Behaviour of Foundations and Structures

Comportement des Fondations et des Structures

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Chairman: B.A. Kantey

May we begin, please? Gentlemen and ladies, we have a very tight schedule this morning, so I would like to begin. We don't mind being interrupted while you take your places.

It has been for me a very great privilege through the past couple of years to receive copies of correspondence between the three gentlemen concerned with the State of Art Report and to watch the interplay of ideas which has led to the production of the State of Art Report covering this session, a document which in my opinion is one of the most significant documents to come out in recent years. It is therefore very much of an honor for me to chair this Session 2 with this distinguished panel which you see before you.

I do not intend to waste your time or the panel's time with any technical comments, as I believe the panel before you will do full justice to the topics under discussion. Sufficient it is to say that it is possibly appropriate that the geographical center of this panel is somewhere in the continent from which I come.

I would like to start off by mentioning the session's Secretary, Professor Kimura, who has been an absolute tower of strength to your General Reporter Dr. Burland and myself since even before our arrival here, and we would like personally to express our thanks to him in front of you.

To your left and to my right, the first member of our panel is Professor Hakuju Yamaguchi, Professor of Soil Mechanics at the Tokyo Institute of Technology. Next to him we have Professor Trofimenkov, Director of the Foundation Design Institute of Moscow. Next to him we have Alexander Vesic, Jones Professor and Dean of the School of Engineering at Duke University in North Carolina, USA. Then we have Jerry Leonards, well known as Professor of Soil Engineering at Purdue University. Going a bit further north in Canada, we have Jeff Meyerhof who is Head of the Department of Civil Engineering of Nova Scotia Technical College in Halifax. On my immediate left is your General Reporter Dr. John Burland, Head of the Geotechnical Division of the Building Research Station. Then we have Dr. Bengt Broms, Professor of Soil and Rock Mechanics, the Royal Institute of Technology, Stockholm, Sweden.

At the far end, we have Victor de Mello. Victor de Mello doesn't need much introduction as a Vice President for South America and Professor of Earth Works Foundation Engineering at the Polytechnic School, University of Sao Paulo, Brazil. He is a Consulting Engineer, and as I have said the outgoing Vice President for South America.

I should like at this stage to draw your attention to the fact that the General Reporter has in the Bulletin specifically selected five topics which he wishes to be discussed at this session. Unfortunately, some of you who have put in for discussions have not read those five topics, and if your name is not called up for discussion during the period allowed for discussion, it will be because we are going to give priority to those who have obeyed the instructions of our General Reporter.

We also propose, because of the wide range of topics to be discussed, not to have the break of 20 minutes but to carry straight through, and for those of you who cannot sit for that length of time, nobody will mind if you quietly get up and walk out and come back at a later stage.

Without further ado, I would like to call on John Burland to present his report.

General Reporter: J.B. Burland

(The General Reporter's presentation is ommitted here because it is essentially the same as the contents of the State-of-the-Art Report in Proceedings Volume II and of the General Report in Proceedings Volume III.)

Chairman Kantey

Thank you, John. I think that what John Burland has said is right up Victor de Mello's alley, and I would like him to start the ball rolling.

Co-Reporter: V.F.B. de Mello

I have been requested to summarize some thoughts on practical design of foundations and structures to take account of deformation, structure-soil interaction, variability of ground conditions, and limits in the knowledge of soil properties. It is obviously a request for very synthetic comments on so vast a subject of momentous relevance to the practice of foundation design. It is surprising and sad to note how over many years there have been no papers presented to this Society directing as to possible routines of practical design steps for the average or simple case.

Yet, in the beginning was Practice, and Practive was with Engineering Execution, and Practice was Engineering. In concept, one must go through a single common routine for all cases, to begin to sort out those that might require more attention. Many a worthy development loses sight of the difference between engineering and engineering science, and new tests and theories are compared with other tests and theories, and not with the functionality towards DESIGN DECISION.

Fig. 1 attempts to summarize schematically the diametrically opposite trends in science, and in engineering. In the former we proceed in investigating on by one the additional parameters that may influence a behavior X, and we are elated at each added proven interference, and shout "Eureka". Meanwhile in engineering we recognize a priori that any behavior X is a function of infinite number of parameters, and therefore, by DECISION we begin in the first approximation by considering only one parameter, then gradually two parameters, and so on. It is a conscious act of decision, within which, however, we must recognize that implicitly we must consider negligible or constant the other parameters, not incorporated. Moreover, I strongly recommend that we recognize the interference of DESIRE, since in any decision we subconsciously want, either to repeat what we have done, or to be more daring and economical, or to try out a new approach, or to assume that a pier is no more than a bigger pile, etc.: that is, we are always fitting mental models to suit ourselves. Finally, let us summarily recognize that there is never any such thing as "true" or "complete" DATA: data are, and will always be, nominal, associated with the eyes and theories of the viewer.

1- for SCIENCE $X = f(a)$
$X = f(a,b)$
$X = f(a,b,c)$

2-in ENGINEERING
$X = f(a,b,c,d \ldots z, etc \ldots)$

3-by DECISION
1st APPROX. $X = f(a \ldots)$
2nd APPROX. $X = f(a,b \ldots)$
3rd APPROX. $X = f(a,b,c \ldots)$
the rest being Consciously neglected
because are negligible / are maintained constant

DESIGN ≡ DECISION DESPITE DOUBTS

DECISION = f(DESIRE, etc)
DOUBTS = f ("DATA", etc)
RECOMMENDATIONS
1- double-check as devil's advocate
2-develop by decreasing dispersions

Fig. 1

In Fig. 2 I am trying to summarize schematically the most common design cycle, relying heavily on "INDEX OBSERVATIONS" (transformable into INDEX TESTS for quantification), on PRESCRIPTIONS for DESIGN, and on "OBSERVATION" of the results that yield experience: obviously there is the intervening of check COMPUTATIONS. It is on purpose that I use inverted commas around OBSERVATION, because I refer principally to the observation of the great silent majority of structures that do not require formal monitoring, because they supply information, not so much on what happens, but on the many undesirable possibilities of behavior that did not occur. Man quickly notes what is undesirable and has always developed experience by an intuitive application of Bayes theorem of probabilities.

It is my contention that in civil and foundation engineering we have been misled by the comprehensible fear of failure, into attempting to adjust our computations to F = 1.00 at "failure". Failure is an extreme event, and computations concerning the statistics of extremes are bound to be fraught with frustration (de Mello 1977). From failures we must learn the physical model to our problem. Meanwhile, from the vast number of operational non-failure cases, at different or varying nominal F values (or other design criteria) we must adjust our quantified statistical universe of averages to establish and prescribe the boundary criteria between acceptance or rejection. The progress in such an endeavour, or in any link within the design cycle of Fig. 2, can be well quantified by applying Bayes theorem.

It is not at all surprising that with "experience" one concludes that a given INDEX TEST or a given CORRELATION or temporary PRESCRIPTION needs to be set aside as definitely unacceptable (Step D, Fig. 2). For instance, it has been concluded that in saprolites of igneous rocks the conventional

"DATA" INCLUDES THE CLOSED - CYCLE OF "EXPERIENCE"

GEOLOGIC CONTEXT INDISPENSABLE

WITHIN THE TOOLS OF GEOMECHANICS THE ENGINEERING CYCLE COMPRISES:

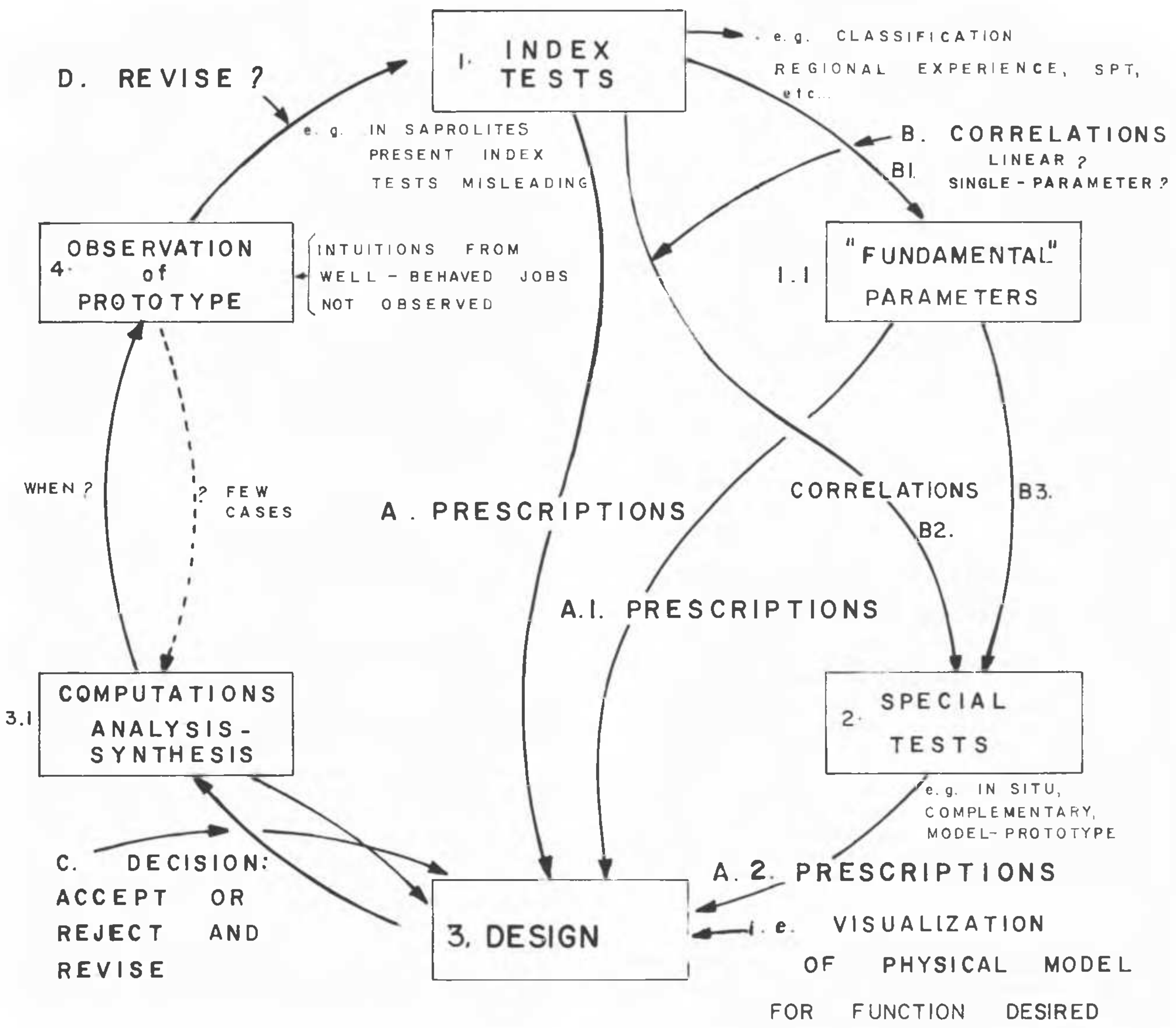


Fig. 2

index tests lead to widely erroneous predictions of behavior (de Mello, 1972).

Similarly, in many a design-prescription type A (such as involved in establishing allowable footing pressures based on SPT), or even of type A2 (such as involved in applying a factor of safety with regard to load test failure pressure or load, in establishing the allowable design values) the inexorable recognition arises that design acceptability in step C cannot be conditioned by factors of safety on failure, but must be proven with regard to limiting settlement acceptances (de Mello 1969). Although most salient cases of failure (catastrophic) are concerned with a physical model of real failure, most revisions of design to within acceptability are imposed on account of settlement and differential settlement acceptance criteria, of relatively indefinite boundaries. Present serious limitation in our knowledge has to do with the many parameters implicit in any given statistical universe of experience transcribed in over-simplified prescriptions or correlations that met early requirements of first-order approximation. Correspondingly the principal "failures" (purposely used in inverted commas to signify a technical K.O., an unacceptable performance) occur when one (a) fails to recognize the statistical dispersion implicit (hopefully to be explicited) in any correlation or prescription, and (b) principally when one transfers satisfactory practices from one region or type of structure to another, without appropriate adjustments.

In the light of such reasoning, it appears worthwhile exemplifying with some of the shamefully unsophisticated routine correlations and prescriptions that were established in Sao Paulo around 1945-55 and are in very wide use, apparently with no overt complaint, except when an entirely different condition, of statistical universe, is at stake. Even an improvement in a sampling, testing, or computing method may introduce temporary trouble until the adjustment coefficients within the closed cycle of EXPERIENCE are reset. But one need not despairingly await for new cases for proving a new procedural cycle, since if we are honest with ourselves, case-histories may be reanalyzed as if under Lambe's (1973) type A prediction. And the only excuse for such a presentation is to draw on other such, from within the files of routine case-histories of design organizations.

Most of the correlations and prescriptions very simply summarized in Fig. 3 are of common knowledge. What is the experience with their use? For instance, Terzaghi and Peck's allowable σ values referred to SPT would be type A prescriptions. A typical A.1. prescription is such as would limit the allowable bearing pressure on footings to the pc value (preconsolidation pressure).

The principal point is to summarize a routine procedure of design decision (preliminary) based on simple prescriptions relying predominantly on highly simplified correlations using SPT values. Shallow foundations are assumed firstly: the implicit correlations are with coefficients of subgrade reaction k_s, t/m^2 per cm of settlement of a 0.8 m diameter plate load test, even though appearing to establish a nominal F value with regard to failure. What are the applicable scale relationships? How significantly do correlations and scale relationships vary with meticulous soil classification? No trouble has been experienced, up to footings of dimensions of about 50 m^2, although hundreds or thousands of buildings have been put up doubtless under such prescriptions crudely applied.

If the presumed settlements are anticipated to be unacceptable, and the designer resorts to piles or piers, the principal prescriptions have been with respect to establishing base or point allowable bearing pressure on the basis of cone penetrometer CPT point resistance qc, assuming no lateral friction on the pier: also, with respect to estimating lengths to which precast concrete piles will penetrate in order to permit (with $F = 1.5$) an allowable load equivalent to that permitted by the allowable concrete compressive stress. The interference of lateral friction may be incorporated in the rule-of-thumb suggestion for piles, but in piers the routine should take its toll because of the absurdity, principally because full friction develops at about 5 to 10 mm of settlement irrespective of diameter of pier and base. But is not the principal variation, presently left to qualitative intuitions, that of so-called EXECUTION EFFECTS?

Finally, with regard to establishing damage criteria, it is my fear that the "start" of tensile cracking is, and will always be, elusive, not only because of great variations of multiple intervening factors, but principally because it is always much more difficult to determine a certain "starting condition" (e.g. of initial stresses, etc.) than to determine the rate change of crack width with change of differential settlement. Tension cracking is obviously much conditioned by the weakest link concept of statistics of extremes. And incidentally hairline cracks are negligible and may be classed as acceptable or even desirable, ... like the advantage of having measles as a child. Thereupon, the principal concern need not be that of predicting or attempting to record the onset of hairline cracking, but the quantification of crack propagation. A useful expedient may be to introduce weakened sections in wall panels to be used as fuse-plugs for early indication for start of monitoring on rates of changes. It is suspected that some existing criteria may suffer significant revision if we extrapolate backwards curves of rates of change of cracks vs. differential settlements.

Dr. Burland has very well summarized these points and our principal deficiencies, and it is my hope that we may draw on the vast cellar of statistical experience from un-

EXAMPLES

1- CORRELATIONS

B.1-a) FOR VERY ROUGH SETTLEMENT ESTIMATE, SEDIMENTS

$C_c \simeq 0{,}009\,(W_L - 10\%) \pm ?\,\%$

and $c/p_c \simeq 0{,}115 + 0{,}00343\,PI \pm ?\,\%$

∴ from $c \approx f(SPT)$ can get p_c and OCR

b-) FOR VERY ROUGH INDICATIONS ON STRENGTH

CLAYS $\varphi = 0°$: $c \approx SPT/8 \pm ?$ kg/cm²

also $SPT \approx 4{,}3 + 3{,}6c + 1{,}8z \pm ?$ Z in m

(São Paulo, cf. Mello 1971)

SANDS $c = 0$; $\varphi = f\,(SPT, \sigma, z) \pm ?$

(cf. Mello 1971)

c-) FOR SETTLEMENTS IN COMPACTED CLAYEY MATERIALS (Mello, in publication)

$C_c \approx 0{,}002\,(W_L + 63\%)$

better $C_c \approx 0{,}21\,(2{,}70 - \gamma_d\,max$ Proctor)

$p_c \approx f$ (Percent Compaction, $\gamma_d\,max$)

Measured settlements $\approx$ (20 to 40%) of computed from block sample oedometer tests

B.2-SÃO PAULO CLAYS $3 \lesssim SPT \lesssim 15$, 0,8m DIAM. PLATE

$k_s \approx 3\,SPT \pm 60\%$ t/m²/cm

CLAYEY SANDS SUBMERGED $3 < SPT < 13$

$k_s \approx -24 + 9{,}2\,SPT \pm 40\%$

CLEAN SANDS $10 < SPT < 40$, $40 < k_s < 70$

also q_c of CPT $\approx (4-6)\,SPT$ kg/cm²

$q_c \approx f(SPT, z)$?

B.3-ANY IN USE ?

2-PRESCRIPTION A

(SÃO PAULO, ROUTINE CONCRETE BUILDINGS, etc)

2.1- 1st STEP FOOTING HYPOTHESIS

OF ECONOMIC INTEREST IF $\sigma_{all} \gtrsim 1{,}6\,\frac{\Sigma Q}{A}$

2.1.1- $\sigma_{all} \approx q_f/F$ ASSUMING $F \gtrsim 3$ MAINTAINS SMALL ρ and $\Delta\rho$

e.g. EMPIRICAL $\frac{SPT}{5}$ or $\sqrt{SPT} - 1 \lesssim \sigma_{all}$ kg/cm² $\lesssim \frac{SPT}{3}$

2.1.2- ESTIMATE SETTLEMENTS

a) from k_s. SCALE relationships ?

b) from oedometer p_c, C_c. ADJUSTMENT factors ?

2.2- 2nd STEP IF SHALLOW FOUNDATION SETTLEMENT UNACCEPTABLY HIGH

a) REVISE STRUCTURE (?)

b) RESORT TO PILES OR PIERS

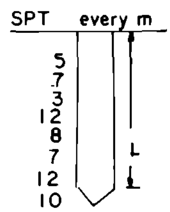
3-PRESCRIPTION A.I.

e.g. PILE POINT $q_L \approx 1/n$ (q_c of CPT)

$n \approx 5-10$?

e.g. PRECAST CONCRETE PILE LENGTH L

L for $\Sigma SPT \approx \sigma$ comp. conc. kg/cm²



EXECUTION EFFECTS ?

Fig. 3

published routines.

REFERENCES

de Mello 1969, VIIth ISSMFE Conf., Mexico

de Mello 1971, IVth PANAM Conf., Puerto Rico

de Mello 1972, IIIrd Southeast Asian Conf., Hong Kong

de Mello 1977, Rankine Lecture, Geotechnique, Sept. 1977.

Chairman Kantey

Thank you, Victor. Jerry Leonards, I think you would like to reply to that.

Panelist: G.A. Leonards

Let me begin by saying straight off that I agree with the General Reporters that the first requisite in the approach to foundation design is a good knowledge of the real soil profile. Unfortunately, the State of the Art Report provides little, if any, guidance on how this good knowledge is to be achieved. My first question is how unsophisticated an approach can one take and still have a "good" knowledge of the soil profile; that is, one that is appropriate for examining what might possibly happen to the structure.

It is however, suggested in the State of the Art Report that the time may have come to interpret whatever we believe to be our knowledge of the soil profile with a statistical approach. As we all know these approaches are becoming more sophisticated and we had a Specialty Session just yesterday to consider some advances in this field. I would like to offer some comments regarding the applicability of this approach (a) in the estimation of consolidation settlements, and (b) in stability analyses.

In 1964 at the invitation of the late Dr. Bjerrum, I had the opportunity to make a study of building settlements in Drammen, Norway. I first attempted to get a "good knowledge of the soil profile" in terms of the preconsolidation pressure, and at one site (Engene, 86) which is well documented in Bjerrum's 7th Rankine Lecture, I arrived at the results shown in Fig. 1. I was nonplussed by the fact that here was a stratum with sharp, random variations in the preconsolidation pressure, and I refused to accept the fact that this was due to differences in sampling disturbance because I had personally participated in taking the samples, transporting them, storing them, extruding them, placing them in the oedometer, and then applying the loads.

Fig. 2 is an x-radiograph of a clay sample from Drammen prior to extrusion from the sampling tube, which was taken by O. Sopp (1964) at NGI. You can see from the shadings (the lighter areas represent lower densities) that there are substantial differences in the soil profile on a scale of a few millimeters, which accounts for the erratic distribution of the preconsolidation pressure, p_c. Referring back to Fig. 1, it is clear that using the average and dispersion of p_c in a layer several meters thick is not appropriate because each value of p_c is associated with a different value of the overburden pressure (p_o) and the net increase in pressure (Δp). In principle, a statistical analysis is possible but the scale of layer thicknesses must often be far thinner than is customary in a conventional statistical approach.

Fig. 3 is a composite of the logs of several vane borings in a deposit of soft clay, which have been plotted to the same depth scale. I will pause a moment to allow you to assimilate the variations in measured shear strength. The data were used to analyse the slope of a cutting for the Kimola canal in central Finland (Kankare, 1969). Initially, only total stress analyses were made but after failures occurred when the calculated F.S. for undrained analysis was 1.5, effective stress analyses were also made (Fig. 4). While the effective stress analysis gives F.S. = 1 (using pore pressures measured one day before the slide) the extent of the actual failure surface was not even approximated. Had the slide occurred along the critical effective stress circle it would have been of no consequence, as over a dozen such small slides occurred and were easily tolerated. The actual slide took place along an inclined weak seam about 7.5 - 8.5 m below the original ground surface; it took out the main road and blocked the canal. Referring back to Fig. 3 you will note that at the 7.5 - 8.5 m depth there are low strength values--and even these values are most likely much higher than those extant in the thin weak seam that controlled the slide. Given the data in Fig. 3, I wonder how many more vane borings the statisticians would have recommended in order to assess the strength variations for a statistically based stability analysis? Unless it is appreciated that we must look at the dispersion in the zone where sliding may potentially occur--which often is a thin weak layer, or a weakness plane due to fissuring

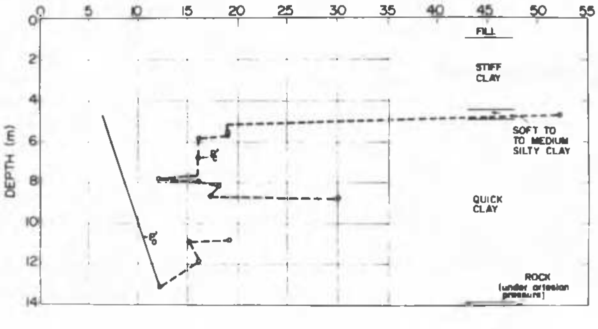


Fig. 1 Preconsolidation pressure vs. depth at engene 86 in Drammen Norway (tests by G.A. Leonards and I. Foss)

or previous sliding--a statistical approach may be more misleading than helpful.

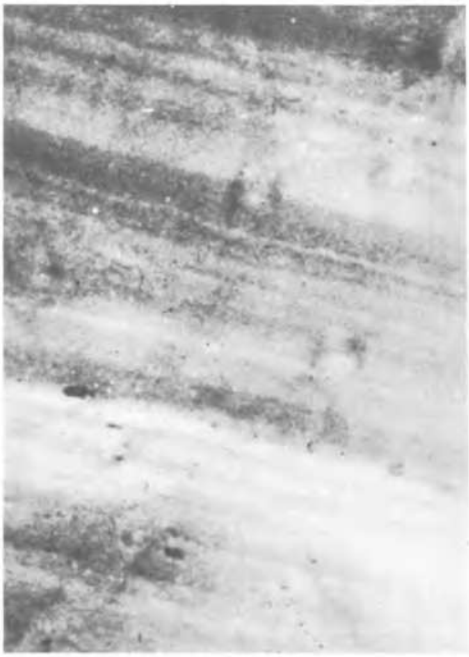


Fig. 2 X-radiograph of a clay sample from engene 86 in Drammen, Norway (taken by O. Sopp, 1964)

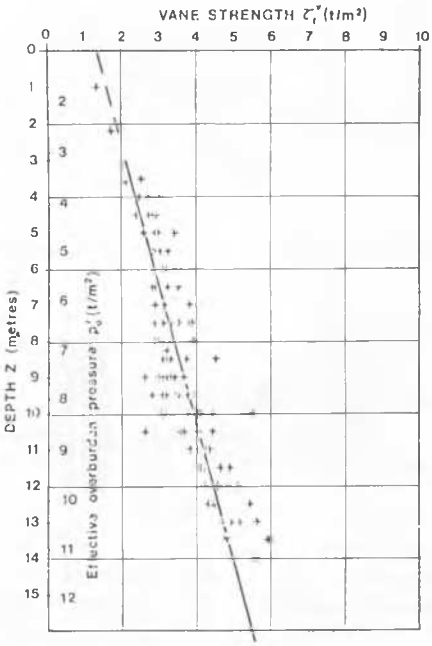


Fig. 3 Composite logs of vane borings at sta 52 + 70, Kimola canal, Finland (after Kankare, 1969)

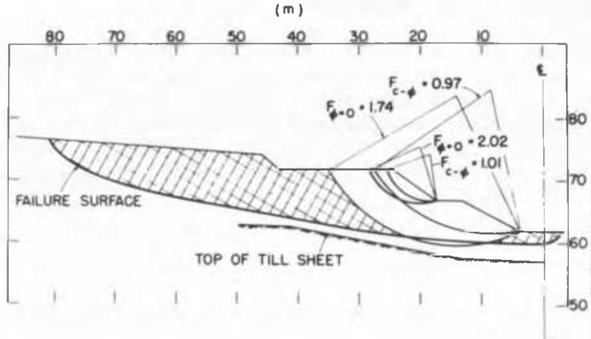


Fig. 4 Cross-section of November 3, 1965 failure at station 52 + 70 on the lower canal

Chairman Kantey

Thank you, Jerry. John, do you have something to say?

General Reporter Burland

I have two comments to make. Firstly, when we stress the prime importance of a knowledge of the soil profile we are not referring to the mechanical properties, the determination of which ranks third in our list (see Section 1.2 of the SOA Report). By a knowledge of the soil profile we mean an understanding of the local geology, ground water conditions and a detailed and systematic visual and tactile description of the soil in each stratum. It is on this information that the majority of foundation decisions are taken.

Secondly, the question of statistics. Of course, the blind use of statistics is very dangerous. A similar example to the one quoted by Professor Leonards is the use of mean laboratory undrained strengths for stiff fissured clays. Such an approach neglects the dominant influence of fissuring and fabric and can lead to an overestimate of the strength in the mass by a factor of two or more. At all times one must understand the physics of the problem.

Chairman Kantey

Victor, I see you looking anxious, 30 seconds.

Co-Reporter de Mello

Well, I agree entirely with Dr. Burland. The basic problem of course is that statistics is nothing but a tool to help us quantify what we think in terms of qualitative experience. We have to use the appropriate models in using it. Otherwise, we would just be using statistics inappropriately.

Chairman Kantey

Right.

Panelist: G.G. Meyerhof

I would just like to interject here that many of the failures we have been looking at in the last few years are due to human error--bad judgment and inexperience--and they have very little to do with the factor of safety, so we should not overemphasize the statistical approach too much.

Chairman Kantey

Well, now I think we must come a little bit further east, and we would like to ask Dr. Trofimenkov to address us for a few moments.

Panelist: J. Trofimenkov

Mr. Chairman, ladies and gentlemen, I would like to make some comments on topic 3 of our session, that is behavior of pile groups and their optimum design. Little is known on the behavior of pile groups because full scale load tests of pile groups are very expensive. Observations on settlements of real structures on pile foundations can widen our knowledge. That is why I think that our experimental data on settlements of some pile foundations will be of interest.

The investigation was carried out into the foundation behavior of a 5-storey panel apartment building with transverse bearing walls. The subsurface profile consists of moraine silty clay of stiff consistency, (liquidity index 20 to 40). Pile foundation consists of a row of 7 driven piles under each transverse wall. Pile length is 4.5 m, cross-section 30 by 30 cm. Center spacing of piles was about 6-pile width. On every pile in a row, load cells were installed, and distribution of load on piles was measured during and after construction. At the same time, settlements of piles were measured, too.

As it is seen on Fig. 1, distribution of loads on piles just after construction was very uneven, from 12 to 30 t. After 2 years, as a result of rigidity of structure, and inter-reaction of structure and foundation, loads on piles were smoothed out, and got almost equal, about 20 t on a pile.

On Fig. 2, it is seen that the mean settlement just after construction was about 5 mm, after 2 years about 8 mm, but more even. In load tests of single pile, settlement under the load of 20 t was less than 2 mm. It follows that settlement of the pile foundation under the measured load in this case was about 4 times that of a single pile tested in a conventional way during some days.

On Fig. 3, settlements of pile foundations on cast-in-place bored piles, (diameter 1 m) on the left, and driven piles cross-section 35 by 35 cm, in the same soil conditions are shown. The subsurface profile consists of silty clay of firm consistency in the upper part of the profile, and of stiff consistency at the pile base. The width of pile groups was 3 m, length of piles 12 m. The settlement of the pile foundations after 3 years was for bored piles 4 times, and for driven piles 1.4 times the average of test loading of single piles.

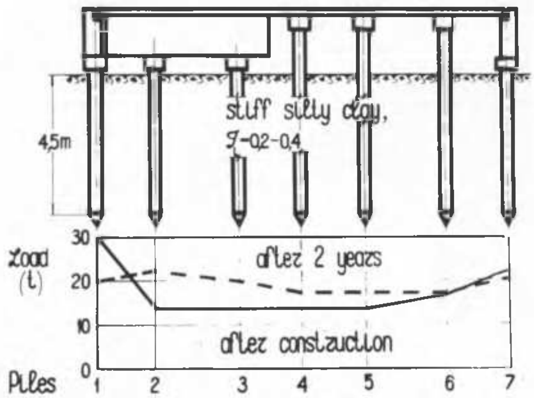


Fig. 1

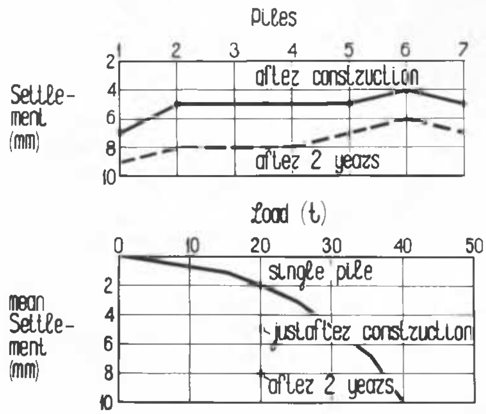


Fig. 2

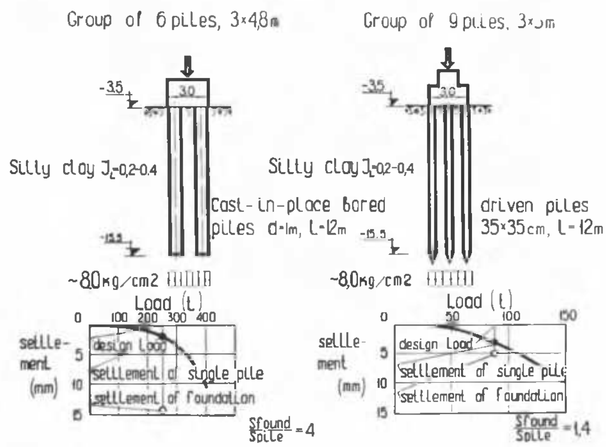


Fig. 3

On Fig. 4, a raft foundation, 540 by 22 m, with 6500 piles. Pile length is 12 to 16 m, cross-section 35 by 35 cm. Center spacing of piles is 1.2 by 1.1 m. Piles are driven in stiff clay. Design load on a pile is 85 t.

The settlement of the pile foundation after 4 years ranged between 25 and 38 mm. In load tests of 50 single piles, settlements, the load of 85 t, were 3 to 5 mm. Thus, the settlement of the pile foundation was about 8 to 10 times that of a single pile.

On Fig. 5, a raft foundation 42 by 36 m with 2000 piles is shown. Pile length is 5 to 6 m, cross-section 30 by 30 cm. Center spacing of piles in the raft was 1 by 1 m. Piles are driven in fine sand of medium density. Design load on the pile is 50 tons. The settlement of the pile foundation after 4 years was 32 mm. In load tests of single piles, average settlement under the load of 50 tons was 3.5 mm. There the settlement of the pile fcundation was about 10 times that of a single pile.

These cases and several others have shown, first, that settlement of structures on

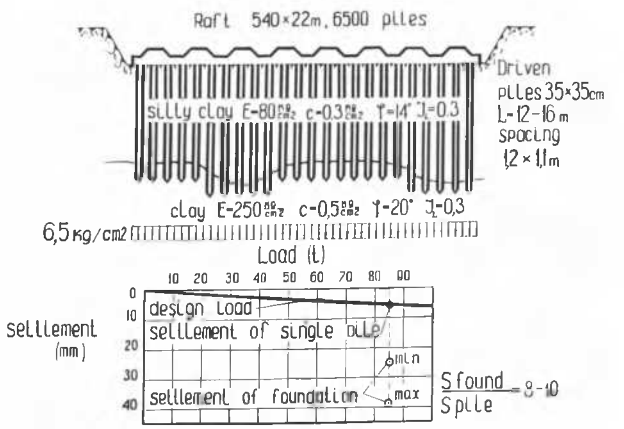


Fig. 4

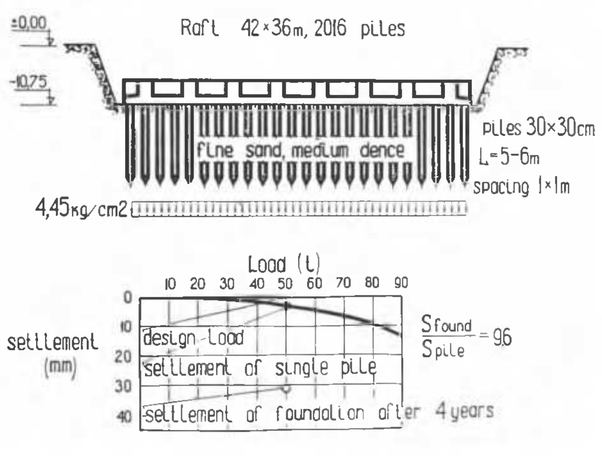


Fig. 5

piled-raft foundation on stiff clays, and sands of medium density is rarely significant and doesn't exceed one-fourth that of a spread foundation.

Second, in these cases of stiff clays and medium dense sand, settlement prediction for pile groups on the result of loading tests of individual piles may be made sufficiently accurately by formula proposed by Skempton in 1953 for pile groups in granular soils. Thank you.

Chairman Kantey

Thank you, Dr. Trofimenkov. Dr. Broms, can we now have your contribution?

Co-Reporter: B.B. Broms

I would like to make two comments. My first comment is concerned with the function of piles as settlement reducers. When the designer of a structure finds out that the settlements will be excessive if a raft or spread footings are used then the attitude of the designer generally changes. The settlement calculations of a structure founded on spread footings, are normally based on results from extensive investigations in the field and in the laboratory of the thickness and the lateral extent of the different compressible layers and of their compressibility.

When the calculated maximum and differential settlements are compared with those that the structure can tolerate the designer may decide that the structure had to be supported on piles. At that particular moment the attitude of the designer with respect to the soil generally changes. He often disregard completely the ability of the soil to carry even part of the applied load. Point bearing piles are normally designed as pin-ended struts which only can resist axial loads without considering the soil between the piles. The design is often restricted to a selection of an allowable load on the piles.

In many cases it should, however, be possible to utilize also the soil between the piles to carry at least part of the load, particularly for floating pile groups where the bearing capacity of the soil within and around the pile group (soil failure) governs the ultimate bearing capacity rather than the strength of the pile material (pile failure). It should be possible to utilize the soil between the piles when the failure mode of the pile group is ductile as is generally the case for a floating pile group and when the total ultimate bearing capacity is equal to the sum of the bearing capacity of the piles and the ultimate bearing capacity of the soil within the pile group. The axial deformation required to mobilize the ultimate bearing capacity of the soil within the pile group is generally large. Normally the ultimate bearing capacity or the total settlements of the pile group do not govern the design. Usually the differential settlements and the average

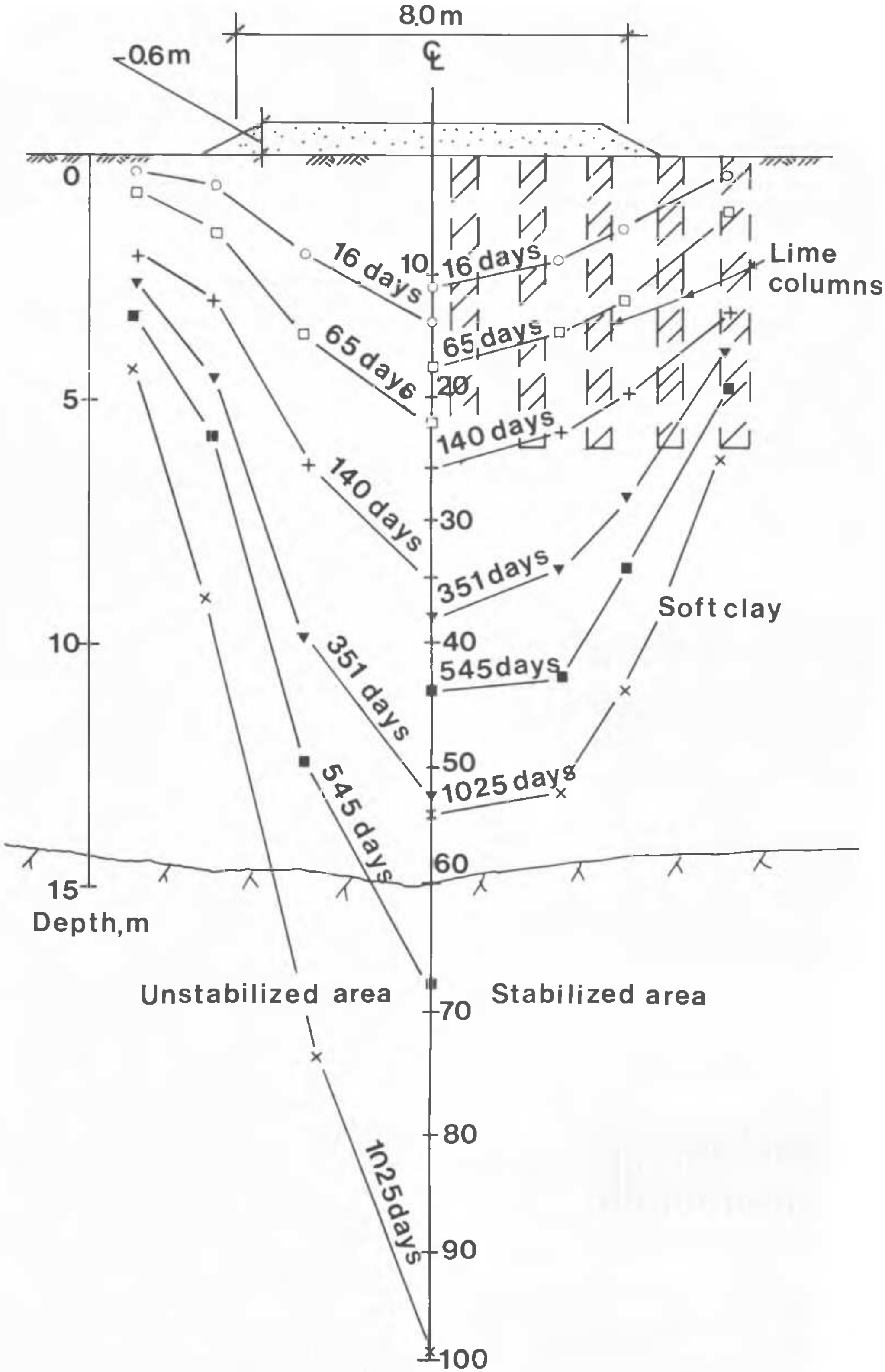


Fig. 1 Results from full-scale load tests at Skå-Edeby, Sweden

shear stress along the perimeter of the pile group are the critical factors.

When the soil between the piles is utilized to carry part of the load it is necessary that the ultimate strength of the pile cap and of the members transferring the load to the pile group is larger than the ultimate strength of the pile group. If the pile group is overloaded the load will be redistributed only when the behaviour of the pile groups is ductile. In the case the behaviour is brittle as is the case for point bearing piles or the ultimate strength of the pile cap or of the members transferring the load the piles is lower than the ultimate strength then the ductility of the pile-structure system might not be sufficient to cause a redistribution of the load from the supported structure. In some cases it may be advantageous to change from point bearing piles to floating piles, particularly if the pile group is affected by negative skin friction and the length of the piles is large.

My second comment is concerned with the differential settlements of floating pile groups. Piles are very effective to reduce the differential settlements of a structure. I would like to illustrate this point with some results from Skå-Edeby located about 20 km outside of Stockholm in Sweden where lime columns have been tested. Two full-scale load tests were carried out at this test field. In one area lime columns were installed while in the second area, which served as a reference area, there were no columns.

It can be seen from Fig. 1 how the settlements of the two areas increased with time. After about three years the maximum settlement below the center of the area stabilized with lime columns was approximately 50 mm. The corresponding maximum settlement of the reference area where there were no columns was approximately 100 mm. The reduction of the maximum settlement by the lime columns was rather small. It should, however, be noted that the degree of consolidation for the reference area was about 30%. The final total settlement is estimated about 35 cm. The degree of consolidation of the area with lime columns is almost 100%. The lime columns had, however, a large effect on the differential settlements. The maximum differential settlement for the stabilized area after approximately one year was 1:850. The corresponding maximum differential settlement of the reference area was 1:130. After three years the maximum differential settlement of the area with lime columns was still very small while the maximum differential settlement of the reference area had increased to about 1:80. This illustrates the large reduction of the differential settlement by the piles within a pile group and that it should be possible to use only as many piles that is needed to reduce the differential settlement to an allowable value. The number of piles can be relatively small.

Fig. 2 illustrates how the differential settlements can be calculated. The angle change (the shear distortion) between two adjacent

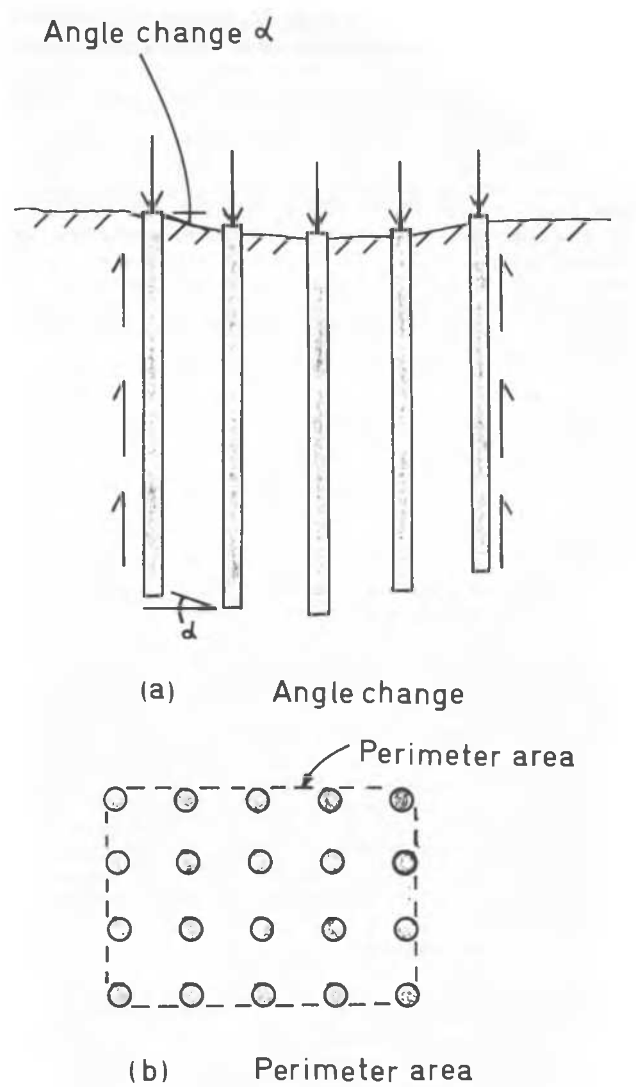


Fig. 2 Calculation of differential settlements

pile rows and thus the differential settlement will be proportional to the average shear stress around the perimeter of the pile group (s_a) and the shear modulus of the soil (G) according to the relationship

$$\alpha = \frac{s_a}{G}$$

By limiting the average shear stress along the perimeter it is possible also to limit the shear distortions of the soil and the differential settlement.

Preliminary calculations, based on the finite element method, indicate that the pile group at low load levels carries the applied load mainly through skinfriction along the perimeter of the pile group and that only a small portion is transferred to the surrounding soil through the bottom of the column group. The load transferred through the bottom will increase with time while the load transferred along the perimeter of the pile group to the surrounding soil will decrease with time.

This change of the stress distribution is compensated by a reduction of the shear modulus of the soil (G). The result is a very small increase of the differential settlements of pile group with time.

Test data indicate that piles are very effective in reducing the differential settlements and that relatively few piles may be needed to reduce the differential settlements to an acceptable level. It is thus possible in the design of a pile group that also the soil between the piles can be utilized since often the maximum differential settlement is the governing factor.

Chairman Kantey

Alex.

Panelist: A.S.J. Vesic

On Significance of Residual Loads for Load Response of Piles

My comments are related to evaluation of settlements of single piles and pile groups. The General Reporter has covered this very complex subject rather extensively, bringing in a good number of significant contributions to the subject in the last ten years. There is, however, one aspect of this problem that was not mentioned, in spite of its great importance for pile and pile group response, as well as overall performance of pile supported structures. I am referring to the phenomenon of residual load and its effect on load settlement relationship.

To our knowledge, this phenomenon was first evidenced quantitatively in field tests by the U.S. Corps of Engineers at the Arkansas River project [1] where residual loads of up to 45 metric tons were recorded in 16-meter long steel pipe and H-piles (Fig. 1). In another such case (Fig. 2), reported by Kérisel and Adam [2] involving tubular steel piles, 43 by 58 centimeters, jacked 5 meters into stiff clay by a force of 88 metric tons, a residual point load of 22 tons, almost equal to the original point load, was recorded. A third documented case (Fig. 3), reported by O'Neill and Reese [3] involving 75 centimeter diameter, bored piles in stiff clay, a single loading to about 130 metric tons, of which 45 tons were transferred to pile point, produced on unloading a residual load at the point of about 20 tons.

The presence of residual loads results generally in an apparent concentration of skin resistance in the upper portion of the shaft, which may cause a substantial reduction in pile settlements. Fig. 4 shows some old measurements at the Atchafalaya River [4]. This was one of the first documented records of distribution of skin resistance on piles. Since no initial residual load was recorded, odd-shaped curves of distribution were observed particularly for single piles (Fig. 4b). Fig. 4a indicating skin resistance distribution for one of the piles in the group shows that driving subsequent piles changes the distribution of residual loads in the system. The net effect of residual loads is to alter the distribution of skin resistance, causing considerable reduction in the pile settlements.

In my contribution to the Third Carillo Lecture in Mexico last fall I have given two examples of recent projects in which the presence of residual loads had an overwhelming effect on pile foundation performance. One of these was the case of foundations for the main building of the Hirchhorn Museum and Sulpture Garden in Washington, D.C. on which I became involved as consultant for Schnabel Associates, Consulting Engineers in Washington, D.C. For architectural reasons, this entire building, about 70 meters in diameter, and four stories high, rests on only four columns spaced about 45 meters apart carrying about 7,700 metric tons each (Fig. 5). These columns were supported by groups of sixty-four 27-meter long steel H-piles, driven through silty clay and medium dense upper sand to an extremely dense stratum of glacial sand with gravel and boulders. Assuming the existence of a residual load, we predicted settlements of only one-sixth of what would have been obtained by any conventional method of settlement predictions, such as those described in the General Report. Load tests fully confirmed this assumption, and the performance of the building since its completion several years ago has been excellent.

A second example from current practice involves the piles for the Brent Field Structure B-1 in the North Sea, which is a 75 by 75 meter platform over 138 meters of water (Fig. 6). This structure is supported on eight legs, each resting on a group of four 180 centimeter outside diameter steel pipe piles, driven through 30 meters of interbedded over consolidated sands and clays into a deep stratum of homogenous, very dense sand. An analysis performed by R. Kirby of Woodward Clyde Consultants under varied assumptions of overconsolidation ratio of the clay present in the profile, showed under the working load of 2700 metric tons in compression predicted settlements of single piles of the order of 2.5 centimeters without consideration of residual loads and slightly over 1 centimeter with inclusion of residual loads. I have since made a revised analysis showing that actual expected settlements should be even smaller. The actual load displacement performance of these piles is still unknown and may never be accurately known as no load testing was performed, and it appears unlikely that actual settlements will be measured under storm conditions. The structure will hopefully remain as one of the many that proved its adequacy of design by just good field performance. However, the examples shown indicate clearly the importance of assessment of residual loads in prediction of pile and pile group settlements. They also should warn us about the doubtful value of numerous theories of pile settlement behavior published in the literature in recent years, which do not con-

(1) Measured compression load distribution assuming no stress in pile at start of test.
(2) Measured compression load distribution after compression test assuming no stress in pile at start of test.
(3) Measured tension load distribution assuming no stress in pile at start of test.
(4) Measured tension load distribution after tension test assuming no stress in pile at start of test.
(5) Tension load distribution adjusted by subtracting Curve 4 from Curve 3.
(6) Compression load distribution adjusted by adding Curve 4 to and subtracting Curve 2 from Curve 1.

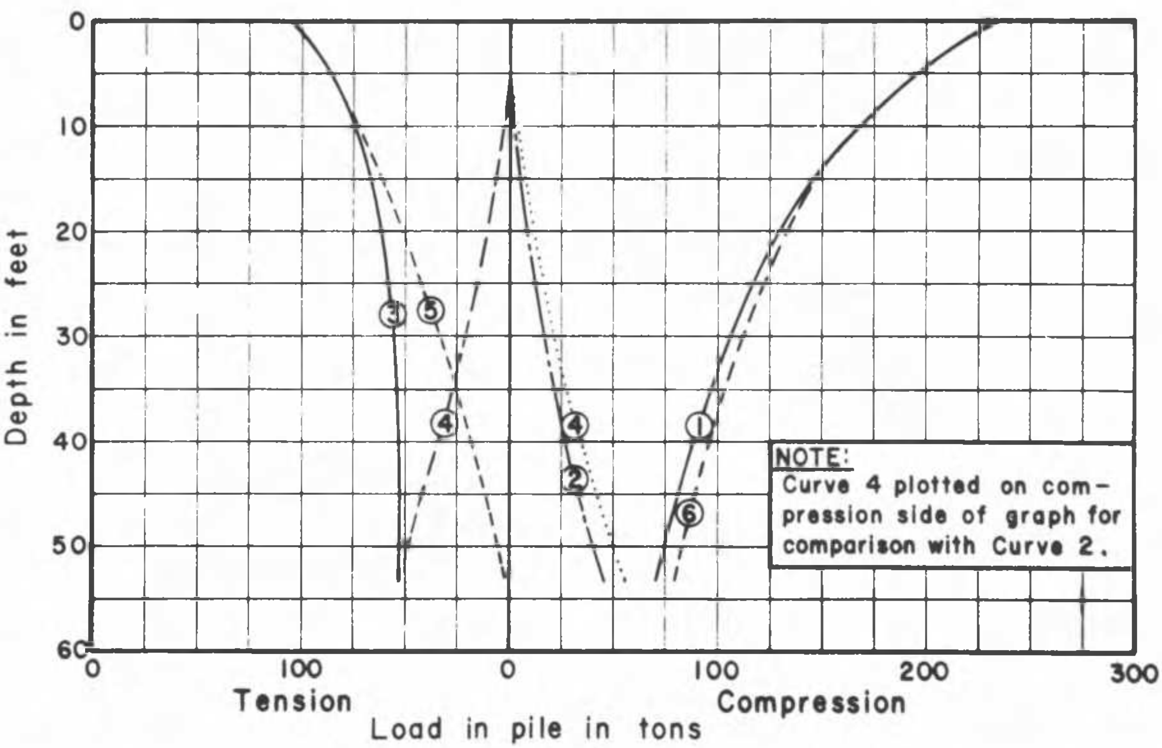


Fig. 1 Effect of residual loads on load distribution in driven piles in sand at Arkansas River (from Mansur and Hunter, 1970)

sider at all this phenomenon. A simple tentative alternative to these theories can be found partly presented in my lectures on pile foundation design at the 1975 Boston Society of Civil Engineers/MIT Seminar Series [6,7].

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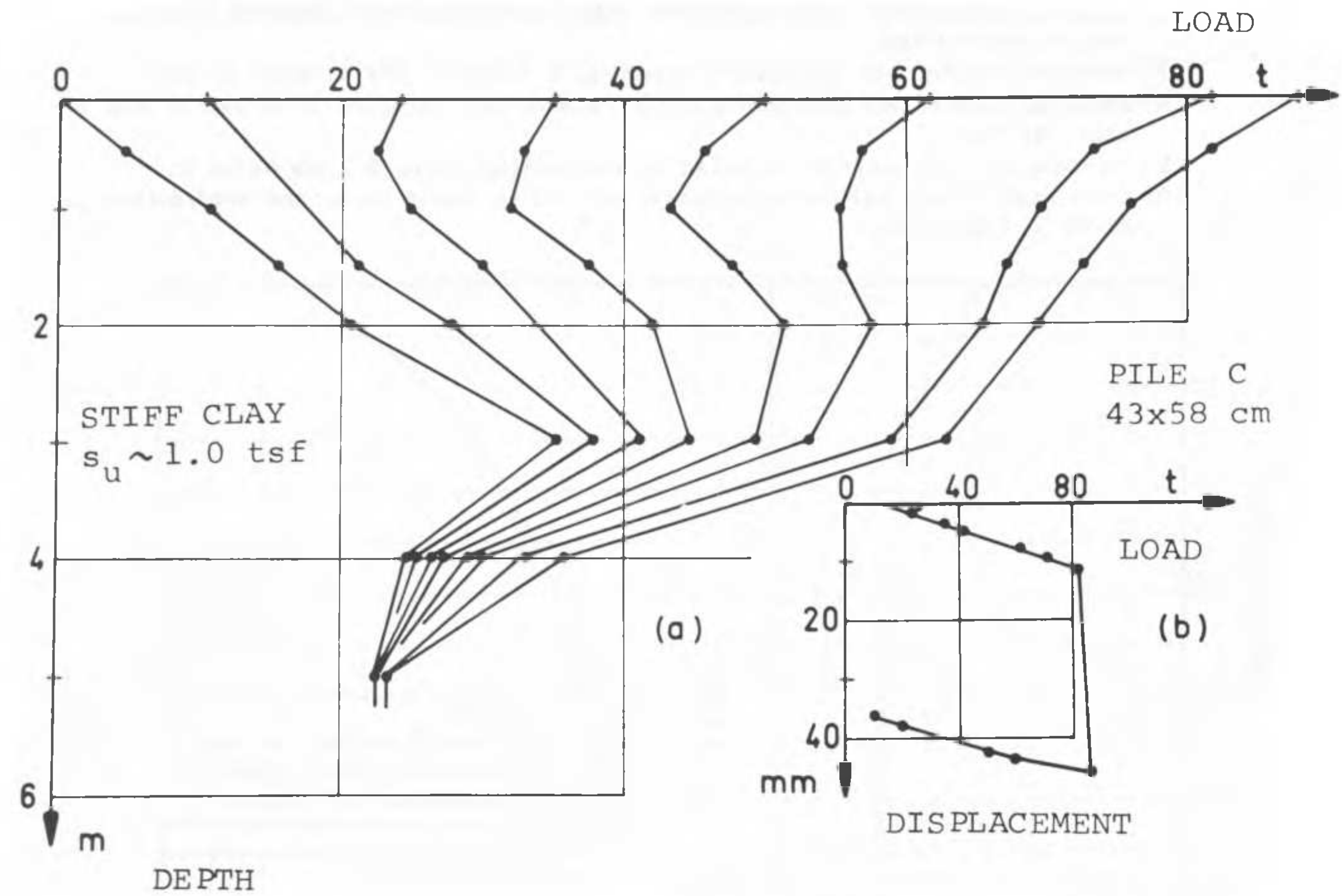

Fig. 2 Load transfer from tubular steel piles in stiff clay (after Kérisel and Adam, 1969)

Program, Transportation Research Board, National Research Council, Washington, D.C., 68 pp.

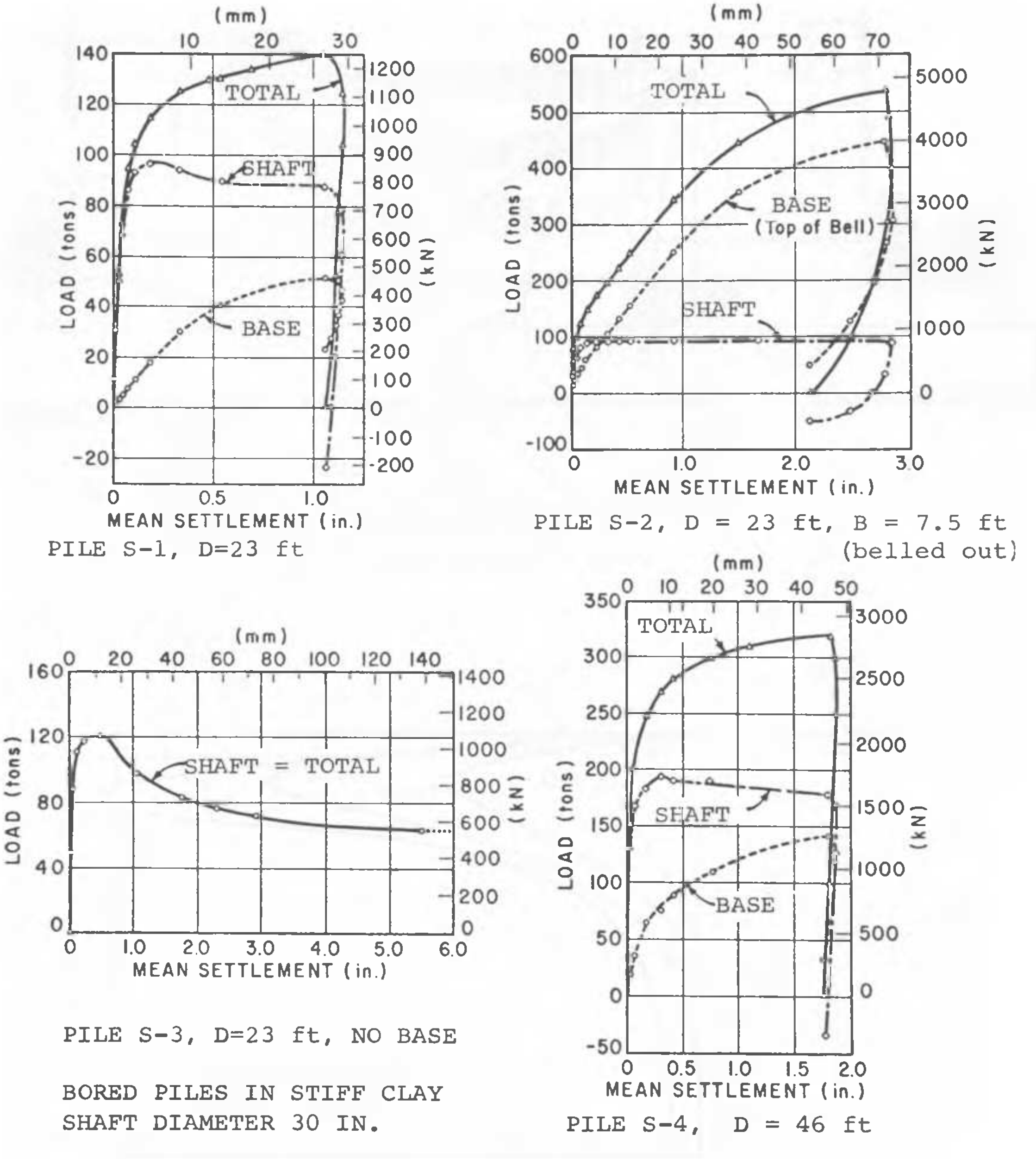

Fig. 3 Mobilization of base and shaft resistance as a function of pile displacement (after O'Neill & Reese, 1972)

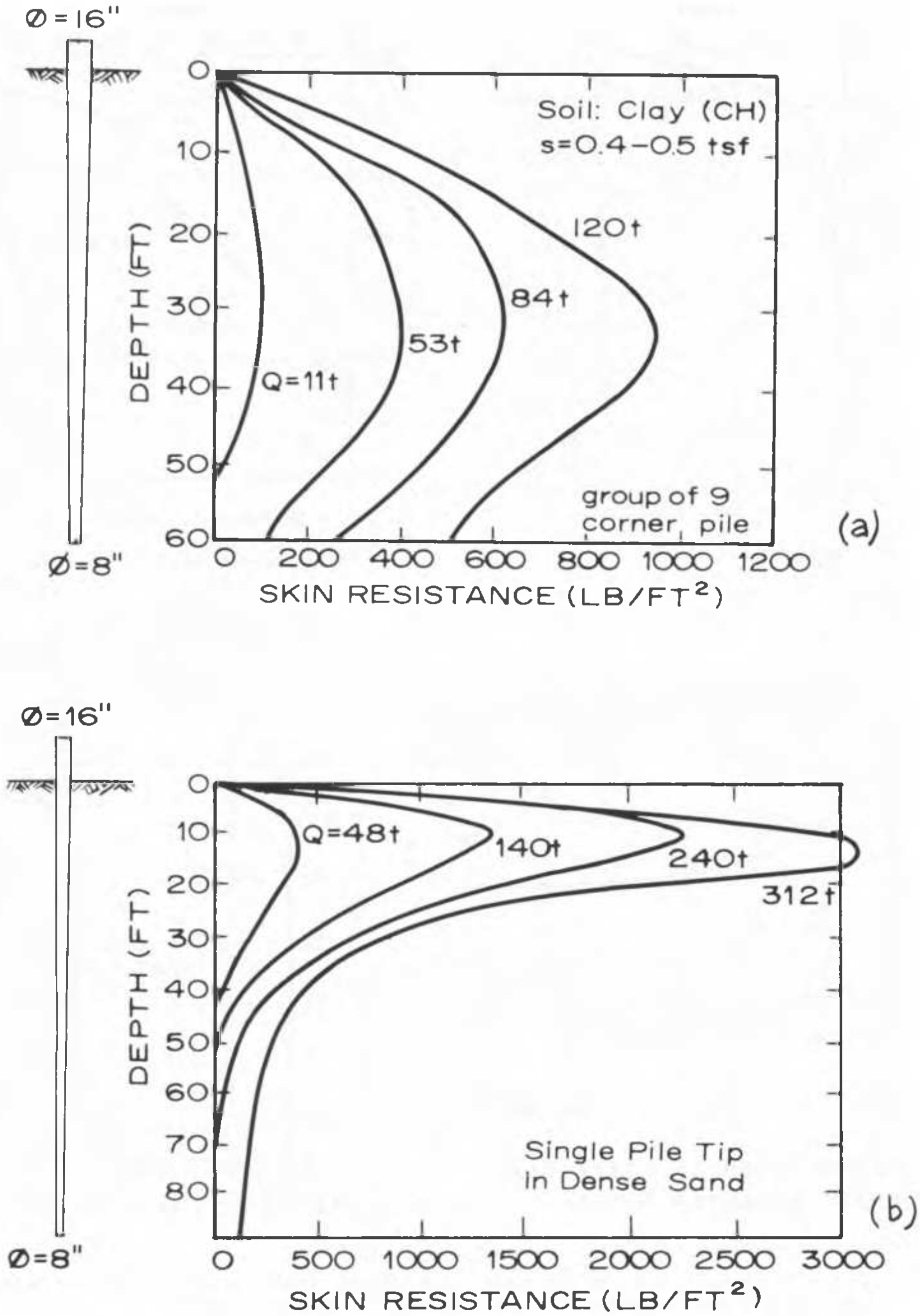

Fig. 4 Measured distributions of skin resistance in clay (after American Railway Engineering Association, 1951)

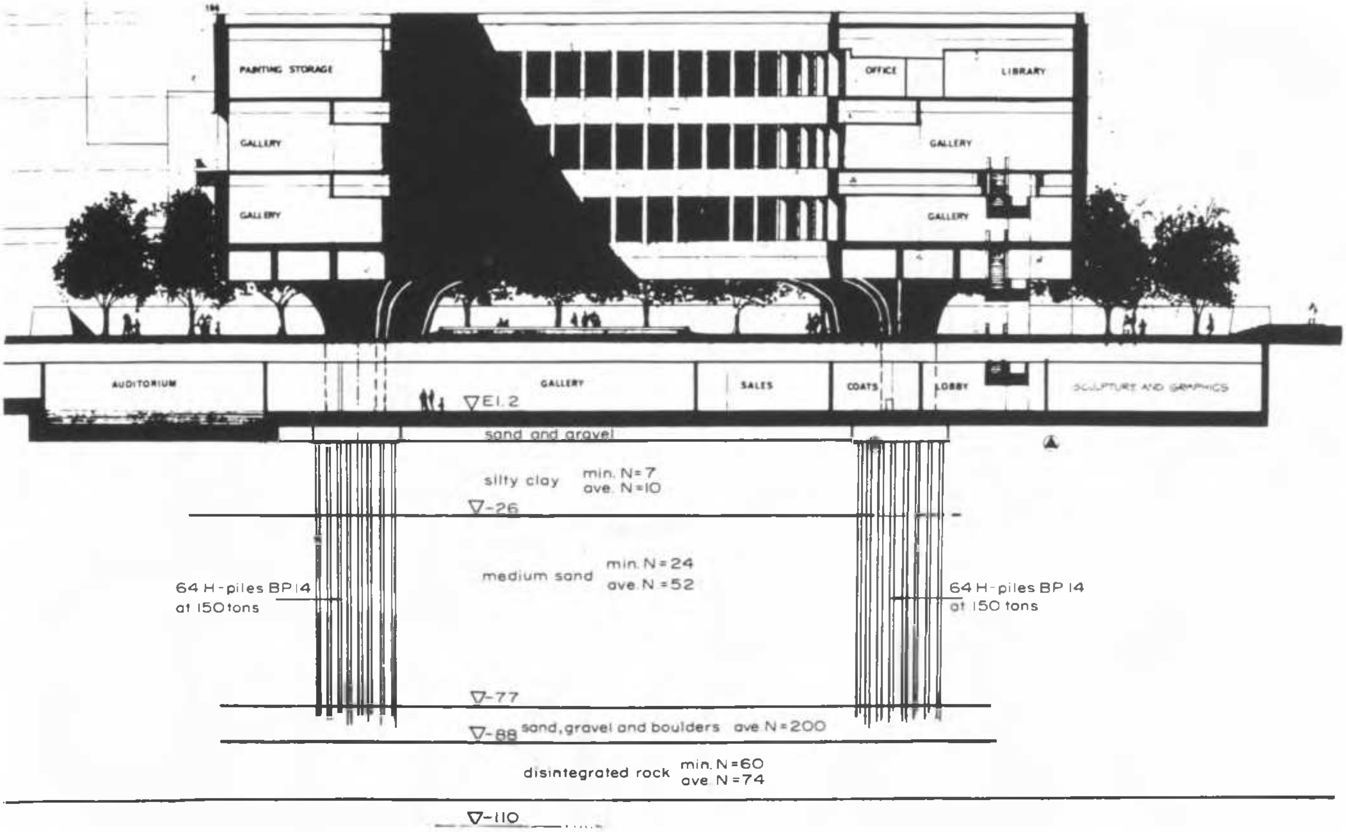

Fig. 5 Foundations of Hirshhorn Museum, Washington, D.C.

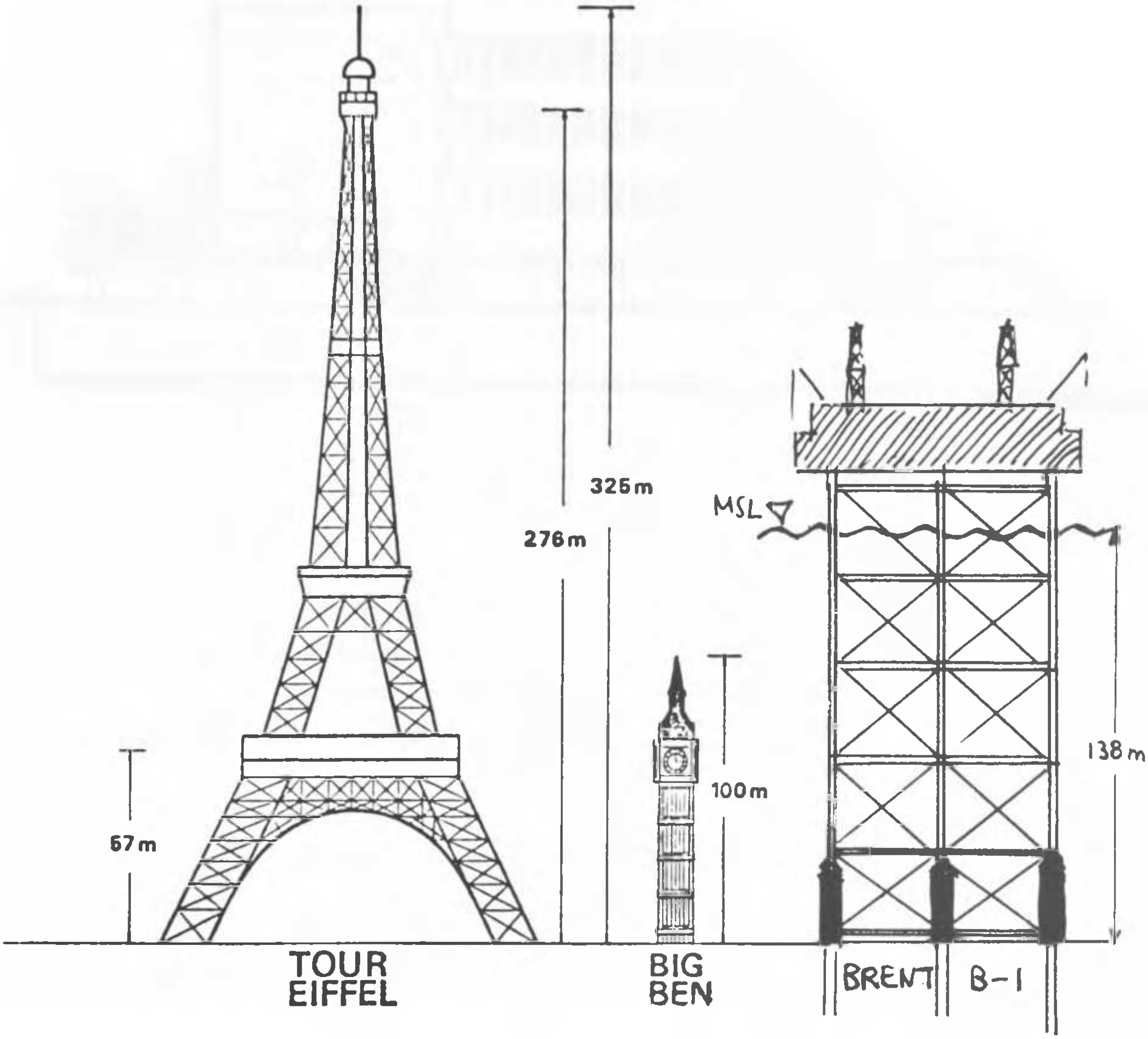

Fig. 6 Brent field structure B-1 in comparison with Eiffel Tower and the Big Ben

Chairman Kantey

Thank you, Alex. At this stage I think we will have a one-minute stretch. We are running a little bit late, and I would also like to warn certain people who put in discussions from the floor about the order in which we are going to call them up in approximately half an hour. First will be Dr. Tatsuro Okamura, followed by Professor Veder, followed by Dr. Preiss, followed by Dr. Fellenius and Mr. Thorburn.
Anybody want to stretch?

(Intermission)

May we continue, please? I would now like to call on Jeff Meyerhof. May we proceed, please.

Panelist Meyerhof

On Allowable Deformation of Foundation and Structures and Criteria for Acceptable and Unacceptable Damage

The General Reporter has shown in his interesting Report that allowable movements of foundations and structures depend on soil-structure interaction, desired serviceability, harmful cracking and distortion restricting the safety or use of the particular structure. Empirical damage criteria are generally related to relative rotation or angular distortion, deflection ratio or tilt of the structure. These criteria differ for frame buildings (bare or cladded), load-bearing walls (sagging or hogging) and other structures depending on the relative settlement ratios after the end of the construction. The General Reporter deals only with foundation movements of buildings for which much information had previously been published.

While the allowable movements of structures can only be determined in each particular case, this is especially true for bridges, which are usually designed to include the effects of anticipated foundation movements. For common types of buildings, however, some early conservative suggestions by the writer (Meyerhof, 1953) are confirmed by the Reporter's comprehensive survey. Similarly, for some other types of engineering structures tentative safe limits may be suggested as a guide. Accordingly, the writer has recently reviewed published data on the failure of earth retaining structures and steel storage tanks. It is found that retaining walls and sheet pile walls may fail if the relative rotation exceeds about 1% or the maximum differential movement exceeds about 1 in. Similarly, for steel storage tanks the limiting relative rotation is found to be about 0.7% and the maximum differential settlement about 2 in. along the perimeter of the tank. Using a minimum safety factor of about 1.5 to cover inevitable uncertainties and limited field data, the tentative limits of relative rotation given in Table 1 may be suggested as a guide for usual types of structures. In general, the design of foundations and structures should include provisions for reducing or accommodating movements without damage, and suitable construction precautions should be taken to prevent excessive yield and movement of the ground.

Table 1. Tentative Rotation Limits for Structures

Relative Rotation (δ/ℓ)	Type of Limit and Structure
1/100	Danger limit for statically determinate structures, retaining walls and sheet pile walls
1/150	Safe limit for statically determinate structures, retaining walls and sheet pile walls
	Danger limit for open steel and reinforced concrete frames, steel storage tanks and tilt of high, rigid structures
1/250	Safe limit for open steel and reinforced concrete frames, steel storage tanks and tilt of high, rigid structures
	Danger limit for panel walls of frame buildings
1/500	Safe limit for panel walls of frame buildings
1/1000	Danger limit for sagging load-bearing walls
1/1500	Safe limit for sagging load-bearing walls
	Danger limit for hogging load-bearing walls
1/2500	Safe limit for hogging load-bearing walls

REFERENCE

Meyerhof, G.G. (1953). Some Recent Foundation Research and its Application to Design. Struct. Engr., London, Vol. 31, pp. 151-167.

Chairman Kantey

Thank you, Dr. Meyerhof. John, would you like to have a word?

General Reporter Burland

I am just a little concerned about Professor Meyerhof's updating of Bjerrum's proposed rotation limits. I do not necessarily disagree with them, but when simple guidelines are put forward they are often rapidly adopted as rigid rules. Thus, if Prof Meyerhof's proposals are reproduced elsewhere I hope they will be referred to as "routine guides'. Moreover, it must be stated in bold print on the table or diagram that each building or structure should be treated on its own merits for its performance will depend on a large number of factors including construction materials, method and form of construction, type of cladding and brittleness of finishes.

Panelist Meyerhof

I fully agree with this, and it will be so mentioned in the discussion.

Chairman Kantey

Would you like a minute, Victor?

Co-Reporter de Mello

I entirely agree with Dr. Burland, and despite the immense respect for the very brilliant solutions proposed I would mention the fact that a lot depends on the physical model selected, and it includes so many variables that are not known that we have to be careful about the overgeneralization. Man is very apt to grab at the first philosopher's stone possible, and we have to watch against that. Dr. Meyerhof's interjected reminder fits in very well with my emphasis on shying away from statistics of extremes, but it does not signify that we can avoid the reality of a statistical approach, hopefully realistic.

Chairman Kantey

Thank you. I would now like to ask Prof. Yamaguchi to give us his presentation.

Panelist: H. Yamaguchi

I would like to make some comments on one of the topics raised by General Reporet, "Allowable Deformation of Foundations and Structures for Acceptable and Unacceptable Damage".

As is well known, the smaller the factor of safety with regard to the bearing capacity Fs, the greater the settlement. To discuss this qualitatively, I assume that the relationship between the load intensity q and the settlement s can be expressed by a hyperbola. This assumption was successfully adopted by Kondoner for triaxial compression test results and by Ching for loading test results on piles. By using this, the relationship between s/q and s becomes linear, as is indicated in Fig. 1(b). In Fig. 1(a), q_u is the ultimate bearing capacity, k_i the initial sugrade reaction and s_e a hypothetical elastic settlement corresponding to the limiting state.

Here I would like to examine if this hyperbolic approximation is valid for actual measurements in various papers presented to this Main Session No. 2. Fig. 2 shows the results for drilled shafts in soft rocks and it can be seen that the linearity is fairly good. In Fig. 3, data for eccentrically loaded piles are shown and the linearity is almost perfect as the author himself pointed out. Fig. 4 is a selection of four results for piles, among which are included slightly peculiar

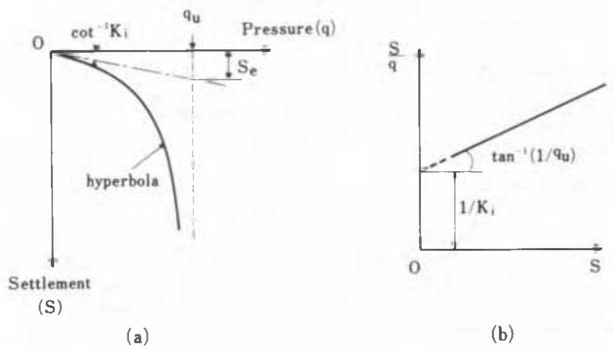

Fig. 1 Key sketch for hyperbolic fitting

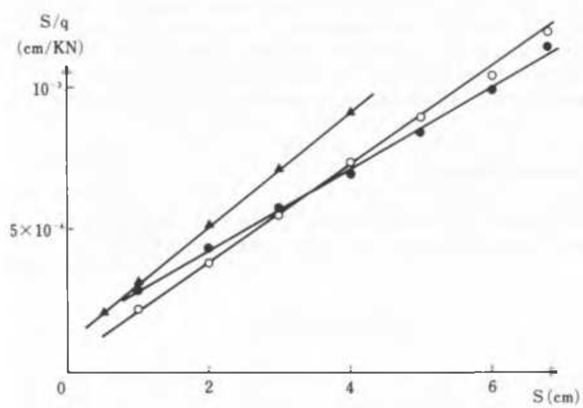

Fig. 2 R.P. Aurora et al. [2/2]

data. As was expected some divergence from the hyperbolic representation is found. Fig. 5 shows the results of surface loading tests and the linearity can be recognized on the whole. Fig. 6 is for bored piles, for which we can conclude that the hyperbolic approximation cannot be applied, especially the divergence is considerable at the initail stage of loading. For bored piles in Fig. 7, the similar trend can be seen. But results for bored piles in Fig. 8 shows a satisfactory linearity except one case. Fig. 9 is the re-

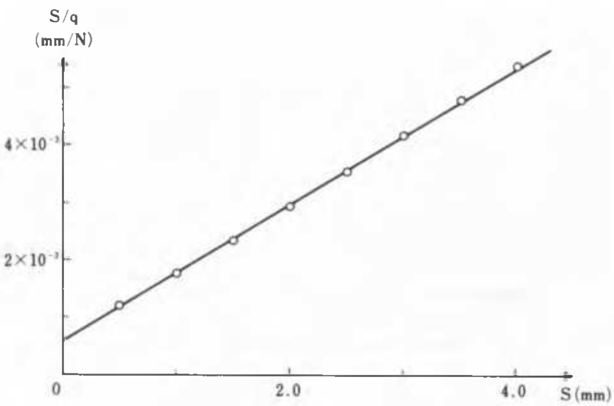

Fig. 3 F.K. Chin et al. [2/19]

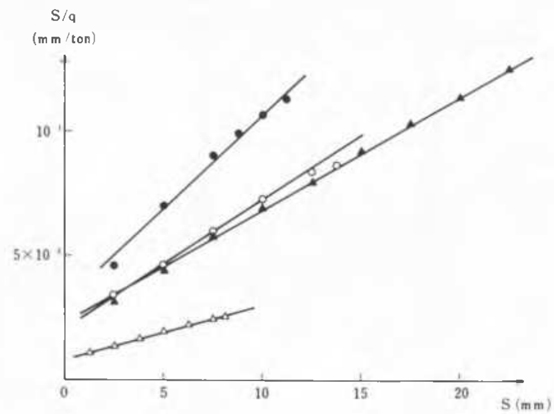

Fig. 4 A. Evangelista et al. [2/27]

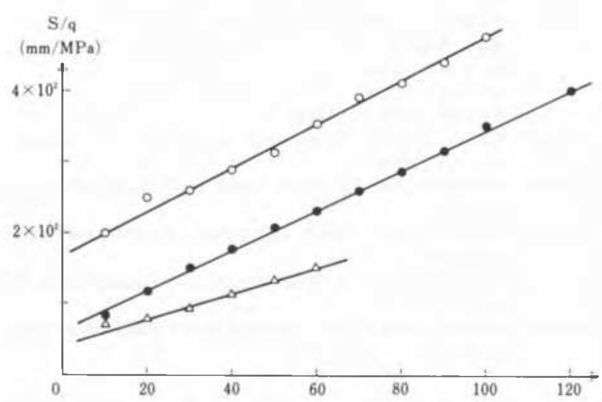

Fig. 5 J. Feda et al. [2/29]

sults for reinforced sand, which yields a very linear relationship. Fig. 10 shows data for footings with piles and the linearity holds generally. The last example is for bored piles with hollow cylindrical cross section.

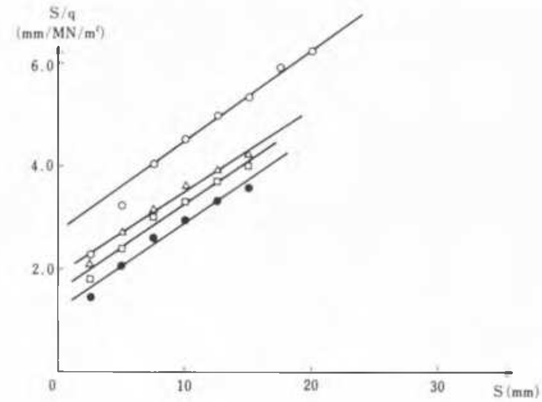

Fig. 6 E. Franke et al. [2/33]

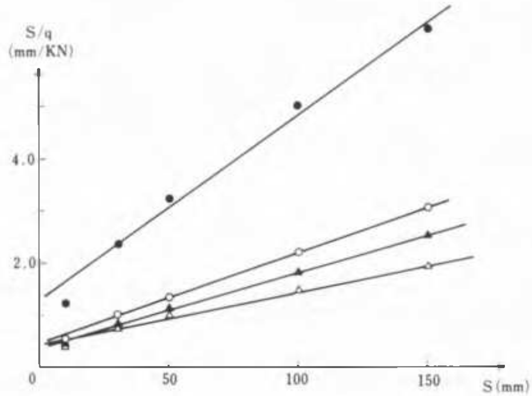

Fig. 7 R. Jelinek et al. [2/42]

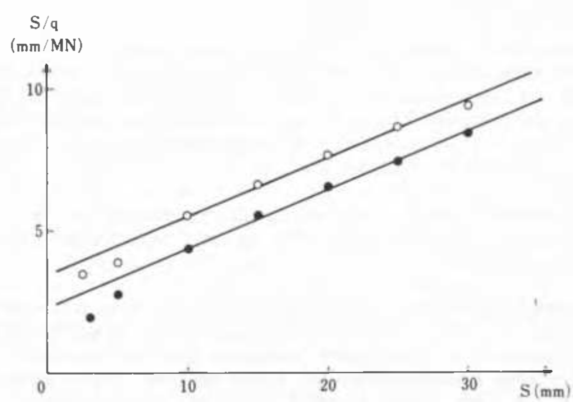

Fig. 8 B. Klosinski [2/50]

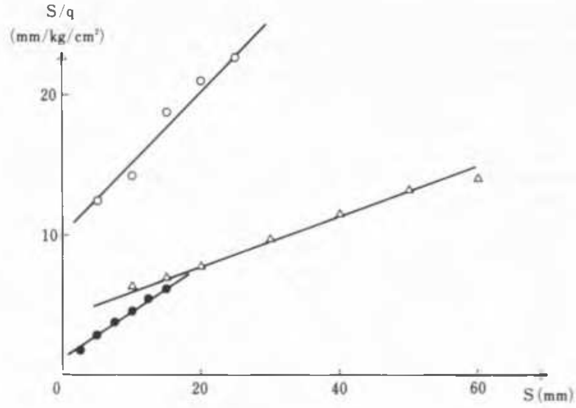

Fig. 9 D. Milović [2/59]

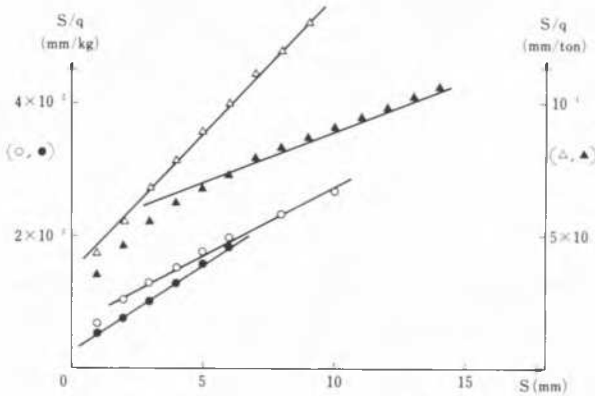

Fig. 10 J. Pałka et al. [2/67]

From these examinations, I am inclined to lead a conclusion that the hyperbolic approximation would not give rise to any serious error for almost all the types of foundations.

The assumption of the hyperbolic relationship between q and s gives the equations (1) and (2) in Fig. 12, where F_s is equal to q_u/q. In the same figure, F_s is plotted against a parameter s/s_e. When F_s is smaller than 2, the settlement s as well as the gradient of the settlement ds/dq are quite significant. On the while, when F_s exceeds 3, there is no substantial change in both values, which shows for this range the ground remains elastic. This conclusion is in agreement with that of Peck et al. and of Davis.

As was pointed out in the SOA Report, in the case of oil storage tanks to which large live loads are applied rapidly, the bearing capacity holds the key to the safety of the structure. Especially as de Beer and Bjerrum and others, as well as Simon and others have warned, there is possibility of local failure around the tank shell.

For oil tanks on the soft ground, total or local factor of safety is generally small, and therefore excessive settlement is liable to occur. As the factor of safety is very small around the shell, significant shell settlement will take place. This also causes differential settlements of the tank shell.

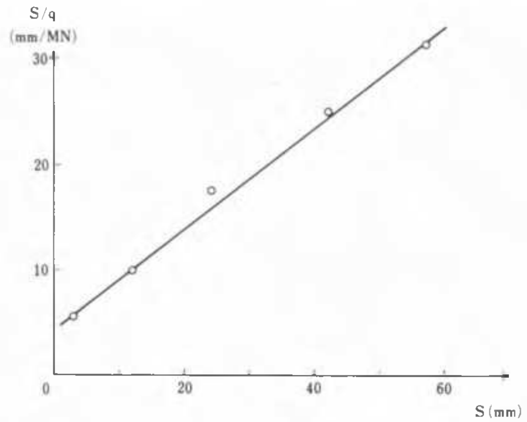

Fig. 11 G. Stefanoff et al. [2/81]

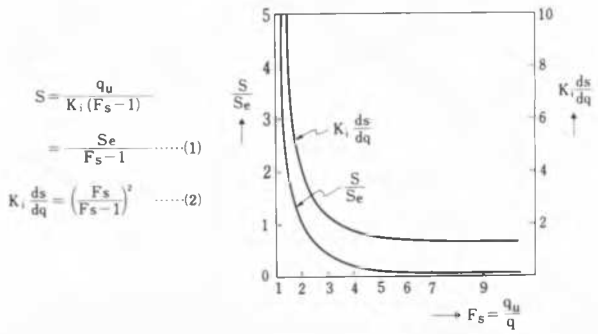

Fig. 12 Factor of safety vs. settlement and its rate

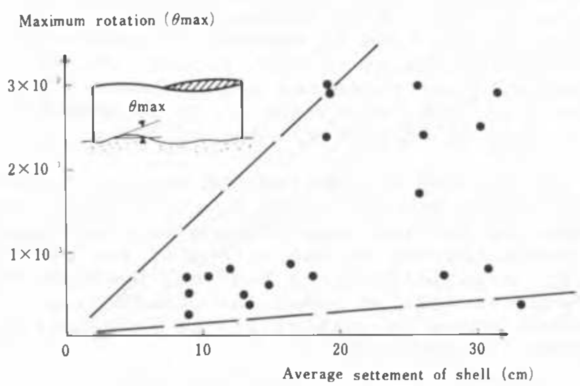

Fig. 13 Oil storage tanks on sandy soil, SAKAIDE

Fig. 13 shows the measured settlement soon after the water tests of large tanks standing on deep, sandy soil in Sakaide area. Although there is some scatter, this indicates that the greater the settlement of the tank the greater the maximum rotation angle.

Fig. 14 is similar settlement data taken in the Mizushima area. The soil is cohesive and the tank was constructed on a sand pad.

A similar trend is detected. Fortunately these tanks have not had accidents so far. It is fundamentally important to improve the factor of safety against local failure and minimize settlement. This will also lead to increase in the earthquake bearing capacity. Thank you very much.

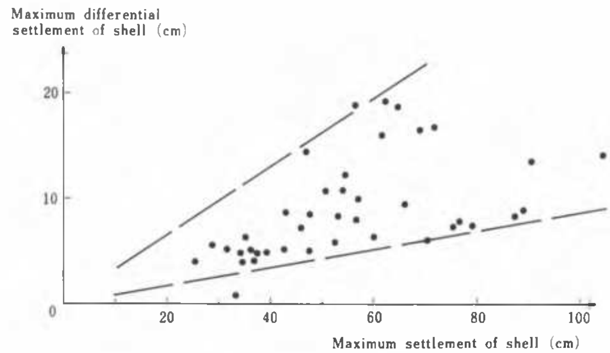

Fig. 14 Oil storage tanks on cohesive soil, MIZUSIMA

Chairman Kantey

Thank you. Jerry Leonards, could you wind up the panel discussion?

Panelist Leonards

Ladies and gentlemen, it is my fond hope that the Ninth Congress, among its many other accomplishments, will reach agreement on the definition of terms to be used in describing the phenomena associated with compression and consolidation of clays.

There is so much confusion with regard to the meanings of terms it has obscured our assessment of each other's data. It does not matter so much which set of definitions we agree on, so long as we agree. My suggested definitions are listed in Table 1.

The term "compression" implies any kind of volume reduction and the term "expansion" to the reverse.

"Consolidation" is a particular type of compression that is accompanied by a significant increase in effective stress due to corresponding reductions in pore water pressure. This is the classic definition by Terzaghi.

"Primary consolidation" is the volume change due to consolidation that results from that portion of the strain-effective stress relation that is independent of the rate at which the effective stress is applied. "Secondary consolidation" is the volume change due to consolidation that results from that portion of the strain-effective stress relation that is dependent upon the rate at which the effective stress is applied.

The term "secondary compression" is reserved exclusively for that portion of the volume reduction that takes place at sensibly constant effective stress. Of course, primary and secondary consolidation occur simultaneously; secondary compression is the time-dependent volume change after consolidation is complete. The distinction between primary and secondary consolidation is useful only in comparing analyses in which strain-rate effects are, or are not, considered.

Table 1. Suggested Definition of Terms

Compression: Reduction in volume

Consolidation: Compression accompanied by significant increases in σ', due to reductions in u

a) Primary consolidation - consolidation due to that portion of ε vs. σ' that is independent of $d\sigma'/dt$. ε vs. σ' relation may be linear or non-linear

b) Secondary consolidation - consolidation due to that portion of ε vs. σ' that is dependent on $d\sigma'/dt$.

Secondary Compression: Volume reduction at sensibly constant σ'

One of the concerns of the designer expressed in Main Session 1, and again in the State of the Art Report for this session, is the validity of conventional methods for estimating consolidation settlements from oedometer tests. The main source of this concern is due to a conception of the consolidation process proposed by Bjerrum, initially in his 7th Rankine Lecture and elaborated at the Purdue Conference (Bjerrum, 1972). I am aware that you are all familiar with the concepts of delayed consolidation, so I ask your indulgence if I review a few important points. Figure 5 shows a series of parallel lines representing void ratio (e) vs. effective stress (σ') relations for different rates of application of σ'. At a given depth in a sedimentary clay deposit the effective overburden pressure (p_o) has been acting for 10,000 years and the existing void ratio is e_o. Now, if a consolidation test is performed applying load increments every 24 hours the supposed result is shown by the solid curve exhibiting a quasi-preconsolidation pressure; if the increments are applied every 30 days, the dashed curve should be obtained. In either case, if ($p_o + \Delta p$) is maintained for 100 years the void ratio is supposed to reduce to the value indicated by the 100-year e vs. σ' curve. Of greatest practical significance is that the quasi-preconsolidation pressure is supposed to "vanish" if the consolidation occurs over a long period of time.

Fig. 6 shows the results of a carefully controlled laboratory sedimentation-consolidation test (Leonards and Altschaeffl, 1964). The initial stress increments were applied continuously and slowly enough so that no excess pore pressures were generated during sedimentation to point A, after which σ' was held constant for 40 days. The curve BCDEFG was then generated by applying stress increments every 24 hours. At a value of σ' corresponding to C, consolidation (p.p. dissipation) was complete in less than one hour and the secondary compression curve was extrapolated to obtain the times corresponding to points X and Y. According to Bjerrum's hypothesis point Y should be reached in 40 days: in reality, point X was reached in 10^6 days and point Y in 10^{60} days. For this reason (see also Leonards, 1972) I do not believe the quasi-p_c' vanishes, and I have used it in practice for over 15 years without a single adverse result.

The data from Berre and Iversen (1972) are most often cited in support of the contention that consolidation settlements in the field, which take place over a period of years, will be much larger than those measured in the oedometer, which occurred in a few hours--or less. Fig. 7 shows a set of data from this paper (for increment No. 5, past p_c'). If consolidation is defined as in Table 1, then the strains for a sample 0.74" thick consolidating in 10^3 minutes essentially equaled the strains for a 17.7" thick specimen consolidating in 30 x 10^3 minutes. Of course, I had to extrapolate the pore pressure curve to make the comparison. This emphasizes the importance of distinguishing the consolidation phase from the secondary compression phase.

Over the years I have made a number of comparisons between consolidation settlements predicted from oedometer tests with those

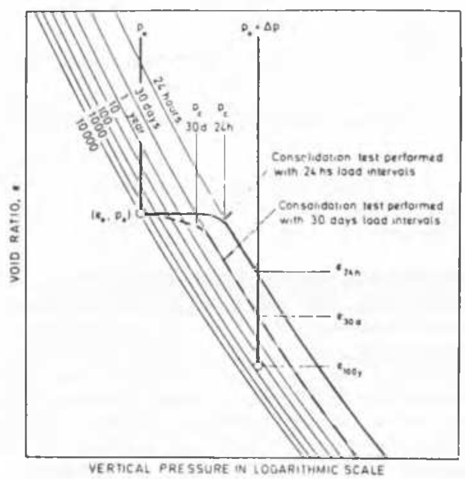

Fig. 5 Void ratio vs effective stress for different rates of loading (after Bjerrum, 1972)

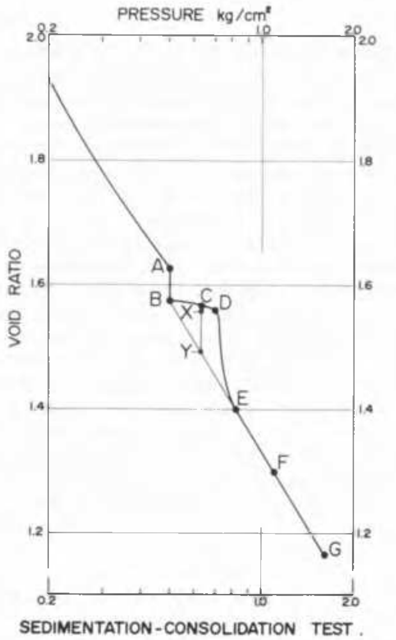

Fig. 6 Laboratory sedimentation-consolidation tests (after Leonards and Altschaeffl, 1964)

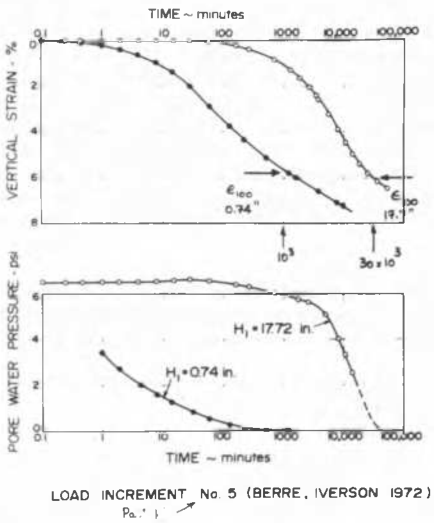

Fig. 7 Strain and pore pressure vs. time for samples of different height (after Berre and Iversen, 1972)

measured in the field, the most extensive of which was made for buildings in Drammen (Leonards, 1968). Fig. 8 is representative of the results, for conditions ranging from $(p_o + \Delta p)$ far below p_c' to well above p_c'. If p_c' is properly assessed--and I know of no way of doing this other than by carefully-run consolidation tests--the agreement is very good indeed. This is strong pragmatic evidence supporting my view that even though pore pressures dissipate much more slowly in the field than in laboratory consolidation tests, the consolidation strain measured in

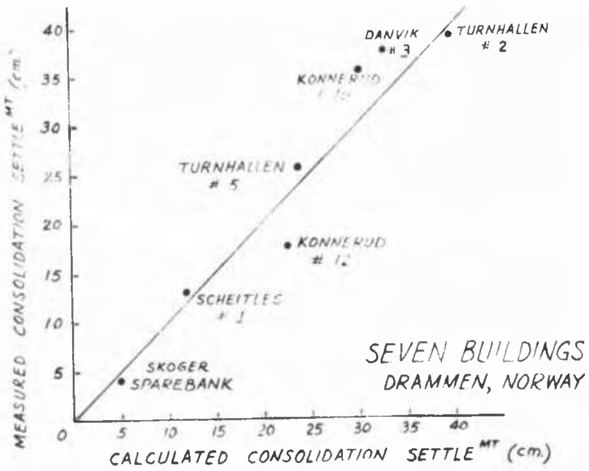

Fig. 8 Comparison of predicted vs. measured consolidation settlements (after Leonards, 1968)

the laboratory is a reliable index of the consolidation strains to be expected in the field.

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Chairman Kantey

Thank you very much, Jerry. I think the very broadness of the scope of this session has led to the situation where we have had to overrun our time despite the fact that we stole a 20-minute remission and now we have very little time left for discussion from the floor. So I would ask those who come up to be as brief as possible, starting with Dr. Tatsuro Okumura.

T. Okumura (Japan)

Thank you, Mr. Chairman. I have just one question and a few comments to Drs. Broms and Boman's paper, lime column, new type of vertical drain. We developed a very similar method for soil stabilization to Dr. Broms' which was presented to the Fifth Asian Regional Conference called "Deep Lime Mixing Method". This DLM method is now widely used in practice in Japan for preventing heaving of excavated trench, increasing safety factor against slope failure, as well as reducing the settlement of foundation.

However, I think it is not proper to use this method as a vertical drain. The data obtained from our laboratory oedometer test indicate that the compressibility of the lime treated soil decreases considerably, and the coefficient of consolidation increases by about 10 times, but the coefficient of permeability doesn't change so much. I'm sorry I didn't prepare any slides of our data, but it's of the order of 10 to minus 6 to 10 to minus 7 centimeters per second.

According to Professor Aboshi and Professor Yoshikuni's paper, the drain material should have permeability higher by the order of 4 or 5 than the surrounding soil. On the other hand, the lime treated soil becomes very hard or very strong. The lime columns behave like a pile, and the vertical stress concentrates to the lime columns, and thus the total settlement decreases considerably, I think.
So I would appreciate it if Dr. Broms will show data on the permeability of lime columns in Sweden and explain in detail on the mechanism of the settlement reduction by your lime column. Thank you.

Chairman Kantey

Thank you Dr. I don't think we can allow Dr. Broms time to explain in detail, but I'm sure that Dr. Broms would like to reply for one minute.

Panelist Broms

There are two effect when lime columns are used in soft clay. One effect is to reduce the compressibility of the soil. The second effect is to increase the permeability of the soil. Numerous laboratory tests, primarily with clays from Sweden, indicate that the permeability is increased 100 to 1000 times by the addition of unslaked lime. The permeability of the stabilized soil is about 10^{-3} to 10^{-4} cm/sec. This means that the stabilized soil will have same permeability as silt or fine sand.

Available field data support the results from the laboratory tests. The equations which have been derived by Barron (1948) can be used to calculate the settlement rate for lime columns. The lime columns seem to function in soft clay as drains in the same way as sand drains or plastic-paper drain (Geodrains).

REFERENCE

Barron, R.A., 1948. Consolidation of Fine-Grained Soils by Drain Wells, Trans. ASCE, No. 2348, Vol. 113, pp. 718-754.

Chairman Kantey

Thank you. Could we have Prof. Veder on the floor, please, and if Dr. Preiss will get ready to follow him.

C. Veder (Austria)

Mr. Chairman, I would like to refer to papers like the ones by Jelinek, Korek, Stock - and others, in which the load distribution of the point and on the skin friction is measured on a test pile. First I would like to speak to you about the measurement of skin friction and base resistance of groups of load bearing elements for 600.000 KN under an important building of a hight of 110 meters. It is a part of the International Administration and Conference Center of the United Nations in Vienna, and I think it is a typical example for examinations of the bearing behaviour of elements of a deep foundation. The subsoil consists primarily of fill and sand-gravel formations near the surface; underlying to a maximum depth of 70 m alternating there are layers of Viennese Tegel. This is a mixture of clay and silt, and middle sand. (Fig. 1)

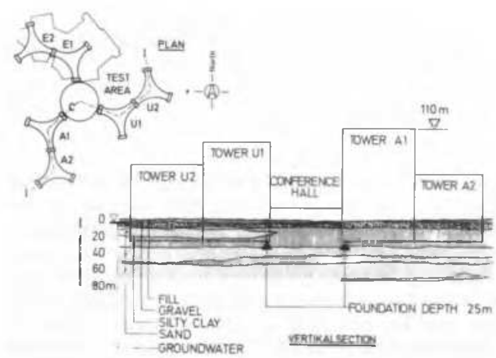

Fig. 1

You see here this high building I referred to, 110 meters, and here these different layers, rather random, of fill, sand layers, and the Viennese clay sand (Tegel).

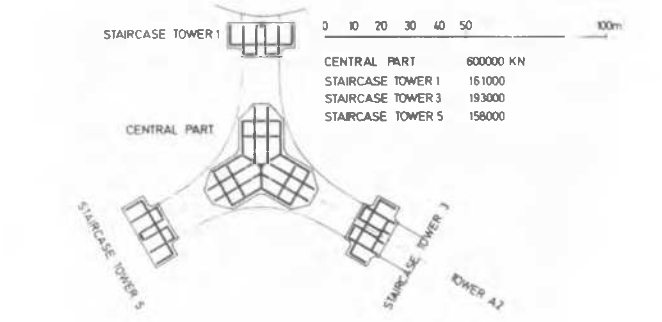

Fig. 2

My colleague Prof. Borowicka and I agreed that the right solution would be a deep foundation down to 25 meters, as indicated here, and you will see on the second slide, how this foundation was built. (Fig. 2)

You see here a grid of slurry trench walls of 80 centimeters thickness. Here you have the center part of this building. It is loaded with 600.000 KN, and in the shape of a Y you see the staircase of approximately 16.000 KN. Borowicka and I were oblidged to guarantee that the total settlement of this building would not be more than 5 centimeters and the differential settlement between the central part and the staircase not more than 2 centimeters. You understand this is a very hard task.

Now, in order to determine more accurately the load bearing behaviour of this diaphragm wall groups, 18 pressure gauges were placed at the bottom of the diaphragm walls. (Fig. 3)

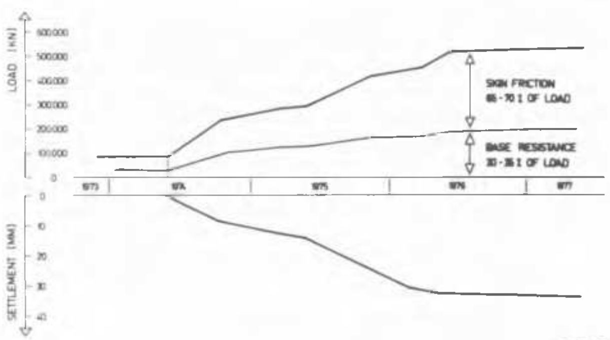

Fig. 3

The base and the shaft resistance rose proportionally, as you see on the slide 3, increasing with the load. The upper curve shows the total which was applied on the head of the slurry trench walls, and we could measure the base resistance by means of the pressure cells placed on the bases, and by the difference we found the skin friction. The distribution of the skin friction along the slurry trench walls and around them is still to be considered, as Broms mentioned, the differential settlement between the diaphragm walls placed on the perimeter and the center of the groups. Now nearly the total load of 600.000 KN is applied, and the settlement is less than the forseen 5 centimeters.

Chairman Kantey

We now have Dr. Preiss, if he is ready, you have two minutes, Doctor.

K. Preiss (Israel)

We have performed integrity tests with specially developed gamma ray scattering instrumentation on 2049 bored piles or diaphragm wall elements, over 5 years on 68 projects (3). Most tested piles or elements were poured into bentonite, but a few shallower piles were cast dry.

We have found that 16% of piles had some defect, usually of no structural significance. Of all the piles, 6% showed defects which required attention. At the minimum, this attention constituted discussion between all the parties concerned and decision to tolerate the defect, but in fact many of these piles were repaired.

Only two papers in the literature summarize results of many integrity tests of bored piles; these give similar numbers of 9% (1) and 7% (2) of piles found to have defects.

On the basis of this considerable accumulated experience it appears that the number of bored piles with some imperfection may be greater than is usually appreciated. One should therefore bear in mind that not only is the soil variable, but also for many structures the load-deflection characteristic for nominally identical cast piles is not uniform. This non-uniformity affects the deflection and stress distributions in many practical cases. Engineers are often surprised by the assertion that as many as 10% imperfect piles may be found in routine projects. However, the fact that distress is not visible in many existing structures under normal load is not proof that the piles are all without defect, because under normal working conditions many structures may manage to redistribute loads between piles without showing distress, so that the effect of defects in piles is often hidden.

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Chairman Kantey

Very well done. Victor, you can have a minute to comment.

Co-Reporter de Mello

I was just agreeing that execution effect is the principal problem. I think we all think in terms of that. Thank you.

Chairman Kantey

Gentlemen, we are all running a little bit late, and I'm afraid we'll have to finish with three minutes from Dr. Fellinius. I'd like him to confine his remarks to the negative skin friction portion of his discussion, to be followed by Drs. Horvat and Hansbo, a minute and a half each, which will give us six minutes. Dr. Fellenius, you have 3 minutes starting from now.

B. Fellenius (Sweden)

The paper by Horvat and Veen, Session 2, provides an interesting reading from an engineering point-of-view. However, the writer takes issue with one aspect of the paper, namely the "Safety Analysis".

The authors present a typical case of a pile having a ultimate bearing capacity $Q^u = Q^u_{end} + Q^u shaft = 145 + 20 = 165$ tons and being subjected to a drag load $P_n = 65$ tons. Based on these values, the authors calculate an allowable pile load, P_a, using a safety factor of 1.7 on Q^u and 1.1 on P_n as follows:

$$P_a = \frac{1}{F_s} \cdot \left(Q^u_{end} + Q^u_{shaft} - 1.1\ P_n \right)$$

$$= \frac{Q^u}{F_s} - \frac{1.1 P_n}{F_s} = \frac{165}{1.7} - \frac{1.1 \times 65}{1.7} = 55 \text{ tons}$$

The writer holds that it is principally incorrect to reduce the drag load as shown above. To determine the maximum allowable load in consideration of the drag load, an approach using partial factors of safety should be used, as follows.

$$f_p P_a \leq \frac{1}{f_q} (Q_e + Q_s) - f_n P_n$$

The particular partial factors to choose will vary from case to case. Generally, partial factors of safety vary from 1.1 to 1.3. Greater values are not safety factors, but ignorance factors. The following numerical values are chosen for illustrative purposes and are not generally valid.

$$1.2 \times P_a \leq \frac{1}{1.3} \times 165 - 1.1 \times 65$$

$$P_a \leq 46 < 55 \text{ tons}$$

To reach the load of $P_a = 55$ tons, the chosen partial safety factors will have to be reduced, provided the Q^u-values are known with greater assurance than implied by the writer's arbitrarily chosen value of 1.3. It is an advantage of the method that the uncertainity of any part is discovered. The above derived maximum value of P_a does not include any transient loads, which are balanced out by the drag load, as shown by Fellenius, 1972. The above approach will show a maximum allowable permanent load on the pile. To conclude the design, the structural integrity of the pile must be checked, whereupon the main point to check is the expected settlements. The structural capacity of the pile can be taken as 2/3 of the strength (concrete cube or cylinder strength, or yield point of steel). The load to apply is the load at the neutral point = $f_p \times P_a + f_n \times P_n$. This structural capacity differs from the usual values of structural capacity of a pile given in Codes and Regulations, which values are given with respect to ordinary pile loads, that is, with various miscellaneous loads such as drag loads already deducted.

Settlements are to be studied by means of conventional soil mechanics theory. The load $f_p \times P_a + f_n \times P_n$ is to be carried by the soil below the neutral point, say in competent layers at or near the pile tip, or in case of no such layer, at the lower third point of the pile length below the neutral point.

One of the above three approaches will determine the maximum allowable load. If this is less than the currently applied load in the local area, measures to reduce the drag may have to be introduced, for instance, bitumen coating of the pile to reduce the drag. The three approaches can then again be used to determine the length of pile to coat to reach an economic optimum.

REFERENCE

Fellenius, B.H., 1972: "Drag loads on piles due to negative skin friction", Canadian Geotechnical Journal, Vol. 9, No. 3, 1972, pp. 323-337.

Chairman Kantey

Thank you very much, indeed. Dr. Horvat, please.

E. Horvat (Netherland)

The allowable bearing capacity calculated with our method is the same as it was calculated in the past when the negative skin friction was neglected.

When we use the method, which was shown by Dr. Fellenius, which we normally do, the factor of 1,3 for the bearing capacity is less. It is 1,15 to 1,2 depending on the soil investigation, method of calculation and other aspects.

I fully agree that the load deformation be-

haviour can together with the bearing capacity be a limit for the design.
It may be remarked again that the calculated bearing capacity with our method is exactly the same as it was in the past and that it is based on recalculations of foundations, from which we know the behaviour for as long as 15 or 20 years. Thank you.

Chairman Kantey

Thank you, Dr. Horvat. Dr. Hansbo, please.

S. Hansbo (Sweden)

I would like to highlight some of the problems involved in consolidation, particularly with reference to the use of vertical drains. The most significant parameters to be used as a basis for the design of vertical drainage systems and the need for surcharge are the coefficient of consolidation in horizontal pore water flow c_h and the preconsolidation pressure. As regards c_h Professor Ladd yesterday gave an example of how this can be determined by means of the piezometer probe developed by Torstensson (1975). As regards the preconsolidation pressure, the panelist, Professor Leonards, made some interesting comments. Professor Leonards referred to the fact that the breakpoint in the excess pressure increase during CRS test indicates the preconsolidation pressure valid for longterm loading. I agree with Professor Leonards in that when the preconsolidation pressure is exceeded it is accompanied by a structural break-down of the clay skeleton. An internal shear failure is obtained, followed by load transmittance from the clay skeleton to the pore water. The preconsolidation pressure could therefore just as well be defined as a critical shear stress level, and using this definition we get a better understanding of the dependency on the rate of strain used in the oedometer test.

Professor Leonards showed one of the results of the excess pore pressure studies in a CRS test presented by Sällfors (1975). I agree with his interpretation. But since the excess pore pressure Δu is directly related to the coefficient of consolidation c_v he could just as well have chosen c_v instead of Δu. If this is done, we find that the longterm value of the preconsolidation pressure corresponds to the effective stress value where c_v starts to decrease. We also find that the stress interval in which c_v is gradually decreasing is quite large, Fig. 1.

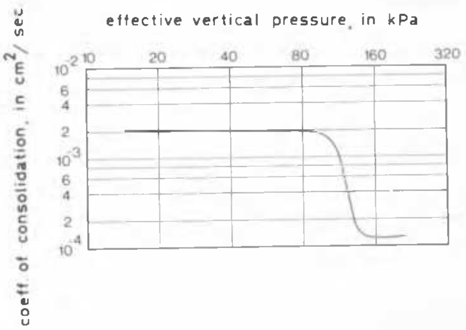

Fig. 1 Coefficient of consolidation c_v vs effective stress for soft, high-plastic clay from Gothenburg. CRS oedometer test (Sällfors, 1975)

This of course complicates the estimation of the consolidation process in a case where the load placed on the soil gives a stress increase within this stress interval. However, in the case of vertical drains, where the drain installation causes some disturbance effects, a design based on the minimum c_v value seems to give the best fit with results obtained in practice.

Quite often, the degree of consolidation obtained after a certain time of consolidation is checked by means of pore pressure observations. I would like to show how difficult this might be.

At Skå-Edeby, with 4 test areas, excess pressures of 5 to 15 kPa have been observed in all test areas, although in the test areas provided with sand drains, primary consolidation ceased 10-15 years ago. Even where excess pore pressure had vanished almost completely due to unloading 15 years ago and where hardly any settlement has taken place since, an excess pressure of about 10 kPa has later been built up with time. A possible explanation to this phenomenon may be secondary compression or cyclic loading caused by varying degree of saturation of the fill with the time of the year in combination with a non-Darcian relation between hydraulic gradient and pore water flow.

In my and Torstensson's article about "Geodrains and Other Vertical Drain Behaviour", presented in the Proceedings of this conference, great attention was paid to the practical significance of the durability - or rather

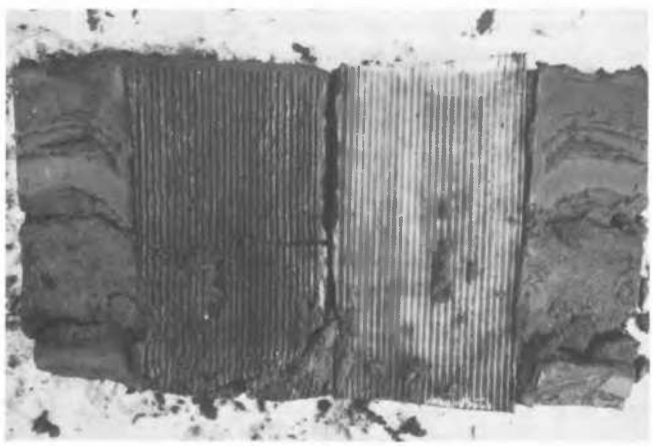

Fig. 2 Split soil sample from Porto Tolle, Italy containing Geodrain. The plastic core seen to the wright and the filter paper to the left. Deterioration of filter paper clearly visible

lack of durability - of the filter paper that surrounds the plastic core of the Geodrain. When the article was written the mentioned durability was a main concern of ours as we thought that a filter paper in good condition was a necessary requirement for the drains to function effectively. Investigations of the filter paper, made on samples taken from different depths e.g. at 11 m in Porto Tolle 17 months after installation, Fig. 2, were quite discouraging. The filter paper had seriously deteriorated.

Further, additional loads placed on old test areas with Geodrains installed 1 1/2 to 3 1/2 years ago did not seem to give much of an effect. We therefore drew the natural conclusion that the drains could not be expected to function effectively for more than 1 to 1 1/2 years. Our conclusion was, however, too pessimistic. Continued tests in Skå-Edeby, Helsinki, Schipluiden and Porto Tolle have given unquestionable evidence that a possible deterioration of the filter paper has no, or at least very little, influence on the effectiveness of the drain. It seems as if a filter cake of soil and partly disintegrated paper has been formed around the plastic core and that this cake has a high enough permeability not to cause any significant hindrance to the pore water flow.

This fact is clearly demonstrated in Figs. 3 and 4. Fig. 3 shows the result of the loading test at Porto Tolle. Comparisons of observed and theoretical settlement curves show that the Geodrains are still fully effective after 2 years.

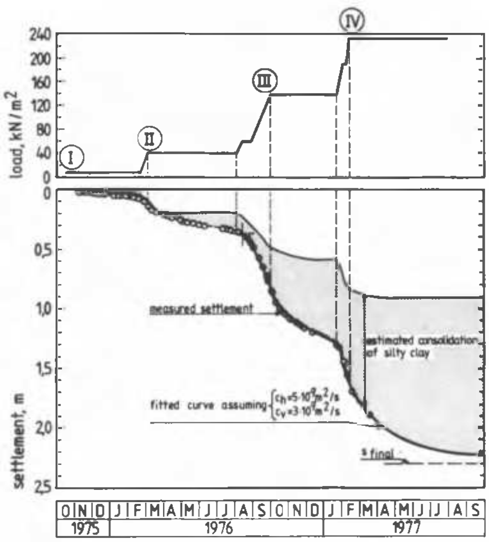

Fig. 3 Result of loading test at Porto Tolle, Italy. Geodrains, 3 m spacing. The settlement curve shows that the drains, in spite of the deterioration of the filter paper, are fully effective after 2 years. cf Hansbo, 1977

Fig. 4 shows the result of the loading test at Skå-Edeby. Comparisons of measured and theoretical settlement curves show that in this case the Geodrains are fully effective

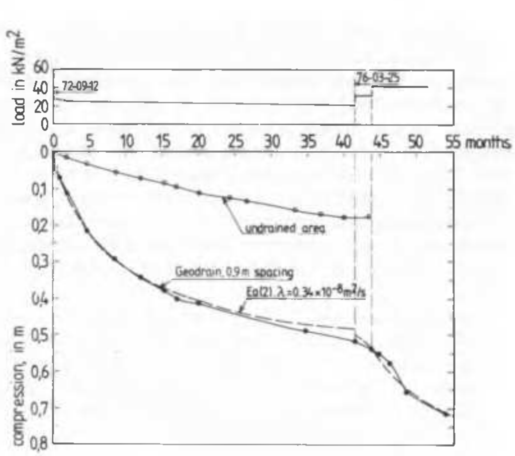

Fig. 4 Result of loading test at Skå-Edeby, Sweden. Geodrains, 0.9 m spacing. The settlement curve shows that the drains, in spite of the deterioration of the filter paper, are fully effective after 4 years. cf Hansbo, 1977

even after 4 years.

The previous suppositions that disintegration of the filter paper would block the drains have thus been contradicted by reality.

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Torstensson, B-A., 1975. "Pore pressure sounding instrument". Proc. Spec. Conf. on in situ Measurements of Soil Properties. ASCE/ Raleigh, N.C./June 1-4.

Chairman Kantey

Thank you, Dr. Hansbo. For the final discussion from the floor I would like to have Sam Thorburn who has something to say on regional studies which I think is an important aspect of this session.

S. Thorburn (U.K.)

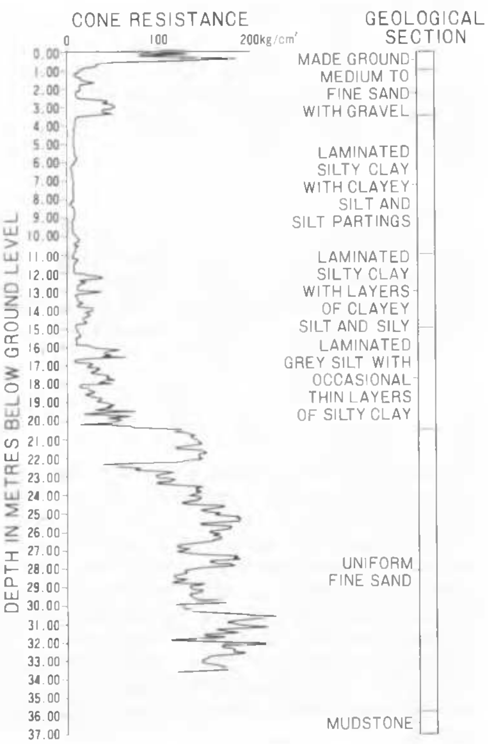
Mr. Chairman, it may save time if we project the first slide while I make a very brief introduction. Dr. Burland mentioned the importance of regional studies, and it may be of interest to the members here to know that structural engineers in the United Kingdom are studying the interface problem which exists between the structural engineer and the geotechnical engineer. The problem may be defined as the difference between the predicted or design performance and the actual performance.

Could I have the first slide, please. In order to resolve this problem, long-term studies are being made of the performances of buildings founded on well known geological deposits. One such study involves the performances of buildings founded on the alluvium of the River Clyde, and this slide indicates the general geological sequence which consists of laminated silty clay, silt and uniform fine sand. The static cone resistances are also shown on this slide.

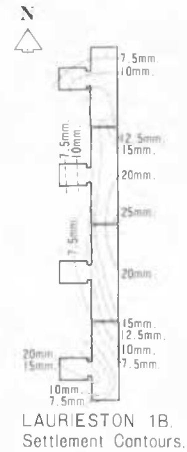
Next slide, please. This slide shown reinforced concrete slab foundation for a four-storey building 100 metres long by 8 metres wide. You will observe from the settlement contours that theory could not have readily predicted the configuration of the contours and that's the problem facing the structural engineer. It's all very well for the geotechnical engineer to feel important and to consider settlement predictions in isolation. The structural engineer is responsible for the structure and must have confidence in the predictions.

Next slide, please. This shows the settlement contours for 15-storey structures, and you will observe the values of maximum angular distortions given beneath the contours. The slide indicates a maximum angular distortion for a concrete shear wall and column structure, 15 storeys high, of 1 in 550. No evidence was found of any cracking in the structure. It performed very well despite the significant amount of structural distortion.

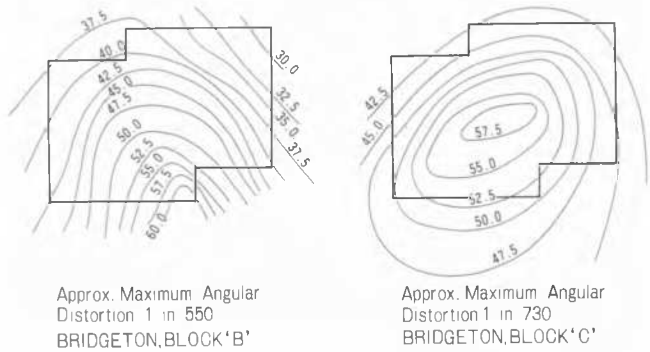
Last slide, please. We are also conducting an investigation into which portion of the



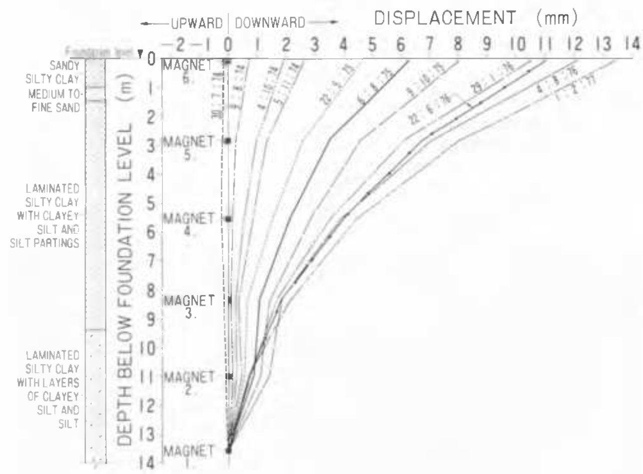
Slide 1



Slide 2



Slide 3



Slide 4

soil makes the greatest contribution to the settlement of structures, and B.R.E. magnet extensometers were installed in boreholes beneath five-storey buildings founded on reinforced concrete slab foundations. You will observe from the measurements that the depth of soil which contributes to the settlement is about one and a half times the width of the structure.
Thank you very much, Mr. Chairman.

Chairman Kantey

Thank you, Sam. I now have an extremely difficult task. I don't know how it is possible to summarize in a few minutes the very wide range of subjects that have been covered, not only in the State of the Art Report but in the panel discussion and the comments that we've had from the floor.

To me, two overriding messages have come through, certainly up to this point in this conference. The one is the vital need to get back to practice, as emphasized so very well by Victor de Mello, and to get away from this drive for ever-increasing theory, and more theory, and refinements of theory.

The other aspect which I haven't heard mentioned yet, except in private discussion, is that it seems to me there is a vast overemphasis on redeposited soils, the soft clays, and I think this is possibly because they are easy to work with, that there is a bit more homogeneity in them than in the residual soils which cover at least 90 to 95 percent of the practicing engineer's work. And I think if these two messages come out of the two sessions that we've had today, we will have achieved the objects of this panel, and certainly our State of the Art Report.

It reminds me to thank you all for your patience and endurance, and to thank in particular the panel for the contribution they have made and to declare this session closed.

WRITTEN CONTRIBUTIONS — CONTRIBUTIONS ÉCRITES

H.K.S.Ph. Begemann (Netherlands)

Comments on the Paper "Negative Skin Friction and Safety Analysis of Piles" Volume 1, Page 551 by E. Horvat and C. van der Veen

The authors state that in Rotterdam a load of 15 tons, caused by negative skin friction had to be taken into account. Now it is known that this friction can be 65 to 120 tons, and nevertheless the foundations made during the past 40 years show no failures. From these facts the authors jump to several (unproven) conclusions concerning lower safety coefficients and more "optimistic" and "reliable" calculation methods.
The fact that the mentioned buildings have shown no failures due to unadequate foundation cannot be used in this way as a prove for the conclusions (statements) made by the authors. It is not necessary that the actual higher skin friction is compensated by higher point resistances than calculated and/or by too high safety factors. There are other possible factors as for instance:
. lower actual pile loads
. more densification of the bearing sand layers in the area concerned than normally may be expected.
Factors as mentioned above have to be incorporated in the analysis of safety coefficients and calculation methods before one can come to a conclusion about a possible decrease of the different safety coefficients. It is very dangerous to reduce the safety factor for the point bearing and for the skin friction and at the same time recommending the most optimistic calculation methods, only because the maximum skin friction had proven to be 60 tons instead of "the old value of 15 tons".
The very last statement is not as bad as it is presented by the authors. For several years ago in the area concerned a point bearing capacity was advised and an extra pile-load of 15 to 20 tons in connection with negative friction (being the difference between the maximum negative and the maximum positive skin friction).
Also the following "statements" of the authors are unproven and disputable:
1) A safety coefficient of 1,1 for maximum negative skin friction is recommended. This is too low (how accurate is the calculation method?).
2) In report 47 (1975) from the foundation Building Research Rotterdam it is stated that the amount of negative friction calculated by the method Zeevaart-De Beer is strongly dependent on the number of layers. Recommended is to use the local friction method and the calculation method according to Begemann (the authors claim the opposite).
3) It is further stated by the authors that for the positive friction the method published by Broms is more reliable than the method based on the local friction. This is not proven by the example given in Horvat's paper or elsewhere.
4) Concerning the point bearing, Horvat-Van der Veen state that the calculation method based on cone resistances 4 times the pile-diameter below and 8 times above the pile tip "is clearly too pessimistic", he takes 1,5D and 5D. It is not clear at all.
   In the paper of Van der Veen, Volume II, IVth Conference 1956, a comparison is given of the results of 14 pile loading tests and the calculated values, using 1D and 3,75D. The ratio's between these two values varied between 0.48 and 2.18, the most extreme being the precast pile no. 10 with a maximum test load of 32 tf and

a calculated value of 70 tf, the 4/8D method giving the correct value! Due to this great variation Van der Veen calculated a required factor of safety of 2.5. It will be appreciated if the authors clearly show how these results fit in their statement that 4/8D method is too pessimistic and how on top of that the factor of safety of 2.5 recommended in 1956 now can be reduced to 1.7?

5) In connection with statement 7 and conclusion 5 the following may be noted. To our opinion one should make a choice between a pile foundation of which the pile tip bearing capacity is an essential part and fully taking into account the negative friction with normal acceptable safety factors which limits the settlement difference to max. 1½ to 2 cm and a "settling" pile foundation of which the bearing capacity of the pile tip is only small, using much lower factors of safety. A solution in between is too risky; some more piles is cheaper than strengthening the construction (see the paper of Inoue, Tamaoki and Ogai, Volume 1, page 561).

A. Burghignoli (Italy) and G. Calabresi (Italy)

In his general Report Dr Burland, commenting on the paper "Consolidation of a thick layer of Soft Clay", has raised some doubts about the Authors' interpretation of the piezometer records at Fiumicino. In particular, the General Reporter pointed out the importance of checking the electric piezometer reliability by contemporary records with twin tube hydraulic piezometers because of the presence of gas in organic soils.

The hypothesis that gas bubbles have entered the piezometer tips was of course the first to occur to Authors' mind and was considered since the beginning of the anomalous increase of pore pressures. However, after many doubts, they have disregarded it.

The piezometers were manifactured by MAIHAK, and have the characteristics described by Scott and Kilgour(1967). The cavity is very small (about 0.5mm thick and 3.4 mm in diameter) and the fine grained porous stone, saturated in the factory, has a thickness of about 20 mm. The same piezometers have performed well for a long period of time in partly saturated soils (Torblaa,1966) and checked against twin tube hydraulic piezometer(Viggiani,1974).

The mechanism of gas diffusion through the water in the porous stone and of bubble forming in the inside cavity has been widely discussed. It seems unlikely that this phenomenon occured at Fiumicino.

The rise of pore pressure has been noticed in the deep layers of more plastic clay, where the organic matter content is lower. Many piezometers have been installed in the upper peat layer or in the more organic clay near the surface, where the unsaturation is marked and the pore water pressure decreased more rapidly because of the presence of the sand drains. However, they followed the pore pressure dissipation and didn't show an anomalous increase of the measured values. In those conditions air should have entered much easier into the piezometer cavity.

At a depth larger than 15 m, with a pore water pressure of more than 2 kg/cm^2, a soil having in the laboratory a degree of saturation higher than 0.96, as the Fiumicino clay, should be fully saturated. If gas pockets at high pressure had been present near a piezometer, they would probably have emptied into the borehole. The inconveniences of pore pressure measurements in organic soil reported in the literature generally refer to low values of pore water pressure: Marsland(1974) refers to a pneumatic piezometer placed below high tide near a peat layer. The deepest piezometer shown in his paper should have a maximum value of pore water pressure of 0.7 kg/cm^2. Buck (1969) reports different records of hydraulic and pneumatic piezometers in the muskeg foundation of a road fill, presumably at small depths underground water level and in a soil with a small degree of saturation. Vaughan and Bishop(1969) think that at high values of pore water pressure the gas diffusion occurs very slowly and the inconveniences in measuring pore pressures are not significant.

Finally, it is worthwhile to remark that the increase of pore pressure values at Fiumicino was continuous over a period of more than four years. Should the piezometer cavities have become full of gas, a sudden change of the recorded values had to be expected and this should have occured at different times for the various piezometers. An automatic reading system is operating at Fiumicino for the more than 150 piezometers and scanning operations were repeated every day for the first three years and weekly afterwards. Such a behaviour was not noticed.

Therefore, the Authors' interpretation of the piezometer records at Fiumicino was considered at length and before concluding that the rise of pore pressure is due to an undrained creep they have doubtfully examined more obvious explanations.

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C. Dinis da Gama (Brazil)

A Soil Mechanics Approach to the Characterization of Jointed Rock-Masses

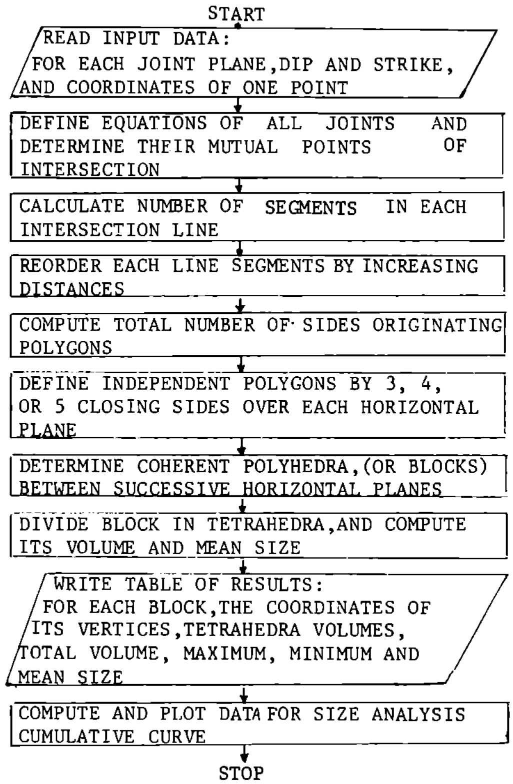
Regarding one of the topics proposed for discussion in Main Session No.2, I would like to present a contribution to the design of dam foundations in rock-masses.

Joints and other geological discontinuities have a controlling effect on the mechanical behaviour of rock-masses. In general, joints increase the deformability and reduce the strength of the rock-mass, with respect to the magnitude of those properties for the intact rock.

It is sometimes a very rough approximation to evaluate the behaviour of a rock foundation by means of measuring its properties on rock blocks only, and that procedure can lead to erroneous design practices, such as overestimating safety factors, or using unreal bearing capacities.

In order to assess the actual behaviour of jointed rock masses as foundation members, especially in the case of dam foundations, it has been recently developed (1) a computer model for their size analysis, using as inputs the common geologic data (dip and strike) of fracture planes existing in the volume affected by the structure under consideration. The program (which if required is available from the author) takes into account the three-dimensional intersection of joint planes within the rock mass, computes the volumes of the individual blocks, and outputs the size grading curves (histograms and cumulative results) corresponding to the volume under analysis.

The flow-chart of the computer program is presented below.



Although other applications of this model have already been attempted (such as rock blasting design and percolation studies) it is also envisaged for the prediction of foundation bearing capacities, using heterogeneous behaviour concepts.

It has been proposed (2) that the lower limit of bearing capacity in a jointed rock mass is given by the Brazilian compressive strength of the intact rock blocks, the strength of which is inversely proportional to block size. Therefore, a realistic evaluation of that lower limit is obtained through the knowledge of the maximum individual block size, which is supplied by our computer model. The strength variability within the rock-mass is therefore available, with respect to the actual block location, thus providing a complete picture of the bearing capacity distribution in a three-dimensional space. Obviously the heterogeneity evaluation becomes more accurate as the amount of information regarding joint attitudes increases, and it is suggested that geological mapping, coupled with core analyses and/or appropriate geophysical techniques, can produce sufficient data to obtain an accurate mechanical characterization of a rock-mass foundation.

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B.H. Fellenius (Canada) and O. Wager (Sweden)

The Equivalent Sand Drain Diameter of the Bandshaped Drain

This discussion deals with the evaluation of the Ska-Edeby test results with respect to the band shaped drains (Hansbo and Torstensson, Session 2). The mentioned authors assume the Geodrain to have an equivalent sand drain diameter of 5 cm, and using this value they evaluate that the drains mobilize a c_h-value of $1.2 \times 10^{-8} m^2/s$. In comparison, the results from the area with the 18 cm sand drains, placed at the same spacing, are evaluated to mobilize a c_h-value of $0.9 \times 10^{-8}\ m^2/s$.

The Geodrains in Ska-Edeby were installed with very crude equipment, far inferior to the modern flat low disturbance mandrels. Therefore, it is surprising that they should have shown such good effects in comparison with the sand drains, as indicated by the higher c_h-value.

The evaluated c_h-value depends very much on the mentioned assumed value of the equivalent drain diameter. The authors mention that the equivalent diameter corresponds to the diameter of a sand drain with the same circumference area. However, the Geodrain gross area of $20\ cm^2/cm$ corresponds to a circumference of cylinder with a diameter of 6.4 cm, not 5 cm stated by the authors. Still, this mathematically adjusted value is not the correct equivalent drain diameter.

While it is true that it is the surface of the drain - band shaped or sand drain - that governs the function of the drain, it is not the total surface, but the free unobstructed surface which should be used in establishing the value of the equivalent sand drain diameter.
The free (or open, or unobstructed) surface of the Geodrain is (was 1972) $13.9\ cm^2/cm$. The free surface of a sand drain is not as simply determined. However, in an arbitrary cut through soil, the area ratio of cuts through voids to cuts through solids is equal to the void ratio (e.g. Windisch and Soulie, 1970). This relationship is valid also close to the outer boundary of the drain. Therefore, the free surface area over the solid surface area is equal to the porosity of the soil. In case of a relatively uniform sand in a medium density state, as in a sand drain column, the porosity is about 0.4. That is, the free surface of a sand drain can be taken to be 40% of the total circumferential surface.

Therefore

$$0.4 \times \pi D \times 1 = 13.9$$
$$D = 10\ cm$$

which is a more logically derived value of the equivalent sand drain diameter to use for the Geodrain. If the Ska-Edeby results would be evaluated with this value, the results would be a smaller c_h-value than presented by the authors and probably quite close to that of the sand drain result.

Determining the equivalent sand drain diameter, as based on the free surfaces, provides a means of theoretically comparing different types of equal width band shaped drains with each other, which means is lacking when using the approach of total surfaces.

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E. Horvat (Netherlands) and
C. van der Veen (Netherlands)

Answer on the Discussions Concerning the Contribution "Negative Skin Friction and Safety Analysis of Piles"

I. The remarks of Dr. B. Fellenius, made during the oral discussion of Main Session II, are in full agreement with our ideas concerning the safety analysis of pile foundations. In view of the limitation as to the length of our contribution it was only possible to show a simplified safety analysis. The use of this simplified method had the additional advantage, that the essence of our theory - the influence of the negative skin friction on the safety - could be demonstrated in a more direct manner.

II. The non-simplified method of safety analysis, published by us in Holland concerning this subject, is herewith given:

$$\underline{P_k \leqslant (d \times P_u)} \text{ and } \underline{P_d \leqslant P_u}$$

P_k = characteristic load, being the working load (P_e) and the drag load (P_n);

P_u = design bearing capacity (ultimate value);

P_d = design load;

d = deformation factor;

The design load is $P_d = (P_n \times f_n) + (P_e \times f_e)$
where P_n = calculated drag load

P_e = calculated working load

f_n = load factor for the drag load, being 1 to 1.1 when the max. value of the drag load is taken into account;

f_e = load factor for the working load, being max. 1.7.

If the assumed drag load is less than the max. value (due to group effect, limited deformation, etc.) the load factor for the dragload must be increased.

The design bearing capacity is

$P_u = P/f_s$, where

P = calculated point bearing capacity and positive skin friction, both ultimate value;

f_s = uncertainty factor.

The uncertainty factor (f_s) depends on the type of pile, the installation method, the quality and amount of soil investigation, the type and strength-characteristic of the construction, the configuration and amount of piles, etc. The advised limits of this factor for driven piles are:

- wood piles, tapered — 1 to 1.2
- prefabricated contrete piles, without enlarged point — 1 to 1.2
- prefabricated concrete piles, with enlarged point — 1.2 to 1.5

The deformation factor is $d = P_k/P_u$ and the advised limits for it are:

- constructions sensitive to settlements: $d \leqslant 0.6$
- constructions not sensitive to settlements: $d \leqslant 0.75$

With respect to the material stresses in the foundation piles the maximum value of the drag load must be taken into account!

III. Regarding the expected remarks of Dr. Begeman and Mr. Heijnen (they were not granted an opportunity of an oral discussion) the following can be stated.

a) In our suggestion the real safety is not decreased, only the components determining the safety are re-grouped!

b) Our suggestion is based on observations made on the behaviour of a large number of buildings and other constructions in Rotterdam and Amsterdam, as well as the result of recalculation of foundations, taking into account the (observed and measured!) much larger negative skin friction than that wich was assumed during the design in the past.

M. Kany (F.R.G.)

Additional Remarks to my Paper 2/44

In the Preliminary General Report, Professor BURLAND asks me, what continuum model is used for evaluating the settlement coefficients $c_{\mu.\nu.W}$ and $c_{\mu.\nu.E}$. Here I will give the requested comments:

For calculating the settlement coefficients for point μ caused by the load of field No. ν, any of the known methods may be used, e.g. (for example) the finite element method according DUNCAN-CHANG or the BOUSSINESQ-method. We have used the last one in the wellknown modified form published by STEINBRENNER (1934)

In the corresponding computer-program we introduced different moduli of elasticity for loading and reloading.

In the computerprogram the following method is used:

At first the settlement $s_{\mu.\nu.W}$ of the foundation μ will be calculated by reloading $q_{\nu.\nu}$ of the neighbouring foundation ν. It results from l parts (for l layers under the foundation μ):

$$s_{\mu.\nu.\nu} = q_{\nu.\nu} \cdot B_\nu \cdot \sum_{i=1}^{l} \frac{\Delta f_{s.\mu.\nu.i}}{W_{s.\mu.i}} \qquad [\mathrm{m}].$$

In this equation the symbols stand in correspondance with my paper of the TOKYO conference (1977). Further is

$$\Delta f_{s.\mu.\nu.i} = f_{s.\mu.\nu.i} - f_{s.\mu.\nu.(i-1)} \qquad [1]$$

= difference of settlement coefficients for bottom side and for top side of layer i

$W_{s.\mu.i}$ = modulus of elasticity [kN/m^2] into layer i under foundation.

The settlement coefficients $f_{s.\mu.\nu}$ in each case we obtain with STEINBRENNER's formula (1934) by superposition of 4 part-areas r with the following formula (for $m=\infty$):

$$f_{s.\mu.\nu.i} = \frac{1}{2\pi B_\nu} \sum_{r=1}^{4} \Big[z_i \cdot \arctan \frac{a_r \cdot b_r}{z_i \cdot c_r} + b_r \cdot \ln\left(\frac{c_r - a_r}{c_r + a_r} \cdot \frac{M + a_r}{M - a_r}\right) + a_r \cdot \ln\left(\frac{c_r - b_r}{c_r + b_r} \cdot \frac{M + b_r}{M - b_r}\right)\Big] \qquad [1]$$

with

$$c_r = \sqrt{a_r^2 + b_r^2 + z_i^2} \quad \text{and} \quad M = \sqrt{a_r^2 + b_r^2}.$$

The settlements $s_{\mu.\nu.E}$ of the foundation μ of the first loading part

$$q_{0.\nu} = P_{E.\nu} / A_\nu \cdot B_\nu \qquad [kN/m^2]$$

of the foundation ν is analogous

$$s_{\mu.\nu.E} = q'_{0.\nu} \cdot B_\nu \sum_{i=1}^{l} \frac{\Delta f_{s.\mu.\nu.i}}{E_{s.\mu.i}} \qquad [m].$$

So we receive the following settlement influence coefficients

$$c_{\mu.\nu.E} = \frac{s_{\mu.\nu.E}}{q'_{0.\nu}} \qquad [m^3/kN]$$

$$c_{\mu.\nu.W} = \frac{s_{\mu.\nu.\nu}}{q_{\nu.\nu}} \qquad [m^3/kN].$$

So the soil pressure q_o under foundation ν is composed by 3 parts (with $q_{W.\nu}$ = uplift)

$$q_{0.\nu} = q_{\nu.\nu} + q'_{0.\nu} + q_{w.\nu} \qquad [kN/m^2].$$

It is the special intention of my paper to show that this method can be used for calculating the interaction between structure (with foundation) and soil for irregular layered soil and for two- or three-dimensional structures of any form.

The method of calculating settlements is published in detail in the first volume of my book "Berechnung von Flächengründungen" (see reference list in my paper). We have published comments of the computer program in a special manual ELPLA.

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H.-W. Koreck (F.R.G.)

Load Tests on 5 Large Diameter Bored Piles in Clay

At the time of submission of paper 2/42 for the Conference (Vol. I, page 571 ff.) the results of vertical soil movements at different depths and distances with respect to the test piles were not complete. As supplement to the original paper the results are being presented herein.

Besides pile 2 and 3 the soil was instrumented as described in the paper. The heads of some bars are shown in a photograph (Fig. 1).

Some of the test results of the movements near pile 3 are shown in Fig. 2. The curves of the observation points (depth of 8.5 and 13.0 meters below surface) show that for a settlement of the pile head of approximately 17 mm the downward soil movement extends to a distance of approximately 2 diameters from the pile axis. Beyond this distance the soil moves upwards. When the pile settlement was more than 17 mm the direction of the soil movement adjoining the pile changes and only upward movements could be measured from then on. Even at a distance of 4.5 times the diameter from the axis of the pile, that is a distance of approximately 5.8 meters, the movement could still be observed. The movement of the soil in the upper level (1 meter below surface) is smaller than in the deeper levels and a clear upward movement is only to be seen in a distance of between 2 and 4 times the diameter from the pile axis. This behaviour must probably depend on the annular space around the pile filled with bentonite slurry up to a depth of 5.7 meters.

In his preliminary report the General Reporter pointed out that the shaft friction of the under-

Fig. 1 Photo showing heads of extensometer bars

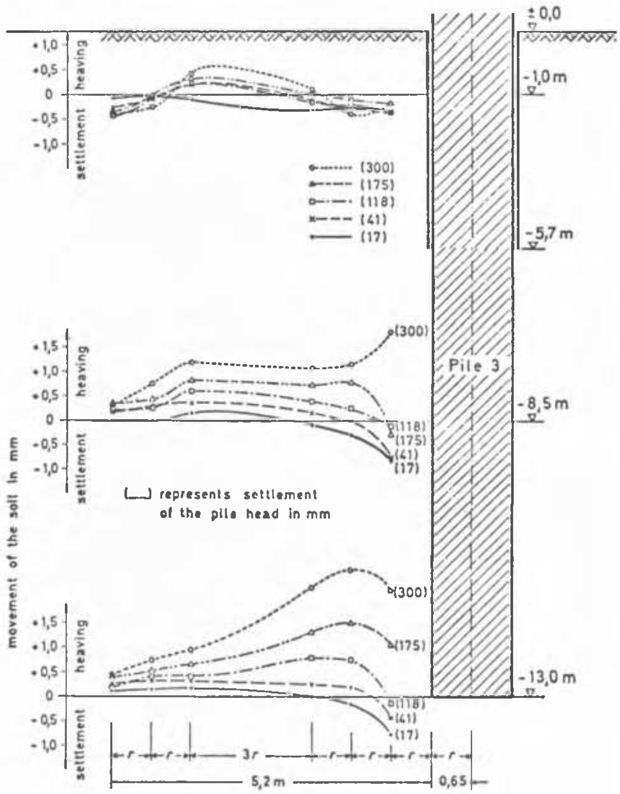

Fig. 2 Readings of soil movement

reamed piles was relatively high compared to the values of the straight-shafted piles, which in his view was due to construction procedure.

In reply the author wants to refer to Fig. 13 in the above mentioned paper which shows no obvious differences in shaft friction for straight-shafted as well as underreamed piles for settlements up to 20 mm. In the authors opinion this fact should be regarded as realistic for greater settlements as well. Possibly the difference in shaft friction between the two types of piles lies in the fact that the shaft friction of underreamed piles increases with settlement at a smaller rate than for straight-shafted piles and has its maximum value at a larger settlement.

A faulty construction could have affected the result of pile 2. The sealing of the annular space of this pile was leaky and concrete crept into the free space. With the help of numerous small borings, one besides another, the space was cleaned again. It is possible that there was still a contact between the two surfaces resulting in high shaft friction values of pile 2. Therefore the results of pile 2 are to be taken cautiously. As regards pile 4 the results are to be taken as correct up to a settlement of 5 cm, beyond which a few difficulties with the loading system were encountered.

G.G. Meyerhof (Canada)

Since the paper on the bearing capacity of piles in layered soils (Meyerhof and Valsangkar, 1977) was submitted to this conference, further tests on the punching resistance of large model footings and piles in a sand layer overlying a weak stratum have been made and some field data have been analysed. It has been found that due to scale and compressibility effects the shape factors for the punching resistance of large piles approach unity and the corresponding punching coefficients K_{ps}

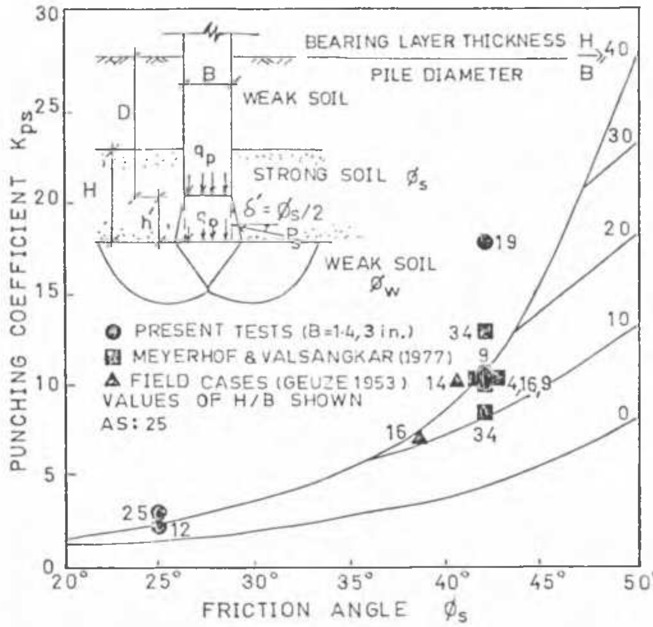

Fig. 1. Punching Coefficient for Short Piles in Cohesionless Layered Soil

in Eq. (9) are much smaller than those given in Fig. 8 of the paper. The results of this additional analysis of the punching resistance of piles in cohesionless soil above the critical depth are shown in Fig. 1, and these revised punching coefficients are suggested for estimating the punching resistance of short piles in practice.

C. Veder (Austria) and W. Prodinger (Austria)

Field Measurement of Skin Friction and Base Resistance

The International Administration and Conference Center of the United Nations in Vienna is a tipical example when examining the bearing behaviour of elements of deep foundations. The towers, which have the same star like layout, are arranged around a centrally located conference hall.

The subsoil consists of fill and a sandgravel formation near the surface and the underlying of alternating layers of Viennese tegel (clay-silt) and middle sand. The thickness as well as the properties of these layers varies. (Fig.1)

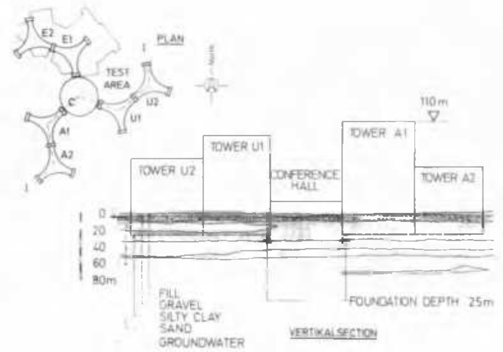

Fig. 1 Scheme of Soil Strata

Viennese tegel has an average value of 0.33 MN/m^2 in the unconfined compression test; the angle of internal friction ϕ ranges between 16^o and 35^o. The middle sand is mainly medium densely packed. The ground water level is 4 m under the surface. The foundations of these buildings had to be designed taking into account the extreme sensitivity toward settlements or differential settlements. The limit values for allowable settlements are 50 mm for total settlement and 20 mm for differential settlement. Because of the subsoil conditions, this requirement necessitated the transmission of loads into the subsoil by means of a deep foundation.

From July to August 1972 load bearing tests on diaphragm panels (50/150 cm) and bored piles (Ø 90 cm) were carried out on the construction site. A comparison of the load bearing behaviour of diaphragm walls and bored piles showed both systems to be equally good.

The foundations of the towers were planned according to the expert opinions of Professor Borowicka, Vienna, and Professor Veder, Graz. The foundation elements are all approx. 25 m deep and arranged according to loads. The loads were distributed by means of a 3 to 4 meter thick head plate. The deep foundation was constructed uniformly and consists of groups of diaphragm wall panels (Fig.2)

In 1973 construction of the diaphragm walls was begun. The allowable limit value for working load was set at 3000 to 4000 kN/m of diaphragm wall.

In order to more accurately determine the load bearing benaviour of diaphragm wall groups, pressure gauges were built into several diaphragm wall elements of the foundations of Tower A1. The gauges were instalied at the panel's base and 2.5 m under the head plate. The pressure gauges are Glötzl type and hydraulically operated.

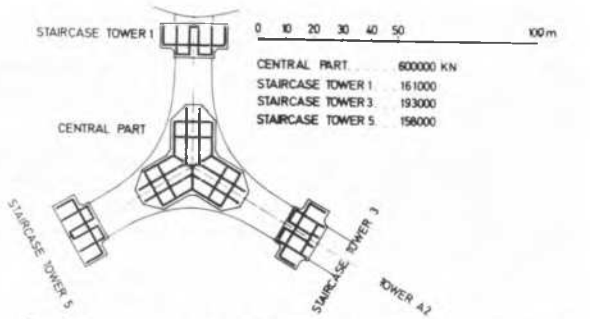

Fig. 2 Cross Section and Load Distribution Tower A1

Pressure measurements were begun in 1974 when the head plate was completed and have not been concluded. The connection between loading and settlement was determined exactly by levelling for all buildings. In case the allowable differential settlement had been exceeded, it would have been possible to correct the settlement by lifting the construction parts using hydraulic jacks.

Basically similar values have been obtained from the measurements made at Tower A1 for the diaphragm wall group of the central core and for that of the stair towers (Fig.3).

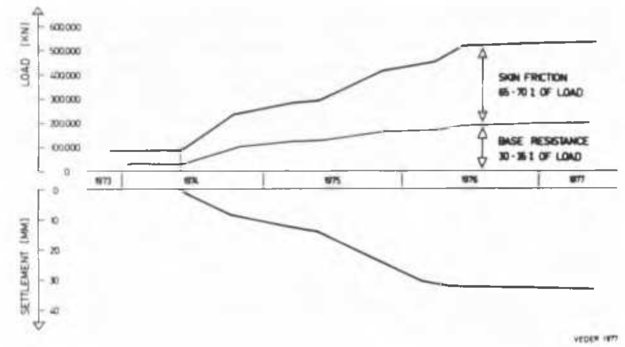

Fig. 3 Load Settlement-Diagram, Tower A 1
-Portion of Skin Friction and Base Resistence-

Base resistance rose proportionally with increasing load; its portion of load transference varied between 30% and 35%; thus skin friction amounts to 65% to 70%. Total settlement of the individual foundation elements is clearly below the allowable limit value of 50 mm. In only one instance were allowable differential settlements minimally exceeded.

A number of informative tests are still being carried out, i.e. the long- term effect of such foundations, the influence of a rigid head plate, the effect of layered foundation soil on skin friction. Long-term loading will probably not cause major changes in base resistance and skin friction. The direct load transference from the head plate into the subsoil is probably less than 10% of the total load.

It is planned that by 1978 research will have been completed and that in the following year significant results will be discussed in a dissertation by W. Prodinger.