Is Limit State Design a Judgement Killer?

The Sixth Laurits Bjerrum Memorial Lecture
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SYNOPSIS

The background for, and main principles involved in, the application of limit state design in Denmark – and the way in which this has been modified in the light of practical experience and theoretical advances over the past thirty years are presented. This is followed by brief reference to an analysis of a large number of foundation failures recorded over many years by DGI, which indicates the factors that, in reality, control the safety of foundation structures. Finally, the ambiguity of the concepts of safety for real structures is elucidated by examples from practice.

The purpose of these rather diverse considerations is to indicate some features relevant to the development of general guidelines for limit state design. This is especially relevant in countries where structural engineers are persuading geotechnical engineers to accept limit state design in a form suited to problems predominant in structural engineering. Reasonable attention must be paid to the special working conditions in geotechnical engineering, such that the judgement and practical experience remain major elements in the design of foundation structures, also where this is performed using limit state design.

1. INTRODUCTION

Thank you for inviting me to give this Laurits Bjerrum Memorial Lecture. I consider it to be a great and undeserved honour which, I suppose, has been granted to me not only because Laurits was a close friend, but also because he was a compatriot; I think of myself today as a kind of spokesman for the many Danish geotechnical engineers who knew and liked Laurits, both as a good and inspiring friend and as an internationally recognized geotechnical expert.

We Danish geotechnical engineers remember Laurits as an outstanding representative of our country's engineering profession, and it can, of course, only please us if, here in Norway, Laurits is thought of as a Dane, even though much of his work concentrated on, or emanated from, NGI.

1.1 Is limit state design a judgement killer?

Last year's memorial lecture was, as you will remember, on the topic of Peck's thought-provoking question: "Where has all the judgement gone?" When I see the progressive introduction of limit state design in many countries, I fear that part of the answer to Peck's question may be that the judgement is being killed by limit state design. The main concern is that limit state design is accompanied by thoughts and principles which, I believe, can be inappropriate in connection with the design of foundation structures in practice.

To justify my concern, I would like to point out that for more than 25 years most foundation structures in Denmark have been designed in accordance with the principles of limit state design. It is, as a matter of fact, almost 30 years to the day since I was involved in the first practical applications of Brinch Hansen's, then new, design procedures based on the theory of plasticity (Brinch Hansen, 1953), which we still use extensively today. After more than 10 years of unofficial practical application, the principles were laid down in the Code of Practice for Foundation Engineering issued by the Danish Association of Civil Engineers in 1965, which, as far as I am aware, is the first code of practice entirely based on limit state design with a pertaining system of partial coefficients. It was so far ahead of its time that, in reality, it was necessary to overrule the current codes of practice for concrete and steel, stating the partial coefficients necessary for calculations involving the strengths of these materials. Danish geotechnical engineers, and especially Brinch Hansen, have done a great deal to gain support for these calculation principles amongst their colleagues in other countries, but unfortunately they have only achieved limited success. Meanwhile we have, as it were, been by-passed by structural engineers and probabilistic experts who now press the geotechnical profession to accept limit state design in a form suited to the problems which dominate these branches of engineering. The present situation is illu-

strated by parallel publications from Norske Sivilingeniørers Forening (1979) and Statens Planverks Författningssamling (1979), by the joint report from the Nordic Committee on Building Regulations (1978), by a series of papers in "Ground Engineering" (Semple 1981, Simpson et al. 1981, Smith 1981, Boden 1981, Krebs Ovesen 1981, Bolton 1982) and by EUROCODE No. 1 (1981).

When it is considered that typical steel constructions and typical foundation structures differ widely as regards types of load, material properties, problems and traditions of calculations, is it not surprising that, especially steel constructors, together with probability theorists, have given limit state design a form, which, when transferred to geotechnical engineering, in my opinion, will lead to the deterioration, in several respects, of normal design practice. It is not least to counteract such a development that I have chosen to describe our experiences with limit state design as a practical tool for the design of foundation structures.

In this way I hope to substantiate the need to pay attention to the special working conditions of geotechnical engineers when general guidelines for limit state design are developed, such that practical experience remains a key element in the design of foundations, also when limit state design is applied.

My lecture consists of three parts:

I will start with a brief outline of the *background and main principles* involved in our application of limit state design, and how we concurrently have modified these with our practical experience gained over about 25 years.

I will then refer to an extensive analysis of the many cases of *foundation malpractice and foundation failures* which we have recorded over a long period of time, so as to indicate which factors, in reality, control the safety of foundation structures.

Finally, I will use practical examples to show how ambiguous even some of our best *calculation procedures* can be, in order to emphasize the need for "judgement" in the design process.

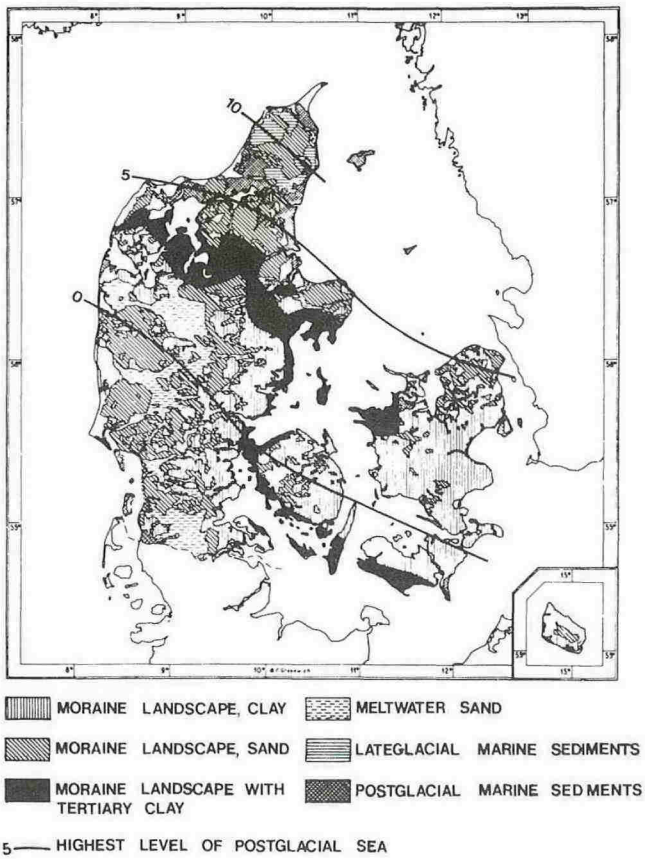

Fig. 1. Geology of Denmark.

2. BACKGROUND AND MAIN PRINCIPLES FOR THE APPLICATION OF LIMIT STATE DESIGN IN DENMARK

2.1 Geological conditions

To give an account of the background for our use of limit state design, it would be reasonable to begin with an outline of the geology of Denmark, since geological conditions always play a decisive role in our assessment of almost all foundation problems. However, I must restrict myself to noting that we encounter a wide variety of surface deposits, ranging from limestone and strongly preconsolidated Tertiary clays and glacial deposits, to normally consolidated, postglacial deposits, often with a significant organic content. The latter includes deposits of the Littorina Sea which occur up to the level shown in Fig. 1 and reach an elevation of > 200 m in Norway.

We do not have clay types as sensitive as the Norwegian quick clay, and rocks only occur at the surface on Bornholm, the Faeroes, and Greenland. On the other hand, we have built up considerable experience with heavy Tertiary clays, similar to London Clay, which have presented us with many practical problems in the blackened area in Fig. 1.

2.2 Soil investigations

Motivated not least by a relatively large number of foundation failures, soil investigations have been increasingly applied over 25 years, such that today, for virtually all buildings larger than single family houses (and also for many of these), geotechnical borings are performed with appropriate geological examination, classification tests and field vane tests.

2.3 Types of construction

Our experience covers most of the traditional types of foundation structures, although I must point out that this does not include large dams. Neither do we yet have

a great deal of experience with off-shore structures, although we are currently preparing a code of practice for Danish off-shore structures. Earthquakes, of course, are of no practical significance in Denmark.

2.4 Calculation procedures

In the form we have used over many years, limit state design primarily involves two separate investigations, one of the ultimate limit state and one of the serviceability limit state.

In the *ultimate limit state* we usually investigate, according to the theory of plasticity, the fully developed rupture without restrictions as to the magnitude of the deformations. How literally we mean this may best be demonstrated by mentioning that in the determination of the ultimate bearing capacity of a pile all layers are considered bearing, including those which, under actual working conditions, affect the pile with negative friction. As examples of fully developed ruptures, with rupture in both the structure and the soil, an anchored sheet wall and a raft foundation are shown in Fig. 2. Obviously the same distribution of the earth pressure and the reactions should be applied in both the geotechnical and the constructional part of the calculations, i.e. it is virtually impossible to carry out the geotechnical investigation of the state of failure without knowing or determining the strength of the structure. Therefore, the examples illustrate the impossibility of separating the calculations (including determination of design criteria) into a geotechnical part and a constructional part, and they furthermore show that the choice of method of calculation for the structure can be of decisive significance for the calculated safety factor in the geotechnical calculations.

In the *serviceability limit state* we try to investigate the actual behaviour of the structure without considering the magnitude of the safety against failure. For most of our traditional foundation structures experience has shown that the investigation of the ultimate limit state is decisive as to the dimensions of the foundation structures and therefore investigation of the serviceability limit state is, in practice, often restricted to a rough estimate. In those cases where greater calculation precision is required, more advanced investigations are performed, in the form of finite-element calculations for example. However, it must be realized that the necessary geotechnical parameters can only be given with considerable uncertainty and – what is perhaps more serious – we often lack reasonable well documented design criteria if these cannot be given as deformation requirements determined by the serviceability of the construction. The most commonly used methods of calculation are described by Lundgren and Brinch Hansen (1961).

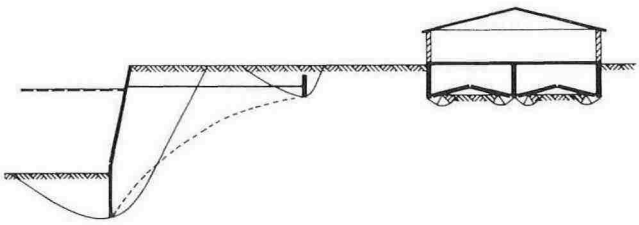

Fig. 2. Ultimate failures.

2.5 Design criteria

In principle, design criteria should be established as the result of cost benefit analysis which takes into account all the consequences of failure, including the political consequences. In practice this is unrealistic. We have instead decided to assume that traditional, successful, foundation structures represent reasonably economical, safe solutions, and have therefore, as far as possible, based our design criteria on backcalculations of such structures.

In Denmark the expression "traditional, successful, foundation structures" has been a relatively well defined concept for over 50 years because, throughout this time, we have had a Code of Practice for Foundation Engineering, issued by the Danish Association of Civil Engineers, providing relatively rigid guidelines for the design of normal foundation structures like shallow footings, piles, retaining walls and sheet walls. On the basis of experience gained through the years, these regulations have been tightened where there have been failures, and gradually relaxed in fields, where, for example, the absence of failures has indicated significant overdesigning.

2.5.1 *Ultimate limit state design criteria*

When we started using the partial coefficient system in the early 1950'es, the values of the partial coefficients were established by comparing calculations with the current code of practice rules. In this way we succeeded, right from the start, in establishing, partial coefficients to which it has only been necessary to make minor corrections. For example, an original partial coefficient of 1.25 on $\tan \varphi'$, measured by triaxial tests, was obtained for cohesionless soil by comparison with sheet wall calculations, while equivalent investigations of shallow foundations and retaining walls indicated somewhat lower values. Model tests and theoretical considerations, together with the absence of failures, have formed the basis for a gradual reduction of the partial coefficients to between 1.2 and 1.1, depending on the load combination, with simultaneous introduction of the so-called plane angle of internal friction which is 10% greater than the angle of internal friction measured by triaxial tests (Bent Hansen, 1979). Although we today design these structures more daringly than ever, based on a relatively optimistically established angle of internal friction (normally φ plane $\simeq 35°$ to $40°$) and using a calculation model which is, theoretically, on the unsafe side (rough walls and bases, kinematically

admissible states of failure with fully developed ruptures according to the theory of plasticity etc.) we have not recorded a single failure of these structures which can be ascribed to overestimation of the characteristic angle of internal friction of the soil. In my opinion, this indicates quite clearly that our calculation model for cohesionless soil, in spite of what is said above, is considerably on the safe side in a way we do not yet recognize.

Similarly, for cohesive soil there is undoubtedly a margin of safety in our design of ordinary shallow foundations based on undrained failure without taking into account at least part of the often considerably greater bearing capacity corresponding to drained conditions. In this respect our heavy Tertiary clay may form an exception with the drained conditions sometimes being critical. For example, a bearing capacity failure occurred beneath a gravity quay wall in Århus, founded directly on Tertiary clay very similar to London Clay, about 50 years after construction, which is completely analogous to Skempton's experience with London Clay (Skempton, 1977).

Some of the tightenings of the code motivated by failures have not been introduced as increased partial coefficients, but, for example, by the reduction of the characteristic undrained shear strength of organic deposits (Bjerrum, 1972), by specifying more rigorous design assumptions such as water-filled tensile cracks in earth pressure calculations, or by raising the minimum requirements for soil investigations for pile foundations. It is particularly noteworthy, that geotechnical engineers, in several cases, have had to modify the requirements of our codes of practice for load, timber, steel and concrete because these had been fixed without paying adequate attention to the special conditions which apply to foundation structures.

2.5.2 *Shake-down design criteria*

The above mentioned *normal partial coefficients* are used for the investigation of the ultimate limit state according to the theory of plasticity, which is usually design-controlling. Experience has shown that this investigation usually also implies sufficient safety against repeated plastic yielding of traditionally fixed foundations, since we have no shake-down problems with these structures, apart from a few underdesigned chimney foundations. If, as a supplement, the problem of shake-down is investigated for a special structure according to the theory of elasticity we have therefore found it adequate to apply *reduced partial coefficients,* established such that the elastic investigation is not decisive for the traditional fixed foundations.

2.5.3 *Serviceability limit state design criteria*

A large number of court cases concerning subsidence damage to buildings shows firstly that there is an enormously wide range of opinions as to what constitutes acceptable subsidence damage in traditional building constructions, and secondly that it is more often the duration of the subsidence, rather than its magnitude, which is the cause of the court case. Relatively large subsidence, and accompanying large cracks, are accepted when they can be repaired within the first year of the life of the building, whereas very minor cracks are considered as unacceptable if they – even at a very slow rate – continue to develop because of relatively limited settlements caused by secondary consolidation, transformation of organic matter, seasonal variations in the water content of heavy clay, or varying negative friction on piles. The result is that we now, as a general rule, require the base of all shallow footings to be below postglacial deposits with a significant organic content whereafter it is the required safety against rupture in the ultimate limit state which is design-controlling for the size of the foundation. Exceptions are very large foundation loads where subsidence criteria can be decisive. Similarly, measures have been developed for foundations in heavy clay, in the form of extra foundation depths, reinforcement of foundations, restrictions on the height of adjacent trees etc., and stricter requirements concerning the determination and treatment of the negative friction of piles have been introduced, i.e. design criteria regarding the serviceability limit state have been replaced by practical rules concerning the execution and the design of the foundations in a number of cases. In more special cases the results of our traditional subsidence calculations are compared with generally known subsidence criteria which, with very few exceptions, have functioned satisfactorily.

Concerning our building materials, we have found it difficult to attain reasonably well substantiated design criteria in the serviceability limit state. Back-calculations of successful constructions show that, in the serviceability limit state, according to our normal calculation assumptions, fully developed yielding may occur in the soil as well as in the construction. It is not necessary to go further than to a traditionally designed reinforced concrete foundation to see that an elastic-plastic investigation of the serviceability limit state with the well known reaction distribution for a rigid foundation may indicate yielding in the reinforcement as well as locally in the soil. General design criteria, based on allowable stresses in the soil and/or the construction, equivalent to a partial coefficient > 1.0, are therefore not acceptable. On the contrary, a value of unity must normally be allowed in the serviceability limit state. In special cases it may be desirable to use a more or less elastic investigation of the serviceability limit state as design basis, for example to ensure a reasonable degree of imperviousness and/or absence of cracks in the raft foundation illustrated in Fig. 2. In such cases we have for a number of years used the same *reduced partial coefficients* as in the shake-down analysis.

2.5.4 *Changes in the design criteria*

In foundation engineering we have tried to establish our design criteria by paying due regard to the old truth that the result of the design process, for example in the form of the dimensions of a foundation structure, can be considered a function of the 4 factors:

- Load
- Material parameters (including soil)
- Calculation procedure
- Design criteria

It is a frequently emphasized fact (Lambe, 1973) that an isolated improvement in one of these factors in the direction of a more correct measurement, calculation and so on, without recognition of the mutual relationship, can result in deterioration of the overall design process. Therefore we have, over the years, refused to introduce such isolated "improvements" in the practical calculations. I will only mention a single example of this:

When our code of practice for concrete adopted limit state design, permissible widths of cracks were introduced as design criteria in the serviceability limit state to safeguard against corrosion. This led to a considerable cost increase of normal foundation structures. We do not yet know whether it is the risk of corrosion or the width of cracks, which have been overestimated, but the fact is that we have not registered any failures which could justify such a tightening, and we have therefore rejected the proposed crack widths as practical design criteria in connection with our normal calculation procedures.

This example clearly demonstrates the problem which geotechnical engineers can encounter, particularly in those countries where structural engineering has already established a limit state design practice for concrete and steel with the relevant design criteria, determined either by probabilistic safety considerations or from experience with structures, the function and purpose of which may differ considerably from those of foundation structures.

I will conclude this section by maintaining that back-calculations of existing, satisfactory foundation structures is the best way in which to establish design criteria, both in the ultimate limit state and in the serviceability limit state.

3. ANALYSIS OF FOUNDATION FAILURES

From an international point of view the construction of foundation structures in Denmark can almost be considered as a prototype test of the practical application of limit state design in the version described above.

The background for this is:

1. For many years, foundation structures have been designed and constructed following well defined guidelines laid down in our code of practice, equivalent to what we today refer to as limit state design.
2. Virtually all significant foundation failures and foundation accidents are registered by DGI. At DGI we have, over a period of many years, investigated more than 100 foundation failures per year, and due to our key position in a small country with a high level of information we undoubtedly learn about all significant failures. In view of our extensive record of failures we believe that the review of causal relations given below covers almost all types of foundation failure of traditional foundation structures under Danish conditions. This means that we are virtually able to reject hypothetical failure possibilities for traditional structures if we have not registered at least one occurrence of such a failure in practice.

These experiences form an integral part of the revisions of the Danish Code of Practice for Foundation Engineering and hence one might wonder why we still experience so many failures. I will try to give an explanation for this below for the groups into which we normally classify failures.

3.1 Highly compressible deposits

Cavities in limestone and chalk, interglacial deposits with a high content of organic matter, and, in particular, compressible postglacial deposits are responsible for subsidence damage to many buildings (including many houses), pipelines, roads etc. Further these deposits have given rise to stability- and bearing capacity failures in connection with earth fills and large live loads, and have under adverse conditions resulted in failure of sheet walls or damage to neighbouring buildings. The reason for these failures is generally the lack of knowledge as to the presence of these deposits and/or their significance for the relevant structure, but not - except for perhaps a few individual cases - the inadequacy of the method of calculation used. The adequate preventive measures are therefore improved geotechnical investigations, together with the requirement that the necessary calculations are performed.

3.2 Heavy Tertiary clay

Even in a climate like that of Denmark, heavy Tertiary clay behaves as swelling clay since seasonal variations in water content can give rise to extensive, both horizontal and vertical, ground movements which can destroy traditionally founded buildings. The extremely unfavourable strength properties have also given rise to extensive stability failures, collapsing sheet walls, tilting retaining walls, and bearing capacity failures.

The necessary preventive measures again do not require alteration in the general methods of investigation, but that it be made clear that any building in an area with high-lying Tertiary clay is a special case which

may require special measures, radically different from traditional Danish practice in both geotechnical design and practical construction.

3.3 Water

Apart from moisture damage, water, in the form of hydrostatic pressures or seepage forces, is a critical factor in many of our failures:

Coastal erosion every year produces stability failure along our coastline, which causes retrogression by several meters a year, occasionally in dramatic events.

Ground water erosion, water flow to or from drains, or leaking pipes, has removed supporting soil strata and thereby caused stability failure or serious subsidence damage.

Direct water pressure on buildings or *pore water pressure* in the soil has resulted in the uplift of embedded tanks and suchlike, collapse of retaining walls, collapse or destructive deformation of sheet walls, landslides, stability failures etc. The damaging water pressure can often be attributed to lacking drainage or inefficient drainage due to frost, ochre formation, pump failure etc.

Increased partial coefficients on water pressures might have prevented some of these failures, but the main reason for their occurrence is that the decisive water pressure was neither recognized nor correctly accounted for. The necessary preventive measures are therefore adequate preliminary investigations to identify the critical water pressures and to monitor these during construction, and after completion. The water pressures and their effects should be controlled by the use of erosion filters, drainage, overflow facilities, pumping on well points or filter wells with the necessary alarm installations and emergency energy supply etc. Further the known and/or controlled water pressure should be correctly entered in the geotechnical calculations, i.e. the preventive measures are not simply a tightening of the calculated safety limits by, for example, increasing the value of a partial coefficient.

3.4 Thermal damage

Both frost and heat have produced subsidence damage in the form of:

- frost heave because of inadequate insulation and/or drainage of traffic areas, building foundations, floors of cold stores and skating rinks.
- heat damage caused by accelerated secondary consolidation in organic deposits which have been heated as a result of pipebursts or inadequate insulation of underground district heating pipes.

3.5 Corrosion

Corrosion of steel sheet pile walls has resulted in the collapse of quay walls.

Corrosion of underground steel oil tanks has caused ground water pollution.

Sulphate attacks in sea water and alkali silicate reactions stemming from unsuitable aggregates have decomposed concrete constructions.

3.6 Neighbouring conditions

We have had extensive records of damage where it is not the building construction itself which is the subject of the damage, but existing neighbouring buildings, pipes, roads etc.

The causes are:

Excavations, pile driving, filling or *stockpiling* which give rise to ground movements or regular stability rupture below neighbouring buildings.

Vibrations caused by excavation, transport, pile driving, withdrawing of sheet walls which have damaged existing buildings.

Underpinning of existing buildings, made necessary by a projected construction, has sometimes been carried out so daringly that dramatic collapse has resulted.

Ground water lowering has damaged neighbouring buildings with poor foundations due to compression of the supporting strata under a foundation, or given rise to increased negative friction on piles, increased shrinking damage in heavy clay, rotting of wooden piles.

Oscillation, because neighbouring buildings, as well as the new construction, can develop resonant oscillation caused by vibrations from a variety of machine foundations and other vibrating sources.

Antiquated foundations, especially middle age foundations on loose foundation stones, are naturally included in this group.

3.7 Faulty design and execution

After the damage has occurred and the cause has been recognized it is tempting to place all failures under this heading. Apart from this, my national pride forbids me to go more deeply into this category. It can, however, be ascertained that neither this category of failures, nor those mentioned above, can be avoided by minor adjustments in the partial coefficient system. Only recognition of the problem and, constructive measures, can reduce the risk of damage.

3.8 Classes of safety

Related to the limit state design method, a classification of structures has been introduced in many countries with different safety requirements regulated according to the consequences of failure, and, in some instances, with partial coefficients determined by the theory of probability. In my opinion the safety class idea is an expression of common sense, but I hope that, with the above review of our archive of failures, I have documented that, for foundation constructions, failures

cannot be avoided by focussing on partial coefficients, but by extra requirements as to the quality and extent of geotechnical investigations, calculations and control measurements before, during and after construction.

4. DESIGN PROBLEMS ILLUSTRATED BY PRACTICAL EXAMPLES

Even when we satisfy these requirements and thereby establish a relatively correct calculation model, we should not blindly apply fixed values of partial coefficents. With some practical examples I will show how it can sometimes be difficult even to decide whether one should multiply or divide by the partial coefficients, and how, in other instances, they have no influence at all on the true safety. At the same time I will try to show that some of the commonly recognized calculation methods can be so far on the unsafe side and others so much on the safe side, that they can in themselves have a decisive influence on both the calculated, and actual, safety of a construction.

4.1 Kinematic and static admissibility

Most geotechnical rupture calculations are based on the so-called *kinematically admissible states of failure,* where it is assumed that the so-called zone ruptures are kinematically admissible.

This method of calculation is, as a rule, on the unsafe side, and has therefore, in the course of time, resulted in several serious cases of underdesign. As I am here in Norway I can hardly avoid recalling that the first time we became aware of this was in connection with a stability investigation of a quay in Horten, constructed in 1953 (Bjerrum et al., 1958). If the coal tip is ignored, Fig. 3 closely represents the quay at Horten. The critical rupture line for a $\varphi = 0$-analysis was long envisaged to be as shown in Fig. 3, until we became aware that this must involve a major exaggeration of the passive earth pressure on the sheet wall. Even though passive zone rupture is kinematically impossible in connection with the considered rupture, the passive zone rupture will represent a much more correct value for the earth pressure. Many other examples could be mentioned, and the conclusion must be that the result of an extreme investigation should not be rejected on the basis of a rupture line being kinematically impossible. On the contrary, it should be ensured that the earth pressure and reaction distributions are established such that it is not possible to indicate (kinematically impossible) rupture lines along which equilibrium cannot be achieved without exceeding the failure condition.

The counterpart of kinematically admissible are *statically admissible* modes of rupture where in principle stress conditions everywhere satisfy the equilibrium conditions without violating the failure condition, but where on the other hand the deformations in the state of rupture are not considered.

This could be thought of as the kinematically admissible solution being a possible *way of rupture,* while the statically admissible solution is a possible *way of support.* Being confident that the construction (or nature) is more ingenious than the constructor, we could expect that the kinematically admissible solution will generally be on the unsafe side, and the statically admissible solution will be on the safe side.

Unfortunately, this principle is not generally valid in geotechnical calculations.

Returning to Fig. 3, which, with the coal tip, is essentially similar to a coal quay being built in Denmark, it would be tempting to start the design by carrying out an ordinary sheet wall calculation and next, with the calculated earth pressures as known forces, from the overall stability determine the total anchor force and the corresponding earth- and coal pressures on the coal wall. Even with a smooth wall, which is usually a safe assumption, the transfer of pressure from the coal wall to the coal and earth will apparently not give calculation problems, since a more or less statistically admissible stress distribution can be indicated without increasing the earth pressure on the sheet wall. However, if a constant volume failure can take place in the coal, a circular rupture line may develop as shown, starting from the base of the coal wall. It would be kinematically admissible to presume that the coal and the earth between the two rupture lines would rotate as a rigid unit, while the coal above the upper rupture line remains immobile. The corresponding earth pressure on the coal wall may be considerably less than that determined by the stability investigation. If the coal is so loosely stacked that failure is accompanied by volume decrease, then the kinematically admissible upper rupture line will be even steeper, and the anchor pull transferred by the coal wall even less, possible equivalent to the earth pressure coefficient $\cos^2\varphi$. The missing stabilizing forces must then be transferred as earth pressure on the sheet wall, i.e. under these assumed deformation conditions the more or less statically admissible solution can be catastrophically on the unsafe side.

The theoretical background for this is that we, in soil mechanics, generally cannot fulfill the condition of nor-

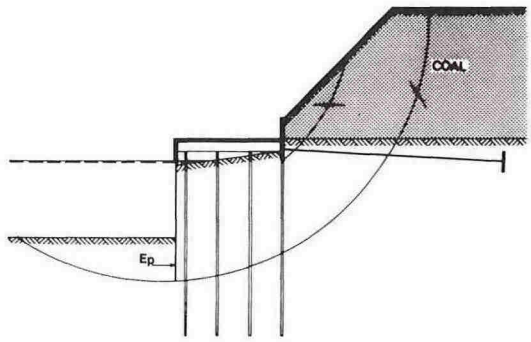

Fig. 3. Cross section of a quay with a coal tip.

malcy according to which rupture is accompanied by an increase in volume characterized by the angle of dilatancy being equal to the angle of friction.

I will conclude from the above that, unless the methods of calculation have been established on extensive practical experience, one should rely on neither purely statical nor purely kinematical considerations, but, as far as possible, take account of both and be particularly critical concerning ways of rupture which more or less clearly require a volume increase in the state of rupture.

Fig. 3 reminds me that, during discussion with foreign geotechnical engineers, I have several times noticed that Brinch Hansen's method for determining the stabilizing effect of piles is not widely known. I refer to Lundgren and Brinch Hansen (1961).

4.2 Bandar Shahpour

As the next example I will describe a quay in the Persian Gulf where I had the pleasure of working with Ove Eide as consultant. Fig. 4 shows a cross section through the 50 m wide quay with 4 rows of relief piles on the landward side. In the period 1976–1979 some 5 km of the construction was completed in the Iranian port of Bandar Shapour after project designed by the Danish-Iranian firm Iran-Kampsax and with a great part of the investigations referred to below carried out by Kampsax's Danish subsidiary company Geodan, under the leadership of the civil engineers E. V. Jensen and Jens Lollike.

Soil investigations indicate rather uniform soil conditions. Of special interest in relation to the following is that a relatively homogeneous delta sediment of soft, silty clay, with thin beds of silt and fine sand is found from original surface level +2 to +3 m, down to -12 to -15 m. Typical data are: natural water content $w \simeq 30\%$: liquid limit $w_L \simeq 35\%$; plastic limit $w_P \simeq 20\%$; clay content $\simeq 30\%$; organic content 0.2%; Cl^- in pore water 7.5%. Below the upper desiccated crust in-situ field vane tests showed values of $c_v \geq 20 + 2z$ kN/m², where z (m) is the depth below mean sea level. Unconfined compression tests and cone tests indicated undrained shear strength of the same order of magnitude. The sensitivity was measured to be

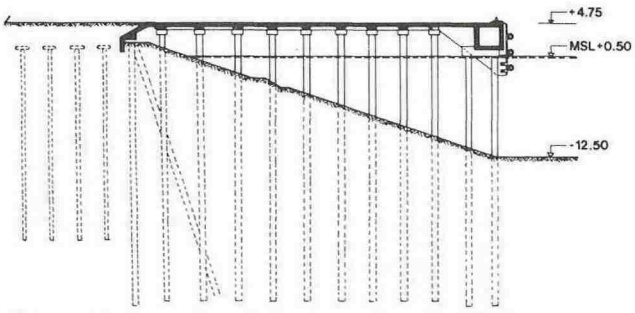

Fig. 4. Cross section of the quay at Bandar Shahpour.

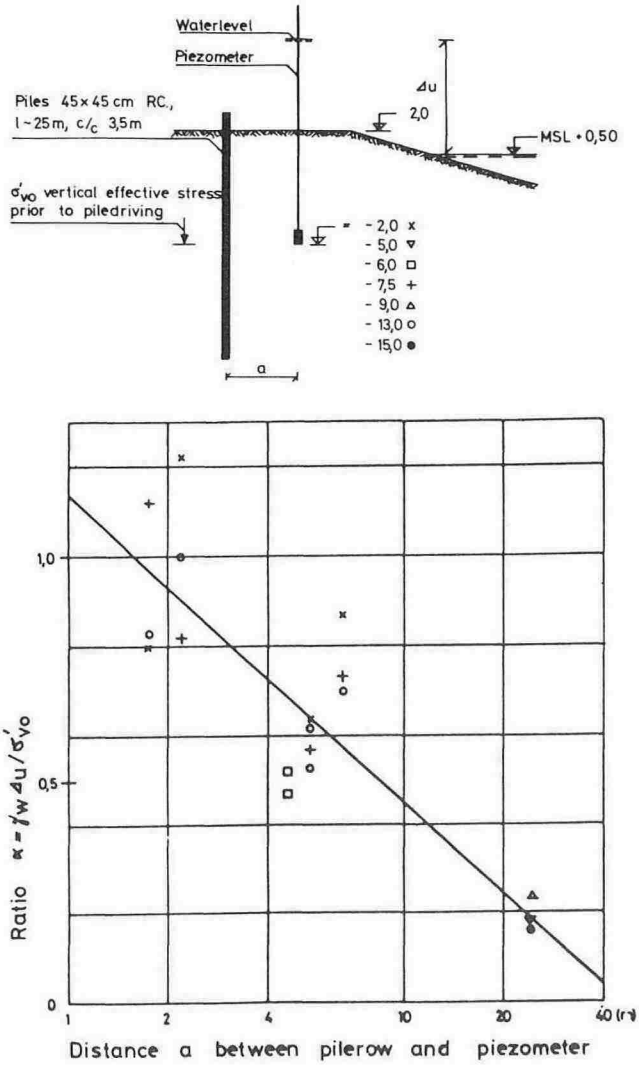

Fig. 5. Increase in pore pressure vs. distance from piles.

between 2 and 4. Effective parameters determined by triaxial tests average $\varphi' \simeq 35°$ and $c' \simeq 3$ kN/m² for both active and passive tests, many of which were carried out at NGI. Consolidation tests, also partly carried out at NGI, indicated essentially normal consolidation below the desiccated crust.

After some preliminary excavation, work commenced with the driving of relief piles with a single acting hammer. A total of 104 piles had been driven in about 2 weeks, distributed in all 4 rows within a length of about 100 m, when a slide occurred damaging almost all the piles.

The remaining ca 2700 relief piles were thereafter driven one row at a time, beginning on the landward side, with extensive control measurements to monitor movements and pore water pressure.

Measurements to bench marks placed between the piles and the excavated slope indicated horizontal movements of from 3 to 5 cm per row of piles (theoretical pile volume 5.8 cm per row) which took place simultaneously with the pile driving. In the course of the

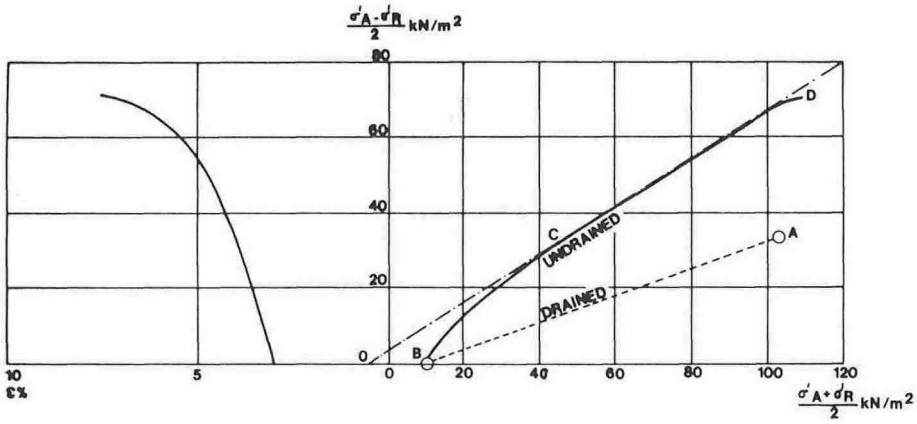

Fig. 6. Preconsolidated undrained triaxial test.

next 2 to 4 weeks, horizontal movements of about the same magnitude in the opposite direction were recorded i.e. the horizontal movements were practically reversible.

The increase in pore water pressure was also measured during pile driving. As an example, Fig. 5 shows the increases in pore water pressure, recorded during driving of the first row of piles, in piezometers placed between ca 2 and 25 m from the row of piles at various elevations between -2 and -15 m. The figure shows that when the increase in pore pressure $\Delta u \cdot \gamma_w$ (Δu is the head of water) is expressed in relation to the vertical effective stress σ_{v0}' before the piledriving, the depth to the piezometer seems to be of limited significance. Regardless of the depth, increases in pore pressure were measured between ca 100% of σ_{v0}' 2 m from the pile row, and ca 20% of σ_{v0}' ca 25 m from the pile row. Two or three weeks after completion of the pile driving the excess pore pressures had dissipated by 50%.

Corresponding to Golder's "lost assumption" (Golder, 1979) the pore water pressure should be counterbalanced by shear stresses in vertical sections. In the present case, however, this would require shear stresses by far exceeding the values of undrained shear strength measured. Most of the registered increase in pore pressure therefore must cause a corresponding reduction in the vertical effective stress. Therefore I believe that, immediately after the pile driving, the earth can be considered as preconsolidated, with $OCR \simeq \sigma_{v0}' / (\sigma_{v0}' - \Delta u \gamma_w)$.

By introducing time intervals, determined by the pore pressure measurements, between driving the individual rows of piles, driving of the relief piles was accomplished without any additional problems. During and after driving the seaward row of piles, pore pressures were measured which, applied to a stability investigation in effective stresses, showed a factor of safety $F \simeq 1.0$, and it may seem surprising that the pile driving could continue under these circumstances without further failures.

The explanation is most easily expressed by studying one of the triaxial tests, illustrated in Fig. 6. The sample is initially consolidated at A, which represents in-situ stresses, and then at B which, after application of backpressure $\Delta u \gamma_w$ represents the preconsolidated state after increase in pore pressure. The test is then continued undrained until active rupture, with measurement of the pore pressure. The line O-C-D determines the effective strength parameters i.e. the section from C to D appears, as far as the stresses are concerned, to represent a fully developed rupture, while the deformation measurements indicate a totally different situation, for example with a factor of safety > 2 at point C. If we consider the sample to be a bearing construction with stress conditions equivalent to those at point C, and we wish to determine its safety against undrained failure, it will, I believe, be illogical to try and express this safety as partial coefficients on the effective strength parameters. There is no doubt as to the value of these parameters, since they have been documented directly by the stress and pore pressure measurements. The stability of the sample as well as the quay structure during pile driving depends entirely on the difference between the pore pressures measured and corresponding values at failure. Therefore, in principle, the partial coefficients should be applied on the available difference in pore pressure or, more practically, on the equivalent undrained shear strength.

The mechanism involved in the triaxial test and the quay structure respectively is analogous to that of a solid block resting on a table and acted upon by a spring as shown in Fig. 7. If the spring is stretched and the block is on the verge of sliding - or if it slides but remains on the table - an investigation of safety based on stresses will indicate a factor of safety against failure of 1.0. If, however, the block is moved by some unknown reason and the spring thereby unloaded, it becomes obvious

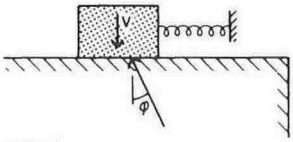

Fig. 7. Sliding block.

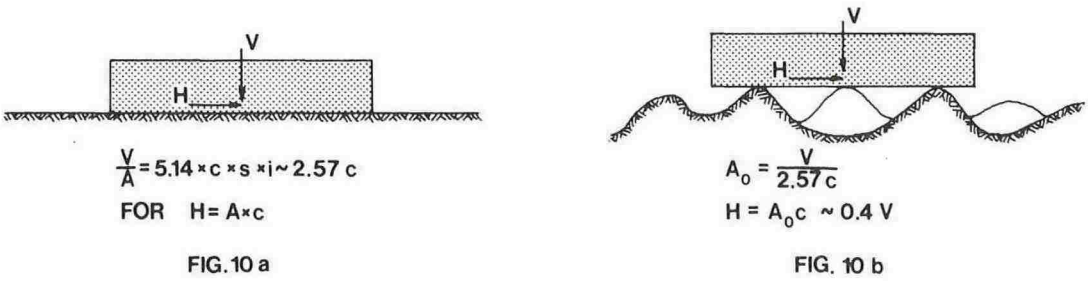

Fig. 10. Problems of anchor blocks on clay.

reason than elucidating the pitfalls in uncritical application of the partial coefficient system.

We can here simplify the problem to that shown in Fig. 10a, with an anchor block of area A on clay with, in the design state, an undrained shear strength c and centrally acting loads V and H.

The vertical bearing capacity can be expressed as

$$\frac{Q}{A} = 5.14\, sci,$$

where s is a shape factor and i is an inclination factor which decreases from 1 for H = 0 to 0.5 for H = Ac. For pure sliding with H = Ac, $s = 1$ and therefore V ≤ Q = 2.57 Ac.

Considering the fully developed failure after initial sliding has taken place, we are concerned about a "light" foundation with $V/A < 2.57\, c$ which, because of unavoidable irregularities in the surface of the earth, will not remain in contact with the earth over its entire base, but only with a part of it, A_0, which is sufficient to ensure the vertical bearing capacity. The problem can perhaps best be illustrated by considering a completely weightless block. After (almost) horizontal movement it will only be in contact with the uneven earth surface at three points, as shown in an exaggerated manner in Fig. 10b, and with $A_0 = 0$ the resistance to sliding vanish, i.e. H = 0. Reapplying the vertical load V during continued sliding, the area of contact increases to

$$A_0 = \frac{V}{2.57\, c}$$

and the permissible horizontal load is consequently

$$H = \frac{V}{2.57} \simeq 0.4\, V$$

The important findings relevant to "light" anchor blocks are:

1) the considerable reduction in sliding resistance in comparison to an ordinary short term investigation, and
2) the fact that even though the normal partial coefficient has been applied to the undrained shear strength which is critical for the considered problem, this has no effect on the calculated anchor pull H = 0,4 V which appears unaffected by the partial coefficient.

Therefore it is clear that traditional use of the partial coefficient system in this case fails completely, and the example serves once again to show the necessity of the critical evaluation of the effect of the partial coefficient system when one is dealing with untried constructions or calculation procedures.

4.5 Anchor blocks for the Storebælt bridge

4.5.1 *Introduction*

A project for a bridge across Storebælt has been considered in Denmark many times over the past 50 years. In 1979 we had gone so far that the first tenders had been submitted before our politicians again agreed to postpone the project. This was, of course, a frustrating experience for the technical experts involved, but, before that, we had been presented with many interesting tasks, both related to the geotechnical investigations and to the design of the foundations. Here we assisted the consulting engineers of the Storebælt group, consisting of the firms Cowiconsult, Højlund Rasmussen and Rambøll & Hannemann. The following is the result of many interesting discussions with civil engineer Klaus Ostenfeld and his co-workers.

4.5.2 *The project*

The bridge project involved a suspension bridge with a free span of 1416 m. The Storebælt group designed the anchor blocks as caissons which were subdivided into cells and filled with sand, founded directly at approximately level -17 m. The horizontal dimensions are ca 152 × 60 m. The design loads are ca 900 MN horizontally and ca 2640 MN vertically, acting almost centrally on the foundation area. The plan was to construct the lowest part of the anchor blocks in dry dock and to complete the construction when afloat, using the principle well known from gravity platforms.

4.5.3 *Soil conditions*

The natural water depth is ca 12 m. Below a thin postglacial cover is firm boulder clay, down to -30 m, which is underlain by a very firm, calcareous variety of Tertiary clay, called the Kerteminde marl.

The natural deposits below the foundation level are so firm and incompressible that further details of the properties are of no importance in relation to the pro-

blem in question which is the risk that the excavated surface of the boulder clay could give rise to significant weakening of the completed construction. Extensive series of triaxial and shear box tests were performed with remoulded boulder clay, with and without increased water content. The effective strength parameters were determined as $\varphi' \simeq 32°$ and $c' \simeq 0$, while the undrained shear strength of the remoulded samples, reconsolidated with a stress path equivalent to conditions up to completion of the bridge, were determined as $c_u \simeq 0.32\ \sigma_v'$, where σ_v' is the vertical effective stress.

4.5.4 *The foundation*

Many ways of avoiding the effect of the softened boulder clay were considered. These included the use of stone foundations, injection, indentations, pumping in vacuum wells, preloading, various excavation methods etc., and I am sure that additional suggestions could be proposed by this audience of off-shore specialists. One of the proposals considered is shown in Fig. 11 with the anchor block founded on a wedge of compacted gravel and (because of the risk of collision) surrounded by a sand island. Dry excavation down to a level of -30 m must be considered out of question. Therefore we found it necessary to foresee a thin zone of softened or remoulded clay at the boundary between the gravel wedge and the underlying boulder clay. Planned model tests to elucidate the strength properties of this softened clay have unfortunately had to be postponed until the bridge project is taken up again.

4.5.5 *Drained - undrained failure*

The pull of the cable due to the dead load is applied so slowly that drained conditions will prevail during the whole period of construction, whereas the hypothetical transition to the ultimate limit state may take place under undrained, as well as under drained, conditions.

All stability investigations of the drained long term state show ample safety against failure. A stability investigation in effective stresses with the *measured* pore pressure will, therefore, undoubtedly indicate satisfactory conditions for the completed bridge. While this procedure was on the safe side in Bandar Shahpour, it will here be on the unsafe side compared to undrained failure using the pore pressure for fully developed rupture. In the case in question there is an additional reason to consider undrained failure since this, as we shall see below, can involve, perhaps sudden, transition from more to less stable stress distributions below the anchor block.

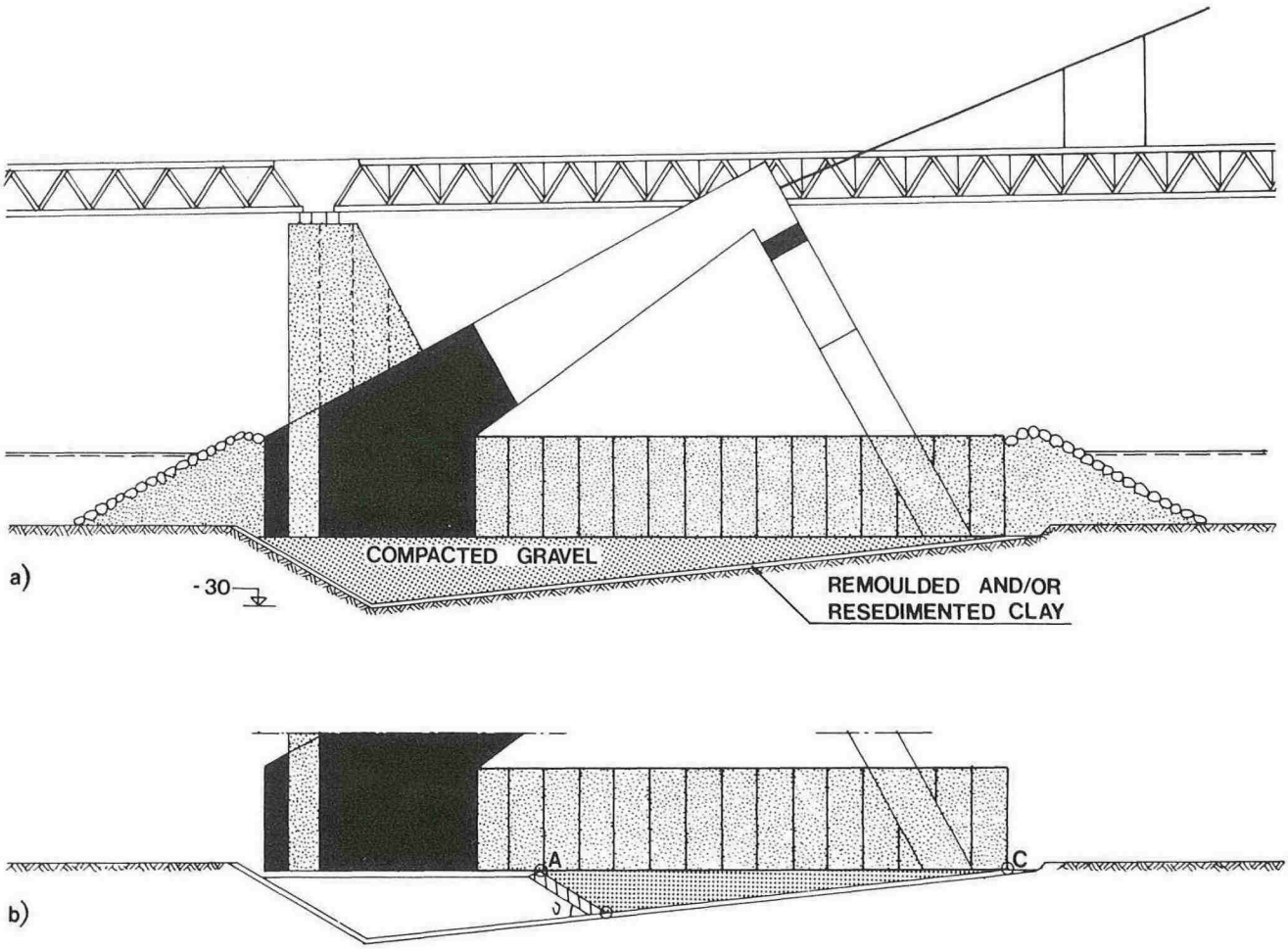

Fig. 11. Anchor blocks for the Storebælt bridge.

4.5.6 *Mode of failure*

In the critical and fully developed state of failure, the anchor block, together with part of the wedge of gravel, slides along the soft layer of clay, with simultaneous mobilization of the passive earth pressure on the end surface of the anchor block, and friction on the side-surfaces of the anchor block and part of the gravel wedge. Fig. 11 b illustrates this mode of failure, in that the shaded part of the gravel wedge is assumed to move as a rigid body, together with the undeformed anchor block, obliquely upwards parallel with the base of the gravel wedge.

Therefore, in the state of failure there is no contact between the anchor block and the gravel wedge to the left of point A.

4.5.7 *Deformation conditions*

The kinematics in the state of failure may be visualized by the strain characteristics shown in the deformation zone in Fig. 11 b forming the angle of dilatancy ν and 90° with the base of the mobile gravel wedge. Since the length of the strain characteristics remain unchanged, contact point A has the predicted direction of movement. The mode of failure is therefore *kinematically admissible.*

4.5.8 *Stress conditions*

If it is assumed that the normalcy condition $\nu = \varphi'$ is fulfilled, which is usually on the unsafe side, AB also represents a stress characteristic. In this case the effective stresses on AB are perpendicular to BC and the safety may be determined by the equation of equilibrium in the direction BC. If the stress distribution can be sustained by the anchor block, and does not violate the failure criterion in the wedge of gravel or the underlying firm clay, the rupture figure (failure mode) is *statically admissible.*

Assuming that point A lies sufficiently far to the left in Fig. 11 b, there are no problems involved in indicating "reasonable" statically admissible reaction distributions. Hence, different locations of the contact point may correspond to different reasonable statically and kinematically admissible modes of failure with completely different factors of safety against sliding.

4.5.9 *Safety conditions*

The safety conditions are almost analogous to those of a centrally loaded column in that comparison can be made between different locations of the contact point A with different, apparently statically and kinematically admissible, deflections of the column (Fig. 12). Just as one would require normal safety against the most critical deflection pattern, it seems reasonable to require normal safety against sliding rupture, corresponding to the most critical reaction distribution. More favourable reaction distributions may be regarded as unstable, because a random, incalculable, deformation effect may trigger the most unfavourable reaction distribution possible.

The most unfavourable reaction distribution is obviously found when point A is located as far to the right as possible without the strengths of the gravel wedge, the firm clay or the anchor block itself being exceeded. It is immediately obvious that the greater the strength of the anchor block and the firm clay, the further can A be placed to the right i.e. the design values for the strengths of the reinforced concrete and the firm clay must be established as upper values, multiplied by partial coefficients > 1. The conditions in the gravel wedge are less clear, but in the actual case where the strength of the reinforced concrete proved to be the dominant factor for locating the contact point, the design value for the angle of friction of the gravel wedge – just like the shear strength of the soft clay zone – is in the usual way, divided by a partial coefficient > 1.0.

Shifting the contact point further to the right causing the anchor block to break, and reestablishing contact at the rear of the block, involves a secondary increase of the safety against failure. In the analogy with the centrally loaded column, this is equivalent to a restriction of the deflection to one of the sides, as shown at the bottom of Fig. 12. With such an unstable calculation model as our starting point, the size of the partial coefficients to be used is clearly open to discussion.

A finite-element analysis, carried out by the Storebælt group, showed that there is contact along the entire base of the block in the actual working state (the serviceability state) and, on this basis, it may be tempting to regard the rupture calculations as unreasonable. This is an almost classical soil mechanics dilemma, and one of the general answers is that most deliberations based on deformations are dubious and they should, as far as

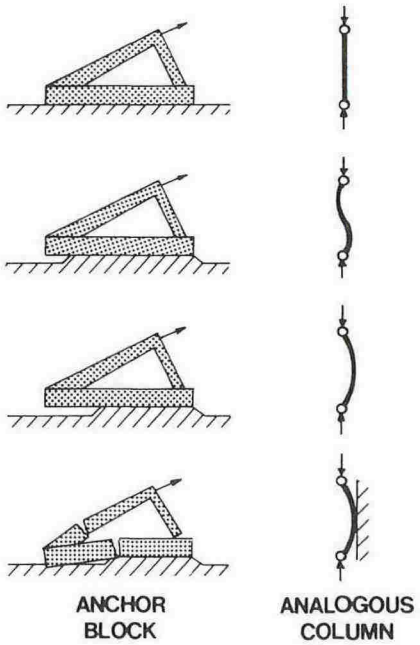

Fig. 12. Safety conditions of an anchor block vs. a centrally loaded column.

possible, not be used as the basis for evaluation of the safety against failure. For example, in the case in question, inadequate cleaning up of the excavation, with subsequent greater subsidence of the gravel wedge to the left of AB, could be catastrophic for the rupture safety based on deformation conditions. The same effect will be produced by "too good" cleaning up of the excavation or compaction of the gravel wedge to the right of AB in Fig. 11 b. The risk of basing the rupture safety on deformation considerations in this case may perhaps be most clearly demonstrated by imagining the construction to be "strengthened" by replacement of the gravel wedge to the right of AB by relatively incompressible concrete.

5. CONCLUSION

If I am to draw some conclusions from these rather diverse deliberations, they must be the following:

1. Limit state design represents a logical calculation principle. It is not in itself a radically new method compared to earlier design practice, but presents a clearer formulation of some widely accepted principles.
2. Within geotechnical and foundation engineering, even our best methods for obtaining the necessary geotechnical data, and our best calculation methods, are inadequate to the point that our factors of safety act, to some extent, as correction factors. For that very reason the best way of determining our design criteria is a combination of experience and back-calculation of successful foundation constructions. This also applies if limit state design is used with or without the partial coefficient system. If this fundamental fact is neglected, and design criteria are based on purely theoretical considerations, then there is a risk that the benefits of extensive practical experience gained within foundation engineering will be lost.
3. The failure analysis presented here clearly shows that, under Danish conditions, almost all foundation failures and errors are due to the lack of recognition, or underestimation, of the cause of the accident, such as the presence of critical deposits, critical water pressures, various forms of building activity etc. The failure analysis also clearly shows that decisive increase in safety against foundation failures cannot be achieved by minor adjustments of the safety factors to be used, but by more detailed and qualified geotechnical investigations, by carrying out geotechnical calculations at a sufficient level to take into account the critical factors in a reasonably correct way, and finally by performing thorough control studies, measurements and observations during construction of the foundation.

   The subdivision of constructions into various safety classes, with safety requirements varying in accordance with the consequences of failure, as has been done, or is planned, in many countries, focusses to such an extent on just such minor adjustments of partial coefficients, that the fact is obscured that it is, in practice, other features which are of decisive significance for the real safety of foundation constructions.
4. The partial coefficient system presents an elegant formulation of the safety requirements for traditional foundation constructions, but it is not a universally applicable system which can readily – with fixed values – be used for all foundation constructions. On the contrary, the examples presented above show that, for less conventional constructions, the effect of the partial coefficient system must be given full consideration in every single case. Also for this reason it seems obvious that an attempt to establish more or less fixed partial coefficients on the basis of abstract philosophical safety considerations would appear to be an attempt to kill the judgement – now returning to Peck's question.

6. CLOSING REMARKS

I must end by noting that I have been telling you about an anchor block which has been filed away, unbuilt, while you have the gravity platforms in the North Sea; that I have described some theoretical anchor problems which we have not yet had the resources to investigate in practice, while you may have been thinking of your advanced off-shore instrumentation; and I have presented a poorly-investigated, small landslide, while you can soon fill bookshelves with reports on thoroughly investigated, dramatic landslides.

This difference in proportions between our two countries was no doubt one of the reasons why Laurits flourished so well here in Norway. But I know that the friendly atmosphere of the NGI-family was at least as important for him. He himself contributed greatly to building up this friendly atmosphere, which you, as far as I can see, have maintained in the nicest way. Alone the faithfulness which is expressed by an arrangement such as this, reflects an attitude and human qualities which are heart-warming. I want to thank you for all the times, through the years, that I have been able to benefit from the inspiring and warm-hearted environment you, together with Laurits, have built up around geotechnical engineering in Norway; and thank you for once again, this evening, welcoming my wife and me into your company.

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