Discussion report
Resumé des discussions

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SCOPE

General Introduction

The discussion session was conducted according to the following schedule:

General Introduction

Theme A: Foundations
- Introduction by discussion leader
- Guest speech on piles by Dr. B. Simpson
- Guest speech on shallow footings by Mr. S. Amar
- Floor discussion on these 3 presentations
- Short presentations from the floor
- Floor discussion on these presentations

Theme B: Excavations
- Introduction by discussion leader
- Guest speech on hydraulic seals by Dr. Schulz
- Guest speech on wall supported excavations and philosophical thoughts by Dr. Creed.
- Floor discussion on these 3 presentations
- Short presentations from the floor
- Floor discussion on these presentations

General Conclusions

The written report of the discussion session reflects this structure. The introductions by the discussion leader and the guest speeches have been extensively covered. Parts of the floor discussions have been summarized. Some short presentations have been transferred to the individual contributions section of the written discussion.

GENERAL INTRODUCTION

A discussion leader should complete 3 main tasks:

one, from experience and correspondence with other geotechnical engineers, determine the needs of knowledge;

two, from the current literature and textbooks, isolate issues which are not uniformly described;

three, from reading the papers submitted to the conference, stimulate a discussion on the topics addressed by several papers.

It would be ideal, of course, that the answers to the identified gaps in our knowledge would come from the contributions to this conference. This ideal goal is not within reach of this discussion session because of the many unknown aspects of the role of water in foundations and excavations with respect to the limited time we were allotted. That is why it was felt it preferable to concentrate on two main themes:

Theme A: Role of water in the performance of foundations; a look at both shallow and deep foundations.

Theme B: Role of water in the stability of the bottom of excavations; principally concerning sealing problems and uplift problems.

Both main themes were structured according to 3 steps:

one, a brief review of some basics, definition of the topics of discussion and some of the questions raised.

two, a limited number of invited speakers presented their views on issues which were recognized as not being straight forward.

three, a discussion from the floor in which everyone was cordially invited to participate.

It is really no secret that the medium we are dealing with consists of 3 phases: solid, liquid, and gas.

The presence of water has a major role in the behavior (deformation + strength) of soil under stress and this fact was first expressed by Terzaghi via the concept of effective stress which is summarized by the equation:

$$\sigma' = \sigma - u$$

σ': effective stress

σ : total stress

u: pore pressure, i.e. the stress (above atmospheric pressure) in the water in the voids of a fully saturated soil.

Since it is the effective stress that controls the behavior of soil under the loading conditions corresponding to foundation and excavation works, it is quite clear that a proper design has to ascertain rather precisely the pore pressure for drained and undrained conditions.

Since the effective stress is the normal stress transmitted by inter-particle contacts, in the case of unsaturated soils one has to separate the pore water pressure, u_w from the pore air pressure u_a. This was expressed by Bishop et al. in 1955 by the following equation:

$$\sigma' = \sigma - \left[u_a - \chi\left(u_a - u_w\right)\right]$$

with x being the ratio of the area onto which the pore water pressure acts.

If

$$S_r = 1, \text{then } x = 1 \text{ and } \sigma' = \sigma - u_w$$
$$S_r = 0, \text{then } x = 0 \text{ and } \sigma' = \sigma - u_a$$

In the general case of an unsaturated soil, the pore water pressure is smaller than the air pressure because of surface tension. This surface tension is also responsible for the capillary phenomena observed in the saturated and unsaturated portions of soil. One therefore arrives at the conclusion that if problems are treated without considering the effects of non-saturation, the initial effective stresses will be underestimated and the analysis will generally be on the safe side.

Whereas the mechanics of saturated soil are well understood, it appears that the behavior of unsaturated soils is not very closely mastered:
What about deformation properties?
What about strength?
What about permeability?
What about water content with reference to the position from the phreatic surface?

All those questions led me to make an inquiry about those problems and the survey that I conducted amongst the authors who submitted papers relevant to this discussion was very fruitful. Topics that were recognized as controversial are summarized in table 1.

FOUNDATIONS		
Shallow	-	stability endangered by water
Deep	-	decrease of bearing capacity of piles especially of bored piles
	-	effect of increasing pore pressures on the performance of piles
	-	drainage and pore pressure dissipation around piles
Structure	-	deformation of buildings and structures
	-	relation between building settlement (rate) and damage
Special Problems		
- Movement of ground water in clays		
- Gypsum soil		
- Reliability of prediction of consolidation based on pore pressure measurements		

Table 1: Controversial topics on foundations.

For the excavations, there was also quite a list of suggested topics for discussion, as shown in table 2. The role of water in the state of stress, the deformation and factor of safety of retaining structures is quite complex and deserves our attention. As the stability of an excavation is linked in practice with the construction efficiency of sealing problems, this important topic was discussed.

EXCAVATIONS
Slope stability, heave and uplift
- State of stress at the bottom of an excavation
- Hydraulic uplift in less permeable soils
- Hydraulic fracturing, local gradients at the foot of sheet piles
- Filling up of water in fissured soils on top of slopes
Lateral pressures and deformations of retaining structures
- Reduction of locked-in stresses behind retaining walls
Dewatering and recharging
- Measurement of head loss in well filters
- Leakage identification of big foundation pits
- Conditions for optimal sealing
- Determination of coefficients of permeability in layered soils
- Technology of artesian pressure relieving systems

Table 2: Controversial topics on excavations.

THEME A: FOUNDATIONS

Introduction

Theme A of our discussion session is the role of water in the performance of bearing capacity and settlements of foundations.

As deep foundations have a higher chance to be concerned with saturated soils than shallow foundations, and that it appears that the mechanics of saturated soils are better known, I preferred to start with this subject.

At this conference, there were very few references to the water problems associated with deep foundations. Does it mean that we already know enough? I am not sure we do and some of the basic questions I thought should be answered are listed below:

1) How should the soil investigation be carried out in order to obtain the properties relevant to the design of a deep foundation in contact with a fluctuating level?

2) What happens to the pore pressure distribution during installation of the piles and how fast does it return to a stationary condition? How does it affect the bearing capacity?

3) Should a rise or depression of the water table occur, how can we quantify its effect in terms of deformation, failure pattern, and failure load? Can we separate the influences on the shaft from the one on the base bearing capacity? In the case of compressible layers, what happens to the negative skin friction?

4) How do the piles interact with the unsaturated portion of the surrounding soil?

Dr. Simpson, the first guest speaker, tackled some of these issues in his paper submitted to session 3 of this conference.

The problem of shallow foundations is much more popular: at this conference, three papers specifically addressed the problem of bearing capacity and three other ones the problem of settlements of shallow foundations. These topics are more exciting probably because of the more extensive influence of the presence of water on the behavior of footings as well as because of the magic associated with capillary phenomena.

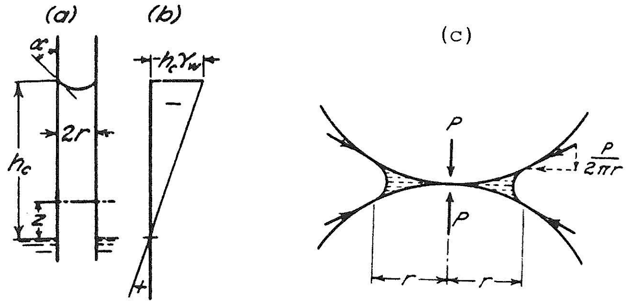


Figure 1 (a): Rise in capillary tube
(b): State of stress of water in capillary tube
(c): Forces produced by contact moisture
(From K. Terzaghi and R. Peck, 1967)

I would like to demistify this. There is much evidence that a liquid surface can resist traction because of the attraction of molecules at that thin interface layer. This resistance to the tensile forces is measured by surface tension, a constant property of the water-gas interface at a given temperature. An evidence of this sought equilibrium in tension is the fact that water rises and remains above the line of atmospheric pressure in a very fine bore, or capillary tube, as shown in fig. 1a. The height of rise h_c in a capillary tube is expressed by the following formula:

$$h_c = \frac{2T_s}{r\gamma_w}\cos\alpha$$

with T_s: surface tension of the liquid-gas interface
r: the radius

γ_w: the unit weight of the liquid

α: the contact angle made between the liquid and the tube (cf. fig. 1(a))

Because the voids left by the solid phase are generally sufficiently small, capillarity enables a dry soil to draw water above the phreatic line and also enables a draining soil mass to retain water above the phreatic line. Because of the soil voids have a given size distribution, there is no unique capillary head, nor are the phenomena reversible.

Above the phreatic line, one can find the following situations (cf. table 3):

- saturated soil with water under hydrostatic tension
- unsaturated soil but continuous water links: water under hydrostatic tension
- areas where there is vapor.

The essential consequences of all of this is that the pore water pressure can become negative (with reference to the atmospheric pressure). This resulting effective stress increment increases linearly with the height above the water table for the continuous water network (cf. fig. 1(b)), up to somewhere between a few cm for gravel to a few meters for silts. This effective stress induces what is called an apparent cohesion, which is very helpful for short term excavations in granular soils.

Therefore, we are faced with the determination of the behavior of shallow footings resting on a medium whose properties locally depend on the proximity to the phreatic line. These problems have been looked at by contributors to this session and figure 2a shows the effects of capillary tension as represented by an apparent unit weight. From this contribution (cf. fig. 2b), you can see that plasticity theorems can be used to assess the bearing capacity of shallow footings taking into account the heterogeneity between the two media: the unsaturated phase and the saturated phase. Also, you can use the methods of characteristics to take into account this heterogeneity (cf. fig. 3). For other assessments, authors have introduced variable deformation characteristics, so that they can assess the deformation properties linked to the variations of the water table. These are more or less theoretical approaches, but there are also contributions which stem from experiments, some of them being carried in the laboratory. Figure 4 shows some typical loading curves obtained from model tests of shallow footings resting on unsaturated sand. Another approach has been taken using full scale tests and Mr. Amar, our second guest speaker, from France, reported on some of the works that have been done at the French Laboratory of Ponts et Chaussees.

Guest speech on Piles, by Dr. B. Simpson

Mr. Brian Simpson graduated from Cambridge University in 1968 and received his Ph.D. from the same university in 1973. His work was on the finite elements and the use of the finite elements in geotechnics. In 1971, he joined Ove Arup & Partners in London and he was made Director of this company in 1985. He works mainly in the Middle East and the United Kingdom on geotechnical design, especially problems of soil-structure interaction. He is the British Geotechnical Society's representative on the Committee drafting a model for Eurocode EC7.

Dr. Brian Simpson proposed to illustrate three situations in which construction or performance of piles is affected by ground water. Firstly, the common situation in which the presence of water is adverse because it softens the soil. Then secondly, a situation in which the presence of water is found to be beneficial during the construction of the piles, and thirdly a situation related to the potential influence of rising water pressures.

In stiff clays, it is usually considered that the appearance of free water in the shaft of a bored pile is a danger signal. If water is available in the clay during construction of the pile, the stiff clay may swell and soften on the surface of the shaft.

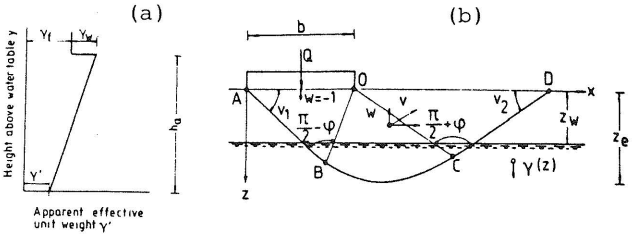


Figure 2a: Apparent unit weight γ'
in moisture zone assuming α linear in y.
Figure 2b: Kinematically admissible rupture figure.
(From B. Hansen et al., 1987)

CATEGORIE	FACTEUR PRINCIPAL REGISSANT SON COMPORTEMENT	PRECEDE D'EXTRACTION	LOCALISATION PAR RAPPORT AUX GRAINS	LOCALISATION DANS LE PROFIL AQUIFERE	DIAGRAMME DES CONTRAINTES VERTICALES
eau de constitution	forces moléculaires	décomposition physico-chimique des minéraux (calcination)			σ : contrainte totale u: pression relative interstitielle σ': contrainte effective ($\sigma' = \sigma - u$)
eau de rétention -hygroscopique - pelliculaire	adsorption	Centrifugation	pelliculaire hygroscopique	zone aérée sèche (eaux liées)	
vapeur d'eau	tension de vapeur	séchage	vapeur d'eau	surface capillaire visible zone aérée humide	
eau capillaire - isolée	tension de membrane à la surface des ménisques	centrifugation	eau gravifique liée eau capillaire isolée	surface de saturation	Succion (u négatif)
-continue	dimensions et formes des particules de sol	gravité		zone capillaire saturée surface libre de la nappe frange capillaire hauteur piezomé-trique	u = o
eau gravifique	gravité	gravité		zone phréatique (eaux libres)	σ' u σ

Table 3: L'eau dans le sol (A. Holeyman, 1973).

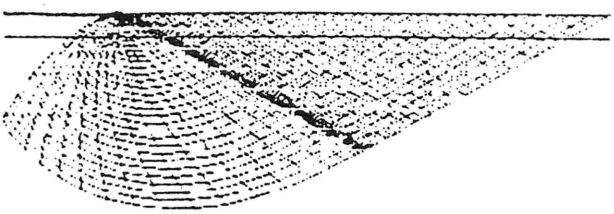


Figure 3: Characteristics mesh for rough strip footing. (From P. De Simone and R. Zurlo, 1987)

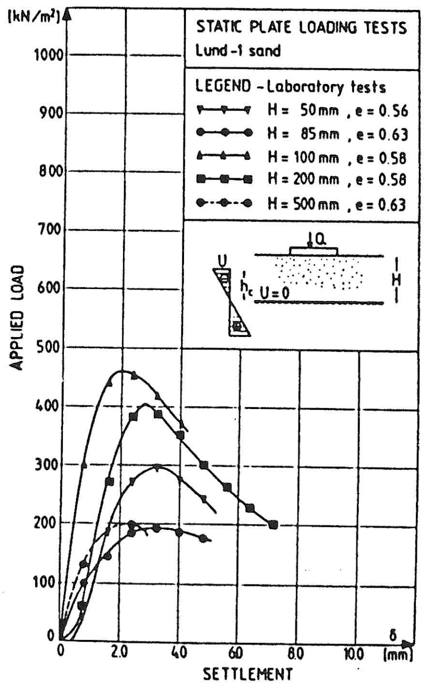


Figure 4: Load-settlement curves for tests on sand. (From J. O. Steensen-Bach, et al. 1987)

For example, Meyerhof and Murdock (1953) observed increases in the water content of the clay within about 50 mm of the shafts of bored piles. It is not entirely clear whether the effect would remain in the long term and it is also possible that the water that was softening the clay may have been derived from the wet concrete itself, rather than the ground. But the important fact is that the availability of free water of some sort had caused an increase in water content of the soil and therefore a reduction in shear strength adjacent to the shaft. This is a simple adverse effect that we are all fairly familiar with in thinking about piling.

A contrasting effect has been observed when forming bored piles in a mudstone and siltstone. A 2.5 m deep socket was required into unweathered rock. The piles were formed by augering and the base of the pile was cleaned with a cleaning bucket. It was intended that all piles would be inspected by descending in a safety cage. However, when the preliminary trial pile was built, it was found to be very wet, and consequently, many of the piles were only inspected by shining a lamp down from the ground level. If the base of the pile was wet, extra care was taken with the cleaning bucket to ensure a sound base. But when some of the working piles were tested, they were unable to carry even the intended working load without gross movement. Clearly, there was a serious problem in the construction of the piles. Cores were obtained through the piles and revealed soft, remolded material beneath the base of the concrete shaft. This could have a thickness of certainly some tens of millimeters. A shaft was also formed alongside one of the piles and a thick zone of softened mudstone was found down the side of the concrete pile shaft. The important conclusion from this investigation was that it was piles which had been formed in dry conditions with no visible water in that the shaft which had failed. The piles which had wet shafts had not failed. In both cases, the mudstone was probably remolded by the boring process. However, where there was plenty of water in the boring, the shaft was washed clean and the remolded material was removed. The solution adopted was to add large quantities of water during piling so as to wash away the remolded material. The water and debris were bailed out before concreting. This example illustrated the fact that water in constructing piles may not always be an adverse action. It may be a very helpful element in some circumstances.

The third situation considered involved the increasing water pressure adjacent to existing piles. Figure 5 shows a north-south section across the London basin. In many areas, the London clay overlies the aquifer in London which consists of chalk and sands. The original water level in the aquifer has been reduced by pumping from the aquifer during the last two centuries. So in 1965, there was about a 60 meter depression of the piezometric level in the aquifer. However, in the last forty years, abstraction of water has been reduced and the water levels are rising at rates of up to 1 m per year.

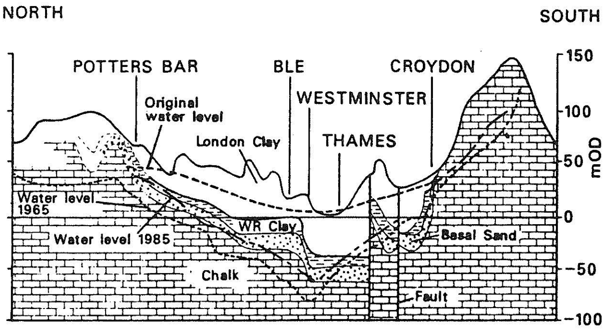


Figure 5: North-South section through the London basin.

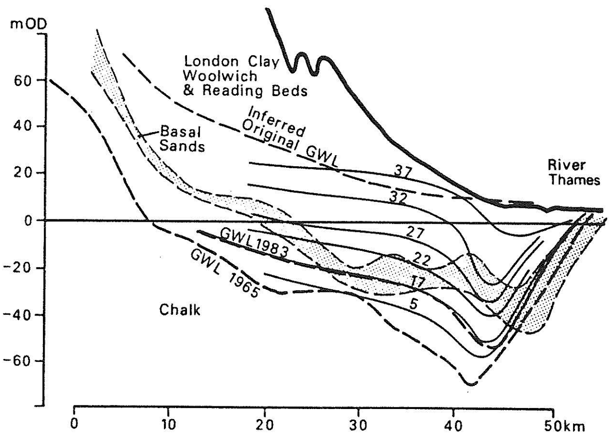


Figure 6: Rise in piezometric level in the London basin.

The rise from 1965 to 1985 can be seen in Figure 6. The present depression, and therefore, the potential rise of the piezometric level in the aquifer is up to about 60 m in parts of London. In common with other major structures in London, the new British Library basement was initially designed on the assumption that the water levels would not rise (cf. fig. 7). However It is now considered possible that in the future, water levels could return towards their initial level. At this site, that would be a level of +10 m O.D., so the piezometric level at the bases of the piles could potentially rise to that level. In these circumstances, the piles beneath the basement would be unstable. There would be about 26 m of water head and only about 12 m of clay remaining beneath the basement, clearly an unstable situation.

The solution that has been adopted has been to double the base areas of the piles and to provide pressure relief wells to prevent the water level rising above -6 m O.D., which is the level of the bottom of the basement. So simple gravity pressure relief wells have been introduced for this purpose. Dr. Simpson pointed out another problem which does not directly relate to piles, but to the effects of water pressure on the design of the toe of the remaining wall. With the possibility of water pressure rising and the clay softening, that had to introduce a thickening of the edge slabs so as to provide additional passive restraint to the toe of the wall.

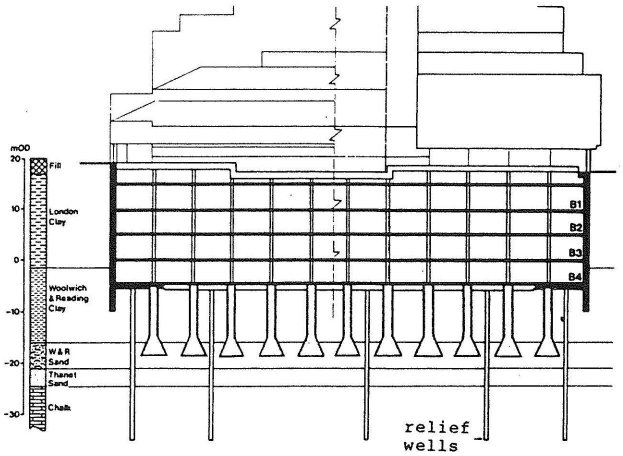


Figure 7: British Library foundation scheme.

In London there are many examples of piles in soft clay or granular soils in which the water pressure has been reduced by under-drainage.

There is concern to know how these piles will perform if the water pressures are increased in the future, and this topic has been considered by Simpson et al. (1987) in a paper to this conference. Based on the work of Armishaw and Cox (1974), (also cf. individual written discussions) they have concluded that driven piles are unlikely to be affected significantly by raising ground water levels. Furthermore, bored piles in stiff clays will suffer only minor reductions in capacity for large increases in pore pressure in the adjacent soils. Exceptions could occur beneath deep basements however, where vertical effective stresses and hence bearing capacity pressure relief were provided.

Guest Speech on Shallow Footings by Mr. Amar

Mr. Samuel Amar is Assistant Head of the Geotechnical Division of Soil Mechanics of LCPC, Laboratoire Central des Ponts et Chaussees, in Paris, France. He is Assistant Professor of Soil Mechanics in the Ecole Nationale des Ponts et Chaussees in Paris. He is also a member of the Eurocode nr.7 Geotechnical Design. His main areas of interest are shallow foundations and deep foundations, in-situ testing, in particular self-boring pressuremeter and penetration testing and also reinforced earth.

Le travail présenté a été réalisé par trois chercheurs: MM. Baguelin, Canepa et S. Amar

L'influence des variations de la nappe sur la capacité portante ou le tassement des fondations superficielles a fait l'objet de nombreuses communications à cette conference. L'étude de cette influence a été abordée de plusieurs manières. Soit à partir de modèles théoriques, soit a partir d'essais en laboratoire ou en place sur modèles generalement reduits, soit à partir d'observations faites sur des ouvrages pendant et après leur construction. La démarche des auteurs a été differente. Ils ont etudié le comportement de fondations superficielles de grandes dimensions, d'un metre, reposant sur un sol réél homogène qui est baigné par une nappe dont les fluctuations sont importantes. Cette recherche entre dans un programme plus vaste sur les fondations superficielles qui est realisé par le Laboratorie des Ponts et Chaussées. La presentation s'est centrée sur les résultats obtenus sur le limon. La figure 8 présente le schema du dispositif expérimental utilisé pour les essais à court terme.

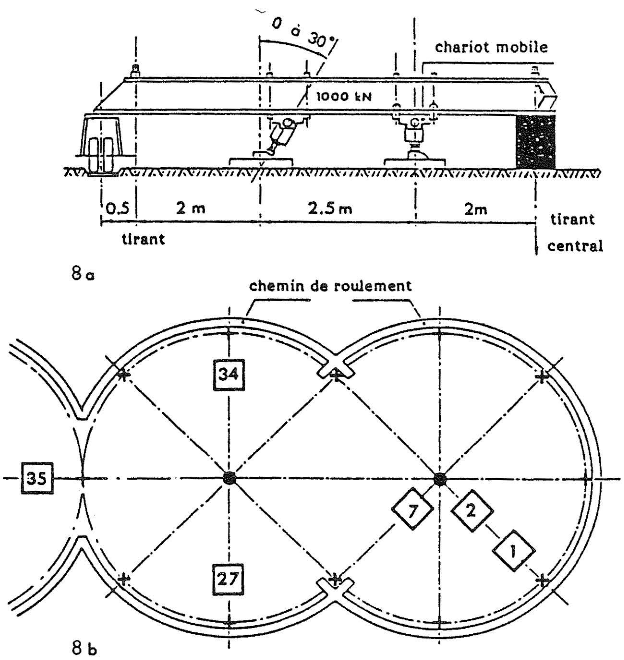


Figure 8: Dispositif expérimental pour essais de courte duree.

On appelle essais à court terme, des essais dont la durée de chaque palier de chargement est comprise entre 30 minutes et 1 heure. Il s'agit d'une poutre qui peut pivoter autour d'un axe. Elle est scellée à la periphérie par des tirants d'ancrage. Le massif de réaction a été conçu pour permettre l'étude de différentes sortes de sollicitations notemment une charge verticale centrée pour étudier l'effet de l'encastrement ou une charge inclinée. Pour les essais de longue durée, avec charge permanente - par longue duree, il faut entendre des charges qui peuvent rester plusieures années sur la fondation - l'équipe du L.C.P.C. a conçu un autre dispositif dont le schéma de principe est montré à la figure 9.

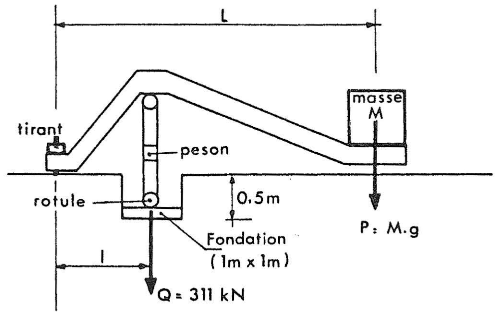


Figure 9: Schéma du dispositif expérimental pour essais de longue duree.

Le sol de fondation est constitué de 3.50 metres de limon surmontant 3 à 4 metres de sable argileux. Ce limon a fait l'objet de nombreux essais in-situ et de laboratoire. Une nappe phréatique fluctue dans cette formation de limon. En hiver, elle est au niveau 0, au niveau du terrain naturel, et en été, elle peut descendre jusqu' à 2.50 m - 3.00 m. La reconnaissance de sol a ete faite avec toute la panopile d'essais en laboratoire et en place. Le Tableau 4 présente les valeurs moyennes obtenues.

Limites d'Atterberg		γ_d	γ_s	Triaxial			Préssiomètre		Pénétromètre stat.	Pénétromètre Dyn.	S.P.T	Scissomètre de chantier	
W_l	I_p	kN/m^3	kN/m^3	Cu kPa	C' kPa	φ' °	P_l kPa	E_M kPa	q_c kPa	q_d kPa	N	τ pic kPa	τ palier kPa
38	14	16	26.5	38	12	32	500	6200	1300	1600	8	90	65

Tableau 4: Charactéristiques moyennes du limon.

Les essais de chargement ont été réalisés par paliers de 30 minutes. Les courbes de chargement pour une duree de charge de 10 ans sont obtenues par extrapolation des mesures de fluage. L'influence de la nappe sur le comportment de la fondation superficielle expérimentale est démontrée en comparant deux courbes de chargement. Une première a été obtenue lorsque le sol était saturé, et correspond donc à une nappe qui était à peu près au niveau du sol. Une seconde courbe a été obtenue lorsque le degré de saturation était de 75% et que la nappe se trouvait alors à 2.50 m au-dessous du niveau de la foundation. La charge limite de rupture a été conventionnellement prise egale à la charge correspondant à un tassement de 15% pour la courbe extrapolée a 10 ans. Mr. Amar explique que le rapport des deux charges limites pour les deux situations envisagées est de 1.25. Lorsque l'on a simplement une variation de 25% sur le degré de saturation, on peut avoir un tassement deux fois plus important. Lorsque l'on s'intéresse au fluage, donc lorsque l'on prend pour chaque palier de charge, la pente de ces droites de fluage, on se rend compte que la différence s'amplifie. Sous chaque palier de chargement, on a pris le coefficient de fluage lorsque le degré de saturation était égal à 1 et 0.75 et on en a fait le rapport. On voit que ce rapport vaut 4.3 lorsque la charge est de 100 kPa et qu' il va en diminuant avec la charge.

Mr. Amar conclut donc qu'une diminution moderée (25%) du degré de saturation entraîne une augmentation modérée de la charge limite (25%). Par contre les tassements sous la charge de service peuvent etre multipliés par deux tandis que le coefficient de fluage pour de faibles chages peut etre multiplié par 4 en comparant la situtíon où le sol est saturé par rapport à celle où le degre de saturation est de 75%.

Floor Discussion

A number of questions were raised about the guest presentations with particular reference to in-situ and laboratory tests which can be used to assess the sensitivity of bearing capacity and settlement to the change in water content. A lively discussion took place between Mr. Amar, Baguelin, Steenfelt, and Holeyman on the shape of the load-settlement curves in adimensional axes. The conclusion of this discussion was that loading curves for various tests on shallow footings can be unified in a s/s_o - $(Q/Q_1)^n$ plot.

THEME B: EXCAVATIONS

Introduction

In order to safely perform deep excavations in water bearing soils, a number of vital functional items have to be designed and in particular:

1) the dewatering system (and possibly the recharging system)
2) the retaining system
3) the cut-off system

The design of these 3 items is interdependent. The main goal of the procedure is to make sure that:

- no water is allowed in the excavation above excavation level, either by percolation, by leakage, or by piping,
- the bottom of the excavation is stable and in particular uplift forces due to water pressures must be mastered,
- the effective stress relief is kept to a minimum in order to avoid elastic heave, and a fortiori plastic heave,
- there should be a sufficient margin of safety against failure of the retaining system,
- lateral deflections of retaining elements stay within allowable limits.

In theory, if the initial ground water conditions and the subsoil hydraulic properties are sufficiently known, the pore water pressure distribution can be calculated taking into account the characteristics of the cut-off system and of the dewatering system.

With the newly established ground water regime, one can therefore look at the design criteria concerning the stability and deformations of both the bottom of the excavation and the retaining structure.

We have the tools to carry out this analysis but in practice, however, a number of problems arise due to:

- insufficient or improper characterization, in particular, failure to identify water bearing layers, water levels and permeability;
- but also very important inadequate construction activities, leading to the discrepancy between the real and the design situations, in particular with respect to sealing conditions.

At the conference, we heard about bad theories. No theory is really bad. What is bad is the application of any theory to a situation that does not represent what is being sought.

The constructional problem is limited to a system that is actively designed by geotechnical engineers, as opposed to the first class of problems, which are originally more nature-dependent.

It appears that the uplift problem is widely addressed by the papers introduced to this session. Following is the clarification of some of these definitions.

Figure 10, extracted from the paper by Ohta, shows the role of constructional practice of retaining structures in the analysis of ground water regime. Under discussion were piping failures observed in water retaining structures. They are usually due to scour or subsurface regressive erosion, leading to the more or less open connection between the upper water storage and a lower point. These will be defined as piping failures by internal erosion.

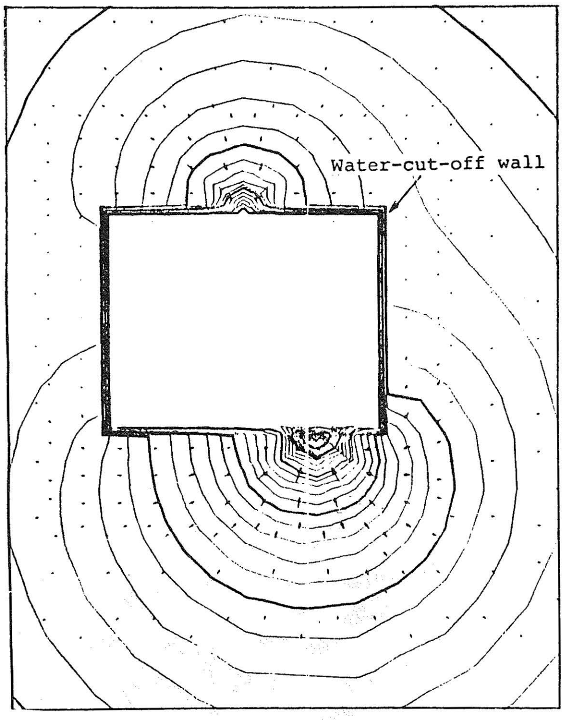


Figure 10: Distribution of the water head at the time of water spouting (from Ohta et al., 1987).

They do not affect excavations as "piping" due to heave, a situation that is encountered when the upward seepage forces start to equalize the effective weight of the soil mass adjoining the downstream toe of the retaining structure (cf. fig. 11).

Looking in more detail at what is happening to the soil, you can simulate this experiment in the laboratory (cf. fig. 12) and convince yourself of what is piping and when you visit some sites, you can see evidence of it as this beautiful boil occurring in sand shown in figure 13. These failures can be defined as piping failures by heave, and they affect mainly granular soils, i.e. those which can boil.

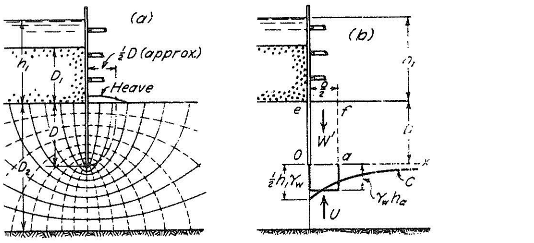


Figure 11: Analysis of piping condition (a) flow net, (b) forces acting on sand within zone of potential heave (from Terzaghi and Peck, 1967).

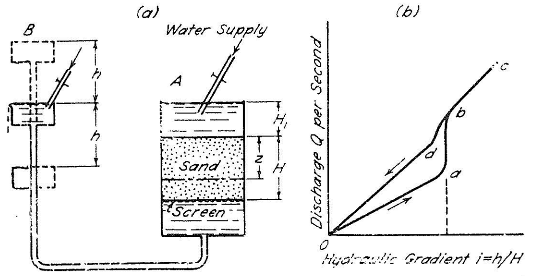


Figure 12: Analysis of boiling (a) laboratory apparatus, (b) relation between upward hydraulic gradient and discharge (from Terzaghi and Peck, 1967).

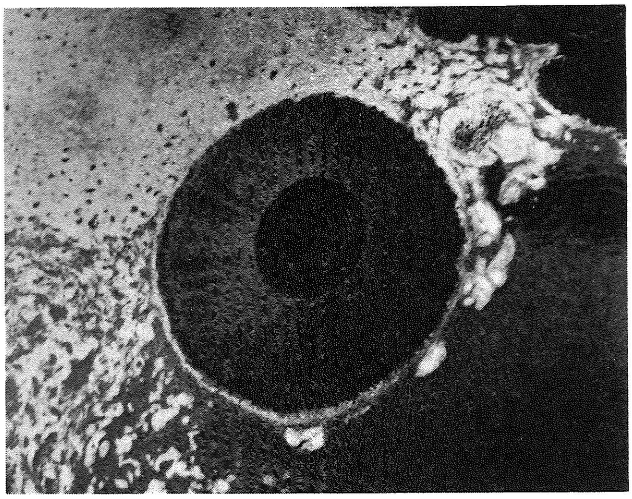


Figure 13: Photograph of a boil in sand.

Another type of failure can be encountered when a water-tight formation forms a plug between the walls of the excavation and that the water pressures are larger than the total pressures acting at the base of the impervious layer. In that case, a general bulging of the bottom of the excavation takes place, eventually cracks and releases the water and pressure flows.

Special note: Kastner et al., have comprehensively analyzed the interaction of water flow and stress conditions around a retaining structure (cf. fig. 14) in an excellent paper introduced in this session.

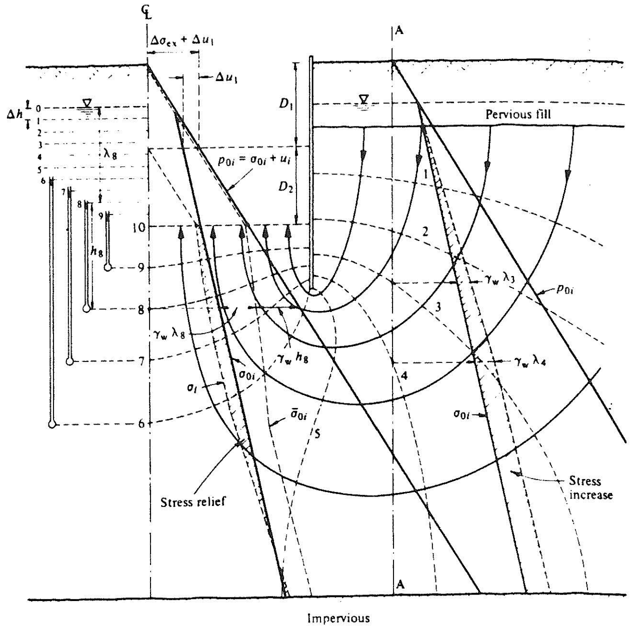


Figure 14: Schemas de rupture, (from Kastner, et al., 1987).

Slides were shown from the well-known book by L. Zeevaert, which illustrate the way to treat the problem of the interaction of water and excavation thus making sure that we know that the theoretical solutions are available. The flow pattern and its influence on the vertical stresses were pointed out (cf. fig. 15). Also explained were the role of the layering and its influence on the pore pressure distribution. There are also some approaches to determine the variation and horizontal stresses due to excavations and water draw-down.

As early as 1949, recharging was utilized for deep excavation in Mexico for the Latino-American Tower (cf. fig. 16, Zeevaert, 1972). These methods are not really new, but the technology has probably improved.

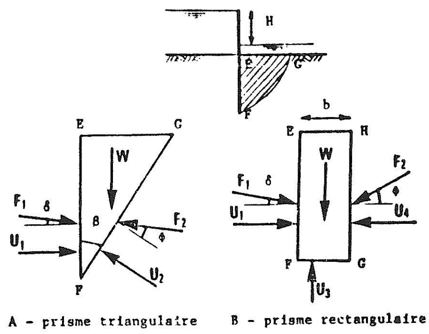


Figure 15: Change in effective stress due hydraulic conditions imposed by a deep sheet pile and a previous bottom stratum (from L. Zeevaert, 1972).

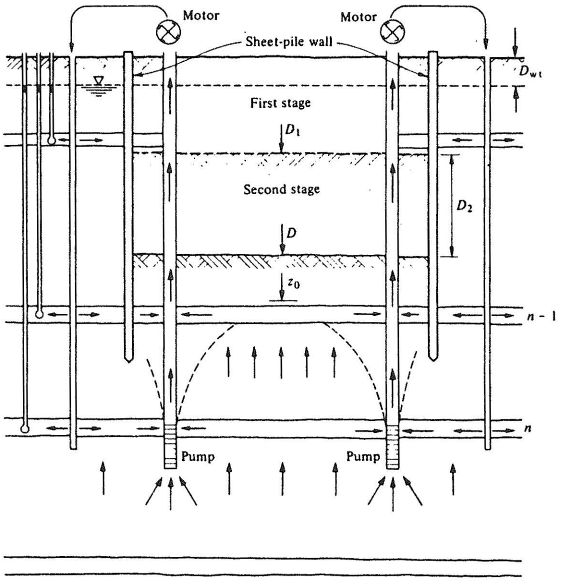


Figure 16: Dewatering and recharging for an excavation (from L. Zeevaert, 1972).

Guest speech on Sealing Conditions by Dr. Schulz

Dr. Schulz, who spoke about sealing problems, graduated in 1965 from the University of Stuttgart in Germany. He then worked on soil mechanics and hydropower plants at the University of Stuttgart and later on foundation engineering and soil mechanics. He obtained from the same institution his Doctoral degree in 1971. In 1973, he changed to the Federal Institute of Waterways Engineering at Karlsruhe, Department of Geotechnical Engineering and since 1976, he has been promoted head of the Department of Geotechnical Engineering. His topics of interest are over-consolidated soils, problems of muds in coastal estuaries, sealing conditions and laboratory testing.

Dr. Schulz observed that conditions for clay sealings at joints perhaps may not be a point of interest because clay has low permeability and good sealing properties per se, and needs no further investigation. Nevertheless, there exists a rather great number of case histories from which we learn that there is a difference of up to 2 to 3 orders of magnitude in permeability between the results of the laboratory testing and the in-situ permeability of sealing projects. In the institute where he works, they have been faced with the problem of the properties of clay linings for navigable waterways that had to be built under on-going traffic for the enlarging of these channels. During the investigations of the problem, the questions of leakage at joints, for instance the joint of vertical wall, arose and led to the assumptions below.

He started his discussion with the normal conditions at the sealing problems that are given in the sealing channels and stressed that the sealing condition he was proposing corresponds to a relative measure for the sealing in such a joint. The criteria is only to examine if there is a possibility to improve the tightness in clay joints.

The normal conditions in channels, are shown in Figure 17: the water depth is given by h_w, and the head difference with respect to the ground water table is denoted by Δh. This head difference is released across the lining of thickness d and is basically the net differential pressure acting in the joint a-a, assuming hydrostatic conditions prevail on both sides of the joint. On the other hand, the effective horizontal stress distribution in joint a-a results from the effective weight of the clay liner and from the seepage forces across the joint. These stresses are shown in Figure 17 by respectively horizontal and vertical shading lines.

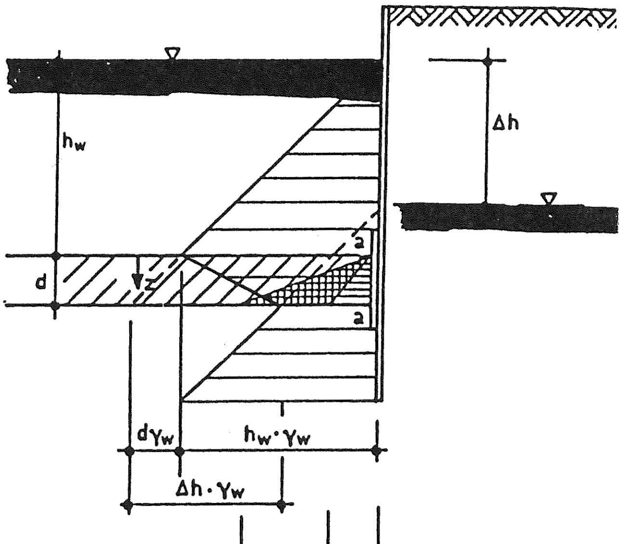


Figure 17: Stress conditions for channel seal.

To come to a mathematical formulation (fig. 18), one has to set up a formula for the water pressure in joint a-a and for the effective horizontal stress, using earth pressure relations in connection with the known effective vertical stress.

Steady-state conditions in a joint a-a:
Water pressure (assumed to be hydrostatic)

$$\sigma_w = \gamma_w . h_w + \gamma_w (1 - i) . z \quad \text{with} \quad i = \frac{\Delta h}{d}$$

Effective horizontal normal stress:

$$\sigma'_h = k . (\gamma d + i . \gamma_w) . z \quad \text{with} \quad K = \frac{\sigma'_h}{\sigma'_v}$$

<u>Sealing condition</u>:

$$\frac{\sigma'_h}{\sigma_w} > 1 \quad \text{at one certain depth z:}$$

$$\frac{K . \gamma_w (1 + i) z}{\gamma_w h_w + \gamma (1 - i) z} > 1$$

For simplicity:

$$\gamma' = \pm \gamma_w$$

Minimum value for K: z=d

$$\boxed{K > \frac{h_w / d + (1 - i)}{1 + i}}$$

Figure 18: Mathematical formulation of sealing condition.

The assumed sealing condition requires that the horizontal effective stress should be greater or at least equal to the water pressure to have optimum sealing conditions. As already pointed out, the proposed criterion is a relative one and if there is a possibility to influence the conditions, then one can look at this criterion. Now inserting the expressions for the water pressure and the horizontal effective stress in this sealing condition and using the simplification: $\gamma' = \pm\gamma_w$. Dr. Schulz arrived at a relation (cf. fig. 18) for a minimum earth pressure ratio K for z=d that will serve for comparison with the available earth pressure ratio K from the shear strength parameters of the used clay.

Figure 19 illustrates this finding: what can be at first surprising is that the K value may decrease with an increase of the hydraulic gradient. If you consider indeed that when the hydraulic gradient is increased the effective horizontal earth pressure in this joint is increased too, then this allowance can be understood. And it is quite clear that if the water depth is increased compared to the thickness of the lining, then you will have to increase the K value as is shown on the diagram. This condition for clay linings in channels can also be used for waste deposits.

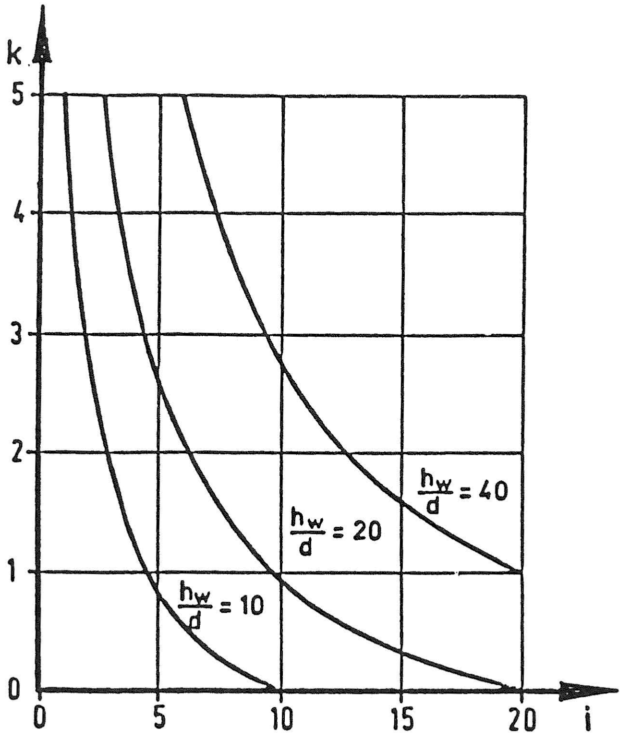
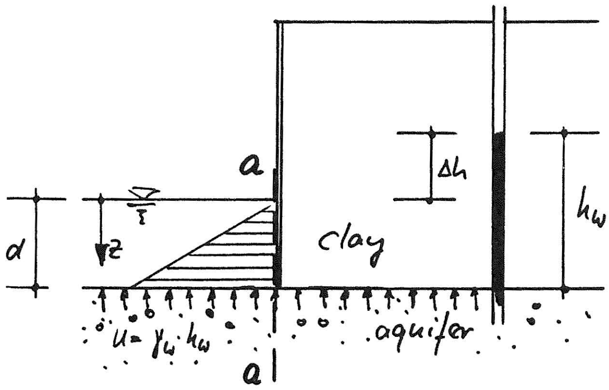


Figure 19: Lateral earth pressure coefficient necessary for a sealing condition (channel).

Dr. Schulz further used the same principle to examine whether a joint with a small opening width Δs, will close again by the imposed load if the active earth pressure is larger than the water pressure. It is assumed that this opening is still small enough to have hydrostatic pressure distribution within it. It is then possible to examine the sealing capability of clay layers or layers of low permeability behind retaining structures when dewatering is taking place beneath that layer. It is moreover possible to examine the condition of sealing between a vertical wall and the clayey soil at the bottom of an excavation. This condition should be met if one wants to reduce the amount of water seeping through this joint, which has to be pumped out of the excavation. As this case was of special interest to this session, Dr. Schulz applied the idea of optimal sealing conditions to this particular boundary condition.

For the simple case represented in fig. 20, an excavation down to a certain depth will leave in place a clay layer of remaining thickness d. The ground water table within the perimeter of this excavation shall be at the bottom of the excavation and beneath this clay layer will be an aquifer with water head h_w. The same considerations as before lead to the expression for the minimum value of K as given in fig. 20, taking into consideration that in this case the head difference reduces the bulk density of the soil. The same assumptions and simplifications will lead to an expression for K which is independent from the depth z and is a dimensionless expression. Therefore, one can obtain a very simple diagram (fig. 21) which shows the needed value of this dimensionless quantity for a given hydraulic gradient.



Water pressure in joint a-a:

$$\sigma_w = \gamma_w . h_w \frac{z}{d}$$

Effective horizontal stress:

$$\sigma'_h = K(\gamma' - i\gamma_w).z \quad \text{with} \quad i = \frac{\Delta h}{d}$$

and

$$K = \frac{\sigma'_h}{\sigma'_v}$$

$$\frac{\sigma'_h}{\sigma_w} > 1 \text{ for } z \neq d \text{ and } \gamma' = \pm\gamma_w :$$

$$\frac{K(1-i)}{h_w/d} > 1$$

$$\boxed{\frac{K}{h_w/d} > \frac{1}{1-i}}$$

Figure 20: Sealing condition for an excavation.

For very small gradients, the optimal sealing condition will be satisfied for $K = h_w/d$; for gradients moving towards a value of one, the case of uplift, this K-value will approach infinity.

From the stability analysis of the excavation, the mobilized K-value should be known and should provide the possibility for comparison. So the sealing effect can be checked. It should be mentioned that for a quick excavation the soil will swell and the suction that will be set up during the process may overrule the hydrostatic pressure conditions in the first phase of excavation. Therefore, the above results are only valid for steady-state conditions.

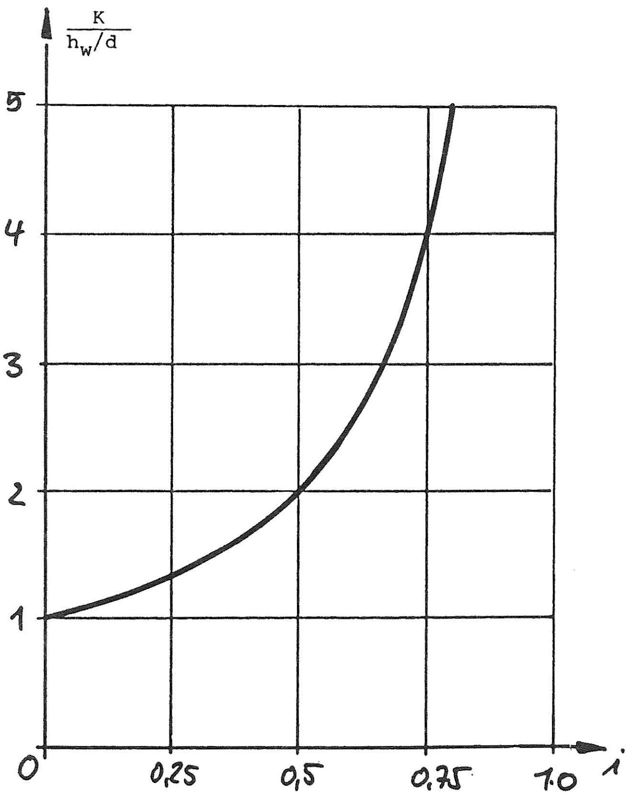


Figure 21: Lateral earth pressure coefficient necessary for a sealing condition in excavations.

Guest speech on Wall supported excavations and philosophical thoughts by Dr. Creed

Dr. Michael Creed graduated in 1973 from the Civil Engineering Department of University College, Cork, Ireland. Having worked with Professor Simons at the University of Surrey on the analysis of wall excavation behavior, he was awarded a Ph.D. by that institution in 1979. Since that time, he has been a lecturer at his alma mater in Cork. His research interests may be summarized as soil-structure interaction and embankments on soft ground. Dr. Creed's speech is transcribed below in order to convey the freshness and informality of his presentation.

"Mesdames et Messieurs, Ladies and Gentlemen, I would first like to thank Dr. Holeyman for inviting me to say a few words to you this morning on the subject of earth retaining structures. I do not wish to detain you nor to blind you with technical details. I merely wish to raise some questions of personal interest and I hope that some of those questions may interest yourselves.

My themes take the form of questions. Firstly, discussion - is it real or mythical, important or not, is it alive or dead? I think it may be alive here today. Secondly, design of walled excavations - where are we going? Ground movements - are we hopeful or helpless?

We have been bombarded with an ever increasing rate of publication of technical papers. In our quest for knowledge, we are often frustrated when we try to derive real and useful benefit from reading these papers. What we really need is discussion. We have quite a literature already. I ask again - is the art of discussion dying or indeed has it died already? I would like to bring you back some 43 years to see a classic example of discussion at its best. Again we go back to the old man himself, Terzaghi, 1944, Stability and Stiffness of Cellular Cofferdams. In this paper, Terzaghi was very critical of contemporary practice in the design of cofferdams. The paper provided an outstanding discussion that included very diverse points of view, from scepticism to good engineering. The paper was 32 pages long; the discussion ran to 83 pages. I ask - could this happen today? The quality of certain aspects of the discussion is best summed up in a quotation which I take from Terzaghi, his reply to the discussion: 'Considered as a whole, the procedure for designing the Kentucky cofferdam, as described by Mr. Hedman, is an outstanding example of sound engineering. This procedure discloses a thorough grasp of all the theoretical principles involved combined with mature judgement, and it deserves the attention of every engineer who faces the problem of designing a cellular cofferdam.' There were not many theoretical principles in those days and I would think that it would be stimulating reading for anybody even today. The point I would like to make is that it was the complete discussion that followed the paper rather than the paper itself that illustrated the state of the art. I think that rarely happens today.

The second example also relates to cellular cofferdams. I will move forward quickly. 1981, Journal of Geotechnical Engineering ASCE: a 14 meter high cofferdam, dewatering of the cell fill to prevent bursting of the sheet pile interlocks, artesian pressure relief using pumps located some 200 meters away to prevent base uplift failure. This is of much relevance to an earlier topic, ground water control. It seemed a very magnificent project. Everything went well, according to the paper. In the discussion, some other engineers drew attention to a cell failure that occurred in an otherwise very successful project. They suggested that over-reliance on the Standard Penetration Test may have been a major contributor to the cell failure. Again, my point is that very critical and pertinent discussion enhanced considerably the original paper.

My summary of the design of wall supported excavations, simplistic perhaps. The main considerations: stability of the base, design of the wall and support system, ground movements. The traditional approach has been to consider stability, then move on to design the wall and forget about the movements. But I think in the 1960's movements became a bit of a worry. Peck, in 1969, suggested that movements could be considered empirically as an interim measure. At that time, or perhaps a few years earlier, we saw the development of soil-structure interaction. There has been much analysis and observation of full scale excavations during the past 20 and especially during the past 10 years. It is important at an international gathering such as this to raise questions relating to the real benefits of such research, to try to establish the state of the art without becoming too involved in the detailed relative merits of various analytical approaches. Again, I quote from our provocative reporter case histories form the basis for improved understanding of the role of ground water in geotechnical engineering'. A non-controversial point, I suppose.

Recent research has involved a cyclic operation, prediction, performance, re-analysis and more prediction. This has occurred for a large number of sites worldwide. I ask - have we made the great leap to produce useful design tools? Are we able to make a priori predictions about the behavior of wall supported excavations? Are we able to predict the movements using these techniques? Is indeed engineering judgement, based on our experience, a better tool? Again I ask - have simplified design methods come out from all this research? I think one may consider it to be very enjoyable being inside in the circle. One is swimming around in the swimming pool for a long time and perhaps when the time comes to get out, one is worried. We are afraid that, when we come out, our clothes may be gone and we may be exposed!

I refer to one case history, a type of back-analysis prediction, Roti and Friis, 1985, San Francisco Conference, a good recent example. They used a fairly simple coefficient of soil reaction approach. The results were quite good. Can this be turned into a useful design tool applicable to soils other than the Norwegian soft clays?

Ground movements. Causes of settlement. These have been discussed quite a lot. I group them wall movement, base heave, and ground water lowering. The effect of these can be estimated to some extent, at least, using quasi-elastic or elasto-plastic methods. In granular soils, we have seepage occurring; can this lead to further settlements due to the passage of fines through the soil? Lastly but not least, the little problem with which the geotechnical engineer is rarely concerned - the broken pipe. In a paper to this conference Rethati suggests he produces convincing evidence that broken pipes, etc., is the main cause of damage to buildings; in fact, over 50% of 800 cases. One must ask what is the mechanism? Is it seepage erosion? Where are the particles going? Is it collapse or wetting? Is suction raising its head again?

I think contributors to this conference have already considered base stability, whether it be piping, uplift, or base heave. In conclusion, may I suggest that for wall excavations, the interim approach should be simple calculation using the most unfavorable interpretation of ground conditions and a factor of safety of one, approaching one, let us say. Wall design, again I ask - can use be made of the modulus of subgrade reaction? Here there could be a role for the new Technical Committee on Numerical Methods? Should their brief be not to continue to move in the analysis circle but to actually make the great leap from the analysis through to some useful design which is acceptable to the practicing engineer? Indeed, the Technical Committee could draw together the work from many countries, Norway, for example. I am aware a lot of work has been done in the U.K. Aussi, je ne dois pas oublier nos collègues français qui recherchaient ce sujet depuis longtemps."

The discussion leader thanked Dr. Creed for his analysis of the late mechanism of publishing.

Floor Discussion

A. Holeyman, Franki, (B)

"As we have done for the first theme of discussion, I would like to break here for a few moments in order to ask the audience for questions. We said that we would give priority to questions and direct comments from the floor before we would move on to presentations from the floor. So May I ask if there is anybody wishing to comment or to ask questions to any one of the three presenters of this second part of the discussion? Yes. Mention your name and affiliation, please."

Mr. Cashman, Cashman (U.K.)

My name is Cashman and my affiliation is Cashman. The last presenter, Dr. Creed has proposed a philosophy: prediction, performance, re-analysis. That also dovetails in with what the reporter said for session 2 of a step-by-step approach to a ground water control installation. Having been in the game for 30 odd years, I entirely agree with this philosophy. However, Gentlemen, as, until 18 months ago, a specialist sub-contractor who had to live in the real world, may I now pose to you a philosophical question. The sub-contractor is required to put forward his proposals for, let us say, 10 wells, 25 meters deep, equipped with 15 kW pumps and put a fixed price to that. How in that case can you reconcile that contractual requirement with the step-by-step philosophical approach?

A. Holeyman, Franki (B)

Would Dr. Creed like to comment on that?

Dr. Creed, Cork University (Ir.)

I think somebody yesterday gave a solution for that - a change in contractual practice. That is really the only solution. I am very much of the opinion, even though I am an academic, that contractors get a raw deal.

A. Holeyman, Franki (B)

I would agree also that the solution lies in the contractual arrangement and a fixed price is not probably compatible with an active design approach and one would rather look at other forms of contracts such as fast track contracting that allow to take into account data that is gathered on the site as construction is proceeding. But I think that if you had to convince financial people that the amount of spending could vary by the same amount as the coefficient of permeability, that would be very difficult.

There was another request. Yes?

Hartwell, Golder Associates, (U.K.)

Just to add to Dr. Creed's suggestion that we adopt a factor of safety of one in considering such things as plug failures, may I add my own thoughts to that. I feel very strongly that is the correct way to approach the problem. It is really one of the very simplest calculations anyone could do, and the most accurate, to consider the weight of the soil and the uplift pressure. We do not have to worry about permeability. As long as we make a safe assessment of the soil density and put our factor of safety in assessing the underlying hydrostatic pressure, then

I suggest that is the place for a factor of safety, and not in doing the calculations and then applying a factor of safety, as a number of cases I have been involved in recently where consulting engineers have imposed an arbitrary factor of safety after doing the calculations. Thank you.

A. Holeyman, Franki (B)

Thank you. Any other comments or questions from the floor? Yes? Do not forget to mention your name and affiliation

Martens, Smet Boring, (B)

What I want to say about the step-by-step approach in the ground water lowering is that one of the most important problems is the time delay between the installation of the ground water lowering and the start of the foundation works. I saw the step-by-step approach in most cases impossible. If it were possible to have a larger time between the installation of the lowering and the start of the foundation work, then the contractor would be able to design the dewatering capacity on permanent regime.

A. Holeyman, Franki (B)

Thank you for this comment, Mr. Martens. I believe that indeed when you look at the step-by-step approach, either your schedule must take that factor into account, which means that you must allow yourself some time to re-direct the solution, or otherwise if you do not want to take that risk in time, you must over-design, and that is the judgement that you have to make. Are there any more comments on the presentations?

Bridge Davis, Bridge Davis, (U.K.)

We all talked about the observational method and how it can be used, but at the end of the day, you have what you have, whether you like it or not. And it still has to be paid for. Now unfortunately, contracting these days, the person who gets the job will put in the cheapest price and he has made the most optimistic assessment. And then when something does go wrong, he can only fall back on unforeseen ground conditions and that is always an argument. I do not think if we are to advance, we have to tackle this contractual issue, how one could set up a proper contract with people you trust. It must involve the owner, the engineer, and the contractor to work together to deal with the problems. Otherwise we should never make any advances in this area.

A. Holeyman, Franki (B)

Coming from the contracting world, I cannot but agree. I think it is very sound. I do not know how it would be easy to convince people to work together. I think it must be under somewhat restrictive competition or negotiated work rather than going for bidding, lowest bidding.

GENERAL CONCLUSIONS

In spite of the complexity of the interaction of water with the performance of foundations and excavations, the following general conclusions can be drawn from the discussions:

- It is unlikely that piles in general will suffer a significant downgrading of their performance from changes in the ground water level; uplift pressures must, however, be dealt with when considering deep basements founded on deep foundations.
- The performance of shallow footings can be more significantly affected by a variation in the water level. When the degree of saturation of a silt increases from 75% up to 100%, the limit load can be decreased by 20%, the settlement doubled, and the creep coefficient quadrupled. A unified adimensional representation of the load-settlement curves seem to apply at all degrees of saturation.
- The design of excavation and dewatering against uplift does not raise critical theoretical issues, and well known textbooks deal in detail with this issue. Failures seem to be rather the result of insufficient soil investigation combined with inadequate constructional practices.
- More thorough approaches to the detailed interaction between soil and water behind a retaining wall and to the sealing conditions of a natural clay layer are now available.
- A need for more intense discussions among geotechnical engineers has been identified, either at conferences or in publications.
- The active design approach has overall advantages recognized by all parties, but fails to be applied due to current contracting practices. Imaginative and revised contractual agreements should be sought.