IX ICSMFE, Tokyo 1977, pp.364-367.

60.2

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.

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.

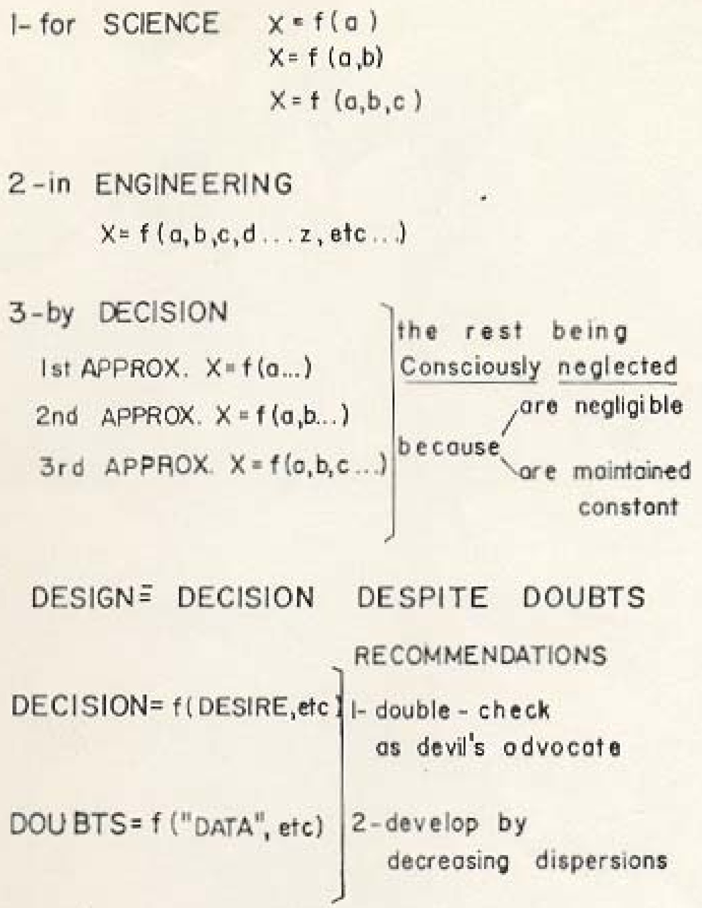


Fig. 1

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:

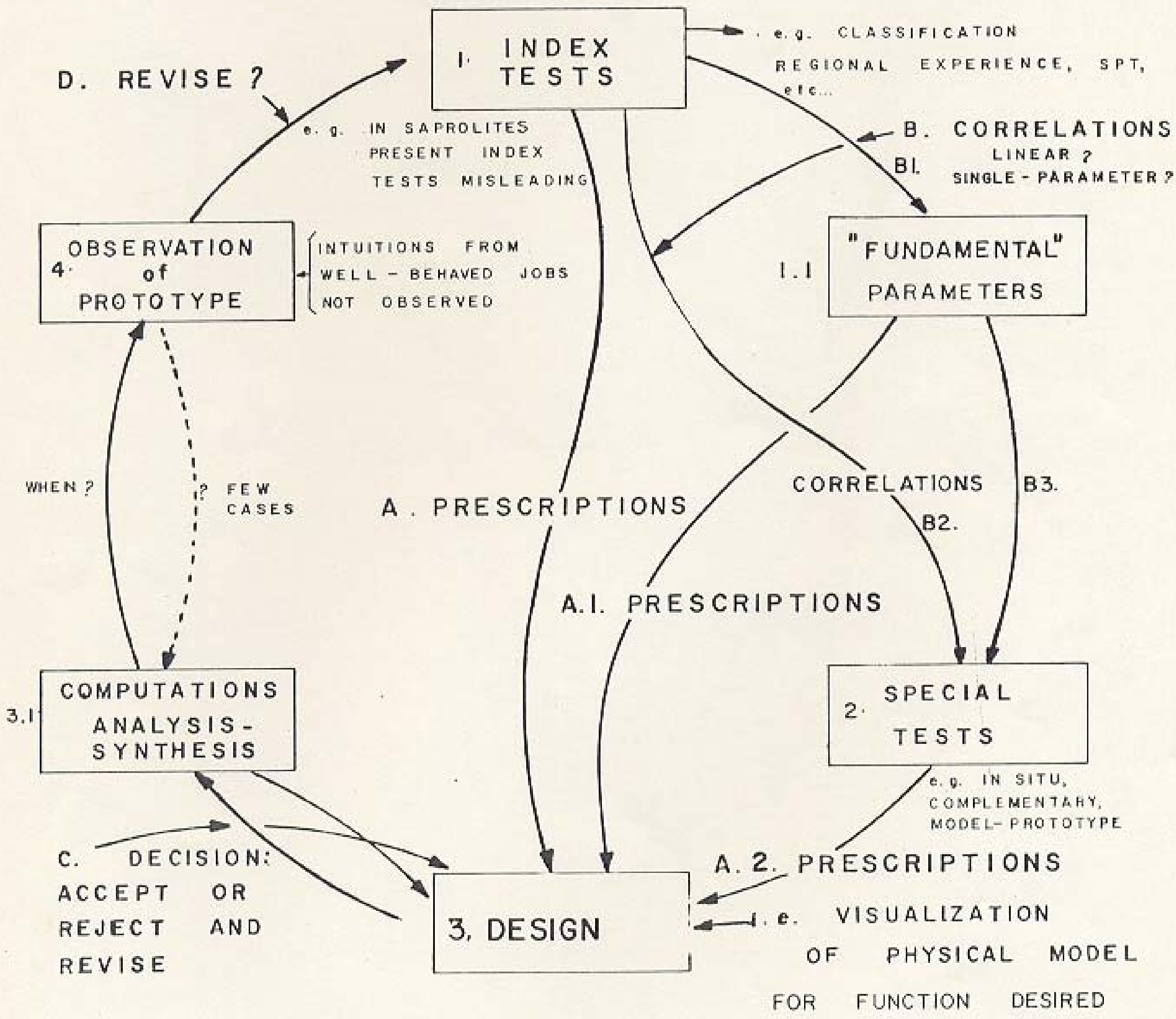


Fig. 2

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

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

$$\text{and}\begin{cases} c/p_c \simeq 0{,}115 + 0{,}00343\ PI \quad \pm ?\% \\ \therefore \text{from } c \simeq f(SPT) \text{ can get } p_c \text{ and OCR} \end{cases}$$

b-) FOR VERY ROUGH INDICATIONS ON STRENGTH

CLAYS $\varphi = 0° : c \approx SPT/8 \pm ?$ kg/cm^2

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

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

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

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

$k_s \approx 3\ SPT \pm 60\%$ t/m^2/cm

CLAYEY SANDS SUBMERGED $3 < SPT < 13$

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

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

also q_c of CPT $\approx (4-6)\,SPT$ kg/cm^2

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

B.3-ANY IN USE ?

2-PRESCRIPTION A

(SÃO PAULO, ROUTINE CONCRETE BUILDINGS, etc)

2.1- 1st STEP FOOTING HYPOTHESIS

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

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

e.g. EMPIRICAL $\dfrac{SPT}{5}$ or $\sqrt{SPT} - 1 \lessapprox \sigma_{all}$ kg/cm^2 $\lessapprox \dfrac{SPT}{3}$

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

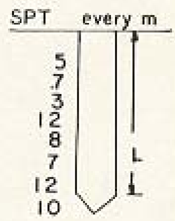
3-PRESCRIPTION A.I.

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/cm^2



EXECUTION EFFECTS ?

Fig. 3

IX ICSMFE, Tokyo 1977, pp. 369, 381, 389.

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)

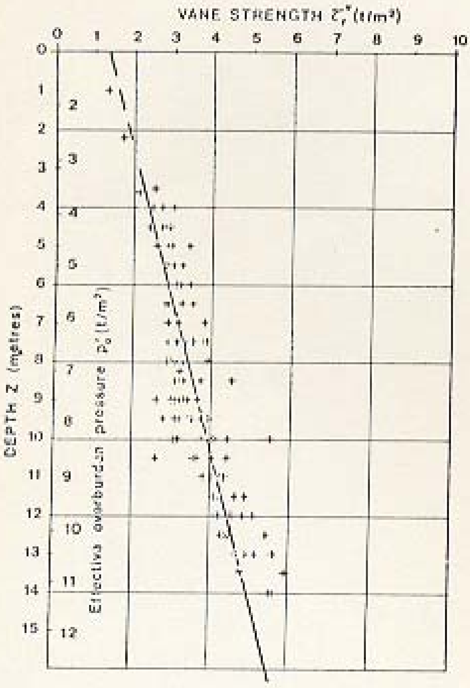


Fig. 3 Composite logs of vane borings at sta 52 + 70, Kimola canal, Finland (after Kankare, 1969)

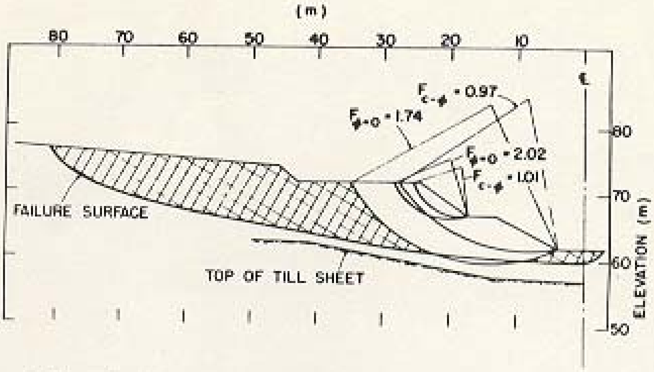


Fig. 4 Cross-section of November 3, 1965 failure at station 52 + 70 on the lower canal

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.

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.

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:

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

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

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-