth at which the field tests were conducted. These block samples were tested in the laboratory using the University of Washington Stress Meter. (Ref. 1) Table II includes the laboratory and field data which indicate a reasonable correlation between the field and the laboratory values of K_{hl}. The same table also shows a very good correlation between the passive soil failure strength Q_o obtained in the field and in the laboratory.

The design value of the coefficients of lateral stresses K_o was determined on the basis of the following formula proposed by Sherif. (Ref. 2):

$$K_o = \lambda + \alpha(Pr - 1) \qquad (1)$$

where: λ and α are functions of the soil liquid limit as indicated in Figure (1) and Pr is the overconsolidation ratio as determined from conventional consolidation tests

Table III summarizes the K_o values determined on the basis of equation (1).



(after Sherif)

FIG. 1. LIQUID LIMIT VERSUS λ AND α.

TABLE I

SUMMARY OF MODULUS VALUES DETERMINED FROM HORIZONTAL PLATE BEARING TESTS

TEST NUMBER	TEST SHAFT	DEPTH (feet)	ELEVATION (feet)	SOIL TYPE	E (psi)	K_{h1} (pci)
PBT-1	TS-1	95	60.7	Very dense SAND	11,400	1540
PBT-2	TS-1	50	105.7	Hard CLAY	18,900	2570
PBT-3	TS-1	30	125.7	Glacial TILL	33,300	4520
PBT-4	TS-2	119	85.6	Hard CLAY	21,600	2930
PBT-5	TS-2	79	125.6	Hard CLAY	9,170	1240
PBT-6	TS-2	60	144.6	Hard CLAY	7,720	1050

TABLE II

FIELD RESULTS (Test Shaft TS-2)

TEST NUMBER	DEPTH (feet)	Q_o (psi)	K_{h1} (psi)
PBT-4	119	194.3	486
PBT-5	79	235	691

LABORATORY RESULTS (Test Shaft TS-2)

TEST NUMBER	DEPTH (feet)	Q_o (psi)	K_{h1} (pci)
SM-2	108	180	600
SM-1	69.5	200	1400

TABLE III

SUMMARY OF K_o AND CONSOLIDATION TEST DATA

BORING NO.	SAMPLE NO.	LIQUID LIMIT (%)	(1) P_o (tsf)	(2) P_c (tsf)	(3) P_r	(4) K_o
C-101	S-20	64	5.64	19	3.36	0.96
C-102	S-16	59	4.74	9	1.90	0.76
	S-33	66	9.30	19	2.04	0.82
C-103	S-14	75	4.11	9	2.19	0.86
	S-27	51	8.05	15	1.86	0.72
C-104	S-18	41	5.33	21.5	4.04	0.88
C-105	S-14	33	3.60	16	4.45	0.90
TS-2	95'	50	5.7	18.5	3.25	0.85
	95'	49	5.7	25	4.39	0.99
	Average Values			16	3.2	0.86

where: (1) Overburden stress (2) Preconsolidation stress (3) Preconsolidation ratio, P_c/P_o (4) Coefficient of earth pressure at rest

ACKNOWLEDGEMENT

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REFERENCES

SHERIF M.A. and TIEN Y.B. (1968), "Cylinder Pile Design on the Basis of New Soil Test Procedures," University of Washington Soil Engineering Report No. 2.

SHERIF M.A. and WU M.J. (1971), "Summary and Practical Implications of the University of Washington Soil Engineering Research," University of Washington Soil Engineering Report No. 7.

RATIONAL CONSTRUCTIONS OF RETAINING WALLS SUPPORTING SOIL FILLINGS. Z.V.Tsagareli, Z.Z.Tsagareli

The problem of lightened constructive forms of retaining walls has occupied the minds of scientists for many years.The researchers tried to find the one-side decision of this problem,they tried to receive the general formulas,permitting to find the requiring retaining wall's thickness of the profiles which one can meet in practice,proceeding from the stability conditions of construction position and soil stability of foundation.

On the basis of researches it has became possible to solve generally the problem of finding the rational profile of retaining wall /1,2/

Almost all constructive forms of retaining walls being used as for slopes supporting and for banks and embankments strenthening also may be combined into three profiles,shown in Fig.1 (1,2).

The contour of the profile in Fig 1b is the most universal in comparison with three profiles of retaining walls,from which we can receive the contour of the profile,shown in Fig.1c, where the height of the cut part h=0,and the contur of the profile,shown in Fig.1a, with h=H.

Therefore in researching the problem of rational profile of retaining wale the universal profile in Fig 1b was taken as the rational one.

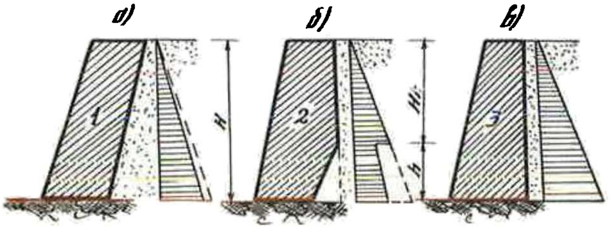

Fig.1.Profiles of retaining walls

For creating the new economical constructive forms it is not sufficiently to define only the contour of the rational profile of retaining wall,it is also necessary to work out within the limits of the noted contour the rational constructions which may sufficient the requirements of economy and modern industrial methods of structures erecting with the minimum loose of space. Moreover, for creating the most economical construction it is necessary to provide such a distribution of material in it,where the effective its application will take place.

Taking into account the rational profile of the retaining wall carried out,the constructive forms of such walls with the gradual approaching to rational profile and almost satisfying the abovementioned requirements have been developed during a number of years

These constructive decisions have been received for the walls partially sectional made of the combined masonry, for the rein-forced concrete,completely sectional structures.

The material of masonry of walls and the armoured concrete and the weight of soil filling have been effectively used in the developed constructions,there have been also applied the most successful methods of limiting of the lateral soil pressure.

Partially sectional constructions,made of combined masonry are presented in Fig.2 (types I,II,III).They are considered as inventions and at different timeswe received the author's certificate for them /3,4,5/.

Sectional rein-forced concrete construction are also presented in Fig.2 (type IVa,b)

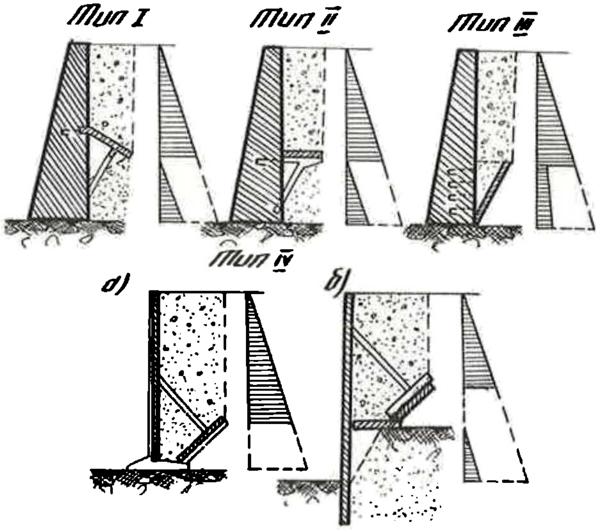

Fig.2. The schemes of proposed constructive forms of retaining walls

The IV type of retaining wall presented in two variants is the lightened constructive retaining wall in the form of thin-walled rein-forced concrete completely sectional constructions,satisfying to the modern requirements in building:

a. Sectional armoured concrete wall with anchor unloading plate on natural base (Fig.2a)

b. Sectional armoured concrete grooved wall with anchor unloading plate for the conditions where the groove concrete driving is possible (Fig.2b).

The brief description of two proposed completely sectional constructions of retaining walls made of armoured concrete and armoured cement are given below.

1. Sectional rein-forced retaining wall with anchor unloading plate (type IV-a)

The proposed constructive form of retaining wall the same as type III,having unloading arrangement in the form of rein-forced concrete boxes,completely corresponds teoretically received rational profile of retaining wall /1,2/.

The most successful separation of the sectional rein-forced concrete retaining wall with anchor unloading plate is presented in Fig.3 (type IVa), where the retaining wall consists of four types-blocks: vertical ribbed plate-1,fundamental block-2,anchor rods-3 and inclined plate -4. Such a separation is effective for the walls with h=8,0m.

The defect of the known retaining walls with the plate sloping to the filling-in is in that these inclined plates should take the considerable bending moments to the account of soil filling weight and therefore their console part cannot be a considerable one.

Moreover in the places where the front wall has been conjugated with the horisontal plate the bending moments of considerable value appeared, requiring of the harge sizes in Section of the front wall.

The constructive defects of these walls are in that they (walls) are unequivalent in stability to shearing and overturning. The construction sizes should be selected taking into account the conditions of wall shearing while the coefficient of stability to overturning being considerable.

To eliminate the abovementioned defects instead of construction of sloping of the end part of low horisontal plate the sloped plates are hanged to the front wall by means of anchor rods. This improves the work of the sloped part of the low plate, which has not already been a contilever, but works as a free laying on two supports therefore its length can be considerable.

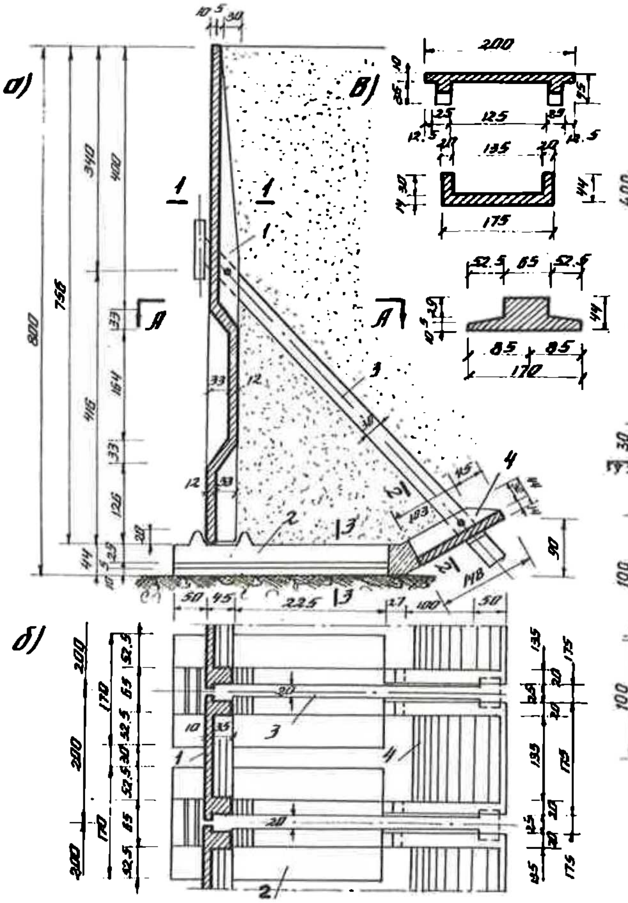

Fig.3. Type IVa. Sectional rein-forced concrete retaining wall with anchor unloading plate: a- cross section; b-A-A-section; c- Sections 1-1, 2-2, 3-3.

At the same time because of the presence of bearing at the place of anchor rod strenthening the webs of the front wall begin working as single-cantilevered beam, the plate being placed in compressed zone, that by all means makes better their static work.

2. Sectional retaining wall with anchor unloading plate made of armocemented folded elements (type IVa).

Sectional retaining wall with anchor unloading plate is a wall, shown in Fig.4, comprising armocemented folded elements; this wall was suggested by Ing. Tsagareli Z.Z.

The walls made of armocemented elements may be recommended for using as bank strentheening on rivers, where the abrasion owing to suspended drifts is impossible, as walls for supporting slopes and as counterorumbled walls.

The wall suggested has the vertical wall I, comprising folded armocemented element with the width 100 sm and the height of fold 30sm; thickness is 5 sm.

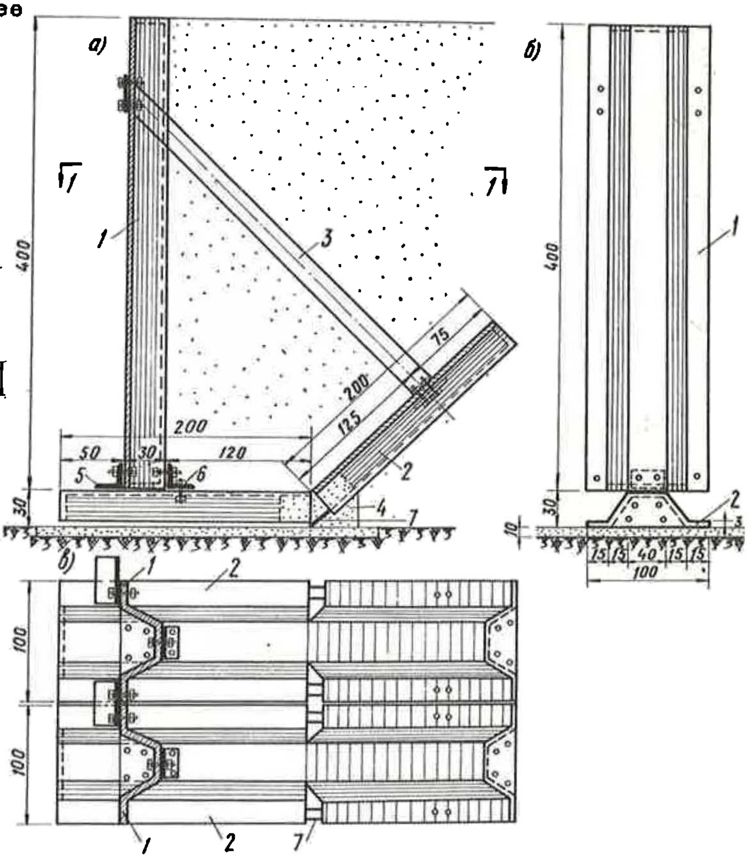

Fig.4 Type IVa. Sectional retaining wall with anchor unloading plate made of armocemented folded elements:
a- cross-section, b-front, c-section 1-1

Foundation plate and sloping plate 2 (fig.4) made of a single armocemented folded elements by sawing up the armocemented folded element, sloping and horisontal parts and bending at an angle of 45° from horizont. Between the bending elements the shortened parts are welded on.

Anchor rod 3 made of sheet steel with the width of 10mm, having the welded quadrangular fashin parts at the ends with the predrilled holes for joining with the elements 1 and 2.

The vertical plate I has been botted to the horizontal part of the element 2.

Element 2 instead of bending becomes monolithic by placing of concrete 4 after setting. Walling made of armocemented folded elements is recommended for producing by following manner: folded elements are manufactured by factory method,having at the ends the diaphragus of one type and size.

All metallic parts: rod 3, angles 5,6 and bolts for assembling are also made by factory method.

Element 2 is manufactured at the place of construction by the abovementioned method; the holes in armocemented elements for bolting of elements 1,2 and 3 are also bored at the place of construction.

After the excavation of the foundation trench being ready its bed is levelled by concrete m-100, the width of concreting is 10 sm.

When the concrete is hardened the folded block 2 is set, the block I is placed above the mentioned block and element I is holded up temporarily then bolted by means of the shortened parts 5,6 and the rod 3.

The following blocks are set in the same manner, the only difference is in that there is no necessity of holding up temporarily the vertical element I. It will be holded by the installed element.

After making monolithic 4, in the case of necessity the draining takes place,and after this the filling is produced.

It must be assumed that the sectional retaining walls with anchor unloading plate made of armocemented folded elements will be used widely in building as the most economical completely industrial constructions,satisfying all the requirements of the modern building.

The data for calculating of folded armocemented elements are given in the article by Z.Z.Tsagareli /6/.

Some of the abovementioned walls having the length of 10 kms., have been inculcated in the Georgian Republic /1,2/ on the Caucasian railway, in the Northern Caucasus, for strenthening of the banks of the Terek river, on the territory of Cavdolomit /according to the design of Giproniinerud (Leningrad)/. "Giprogor" (Moscow) has been designed the embankment of the Kinel river in Buguruslan town. The Caucasian railway in Ahaldabad of the Borzomi branch-line has put into practice two constructions of walls as counter coll apsed /7/.

At the present time SMY-6 in Gori began assembling the completely sectional construction of retaining wall with anchor unloading plate for strenthening the Kura river near Ahaldabad, the length of this wall will be of some km..

REFERENCES:

1. Tsagareli Z.V.- Working out of new lightened constructive forms of retaining walls and their inculcation. Tbilisi, 1964.
2. Tsagareli Z.V.- New lightened constructions of retaining walls, Moscow, Stroiizdat, 1969
3. Tsagareli Z.V- Retaining wall. Author's sertificate N 80601, 1949
4. Tsagareli Z.V.- Retaining wall. Author's sertificate N 88772,1950
5. Tsagareli Z.V.-Retaining wall Author's sertificate N 95617,1953
6. Tsagareli Z.Z.- The definition of theoretical breaking bending moment and height of compressed zone for cross section of folded bending armocemented element. Proceedings of the Academy of Sciences of the Georgian SSR "Concrete and rein-forced concrete", "Metsniereba",Tbilis,1972
7. Roinishwili N.M.,Tsagareli Z.V.- New constructions of countercollapsing walls. "Transport building", N 3, 1967.