**Zeitschrift:** IABSE reports = Rapports AIPC = IVBH Berichte

**Band:** 65 (1992)

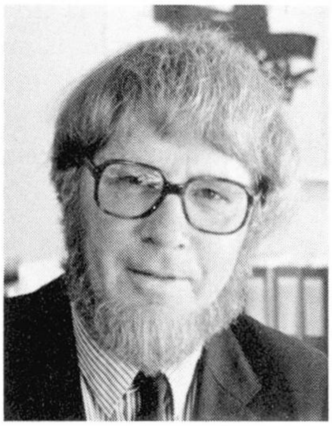
**Rubrik:** Eurocode 7: Foundations and geotechnical engineering

# EC 7: Geotechnical Code of Practice

## EC 7: Norme de calcul en géotechnique

## EC 7: ein geotechnisches Regelwerk

**Niels KREBS OVESEN**
Managing Director
Danish Geotechnical Institute
Lyngby, Denmark



Niels Krebs Ovesen, born 1934, received his PhD in Geotechnical Engineering from the Technical University of Denmark. For 18 years he served as a professor in geotechnical engineering at the Danish Engineering Academy. Niel Krebs Ovesen is now the managing director of the Danish Geotechnical Institute, a research and consultancy organisation under the Danish Academy of Technical Sciences.

**SUMMARY**

This paper gives a general description of the problems encountered in the introduction and use of codes of practice in geotechnical engineering. The use of the limit state design format and partial factors of safety in geotechnical engineering is discussed and the development which has taken place in Western Europe during the last decade towards the establishment of a common code of practice – Eurocode 7 – for the design of geotechnical structures is highlighted.

**RESUME**

Cet article décrit les problèmes rencontrés lorsque les normes de calcul ont été introduites et utilisées dans le domaine de la géotechnique. On discute aussi de l'application des états limites et des coefficients partiels de sécurité. De plus, on souligne les développements qui ont pris place en Europe de l'Ouest durant la dernière décennie en ce qui a trait à l'obtention d'une norme commune de calcul, soit l'Eurocode 7.

**ZUSAMMENFASSUNG**

Der Beitrag beschreibt auf allgemeine Weise die Probleme bei der Einführung und Anwendung eines Regelwerks für die Geotechnik. Insbesondere wird die Anwendbarkeit des Konzepts der Grenztragzustände und Teilsicherheitsbeiwerte diskutiert. Dabei wird die Entwicklung hervorgehoben, die während des letzten Jahrzehnts in Westeuropa mit dem Ziel eines einheitlichen Regelwerks – Eurocode 7 – für die Bemessung von Tragwerken der Geotechnik stattgefunden hat.

# 1. CODES AND STANDARDS

## 1.1 Risks in geotechnical engineering

Risk is an inherent part of all engineering works. According to the different types of works, different types of risk may be involved and there are also different ways in which such risks are evaluated and managed.

One type of engineering works is represented by the unique, complex and highly sophisticated geotechnical structures. Examples of such structures are the foundation for a nuclear power plant or an earth dam for a water reservoir with a depth of more than one hundred metres. During recent years extensive work has been performed in the field of reliability modelling and risk evaluation for such structures. However, the various tools developed for this purpose are still very complicated and time-consuming in their use. As a consequence, they are not at present subject to codification or standardization.

Another type of engineering works is represented by common and routine geotechnical structures, such as shallow or piled foundations for buildings, earth retaining structures, excavations, embankments and slopes. For such types of structures it is important that a safe design is ensured through a design process which is not too complicated or time-consuming. In quite a number of countries powerful tools for the design of this type of structures has been developed over a long time span in the form of codes of practice and standards.

The present paper concentrates on the use of codes of practice and standards for the design of such common and routine geotechnical structures. Special emphasis will be placed on the development which has taken place in Western Europe during the last decade towards the establishment of a common code of practice for the design of geotechnical structures - Eurocode 7.

## 1.2 Definitions

The words Code, Standard and Norm are often used rather casually. The International Standard Organisation, ISO (1986) gives the following terms in the various languages:

| | | |
|---|---|---|
| English | standard | code of practice |
| French | norme | code de bonne pratique |
| Russian | стандарт | свод правил |
| German | Norm | Anleitung für die Praxis |
| Spanish | norma | código de prática |
| Italian | norma | codice di pratica |
| Dutch | norm | praktijkrichtlijn |
| Swedish | standard | riktlinjer |
| Danish | standard | norm |

It may also be worth noting that the Oxford Advanced Learners's Dictionary of Current English defines a code as "a system of rules and principles that has been accepted by society or a class or group of people.

## 1.2 Standards as reference documents

In the present context the word Standard will refer to a document which, according to the ISO definition, is aimed at the achievement of an optimum degree of order in a given context. In geotechnical engineering standards are normally used in the form of testing standards or product standards. Over the years a large number of standards for laboratory and field testing in geotechnical engineering has been established by various national standard organizations such as AFNOR, ASTM, BSI and DIN.

Standards are useful tools for the engineering profession in being reference documents. They assist the designer in achieving an optimum degree of order and they enable the engineers to "speak the same language".

## 1.4 Codes as quality assurance documents

In the present context a Code of Practice describes recommended design practice by defining the requirements which are aimed at reaching a reasonable technical level of quality. The code requirements are normally expressed as functional requirements and they are based on scientific/technical principles. The codes of practice will normally avoid standardising certain methods of procedures of design and construction

Thus it is emphasized that a code of practice in its concept deviates from a standard. A code of practice aims at obtaining a specific technical level of quality while a standard aims at a specific degree of order in a given context.

It follows from these definitions that codes of practice are documents which are directly aimed at the geotechnical design process.

# 2. THE BASIC COMPONENTS OF A GEOTECHNICAL CODE

## 2.1 Design by calculations

A code of practice for geotechnical engineering comprises a set of provisions, compliance with which will ensure a reasonable technical quality for common and routine foundations and earth works. Generally speaking, the code provisions may be formulated in two different ways.

One type of provisions may be termed "prescriptive measures". They consist of advice or conventional and generally conservative details in the design and specification of control of materials, workmanship, protection and maintenance procedures. In geotechnical engineering presceiptive measures are often used to ensure durability to frost action and chemical or biological attack. They may sometimes also be used to avoid unnecessary calculation in very familiar design situations.

The other type of provisions will normally be formulated as design calculation procedures. There are several components in such calculation procedures and in figure 1 an attempt has been made to illustrate these components by means of an example.

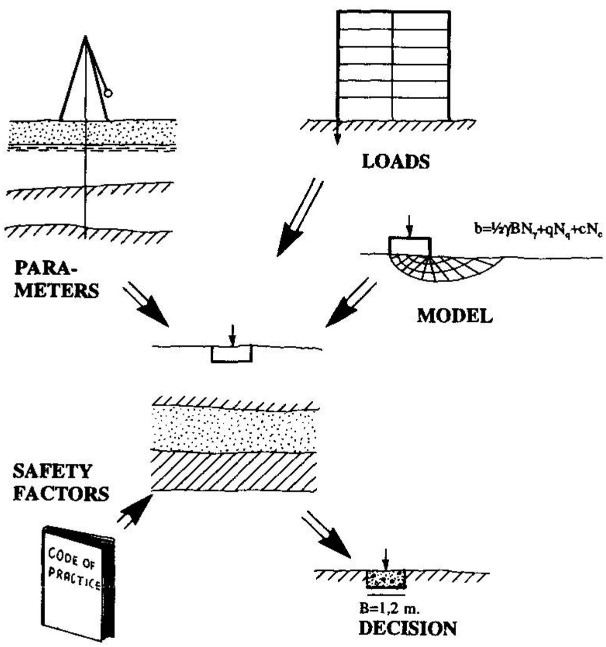



Figure 1. Components of geotechnical design

Figure 1 illustrates the problems facing the designer of a foundation of multi-storey concrete building. In order to design the footings of such a building, the geotechnical engineer may use a code of practice to determine the following four components of the design:

- Loads
- Soil parameters
- Calculation procedure
- Safety elements

## 2.2 The loads

The loads on the structure consist of the weight of the structure and live loads due to fittings and furnishings, persons, snow, wind, etc. Let us consider as an example the live load on the floors in office buildings. Investigations indicate that this live load in most office buildings will be in the actual range of .2 to .5 kN/m$^2$. For the design, however, a typical code may specify this live load to be assigned a so-called characteristic value of 2-3 kN/m$^2$ in design of buildings to be used for offices, schools, restaurants, etc. In modern terms, the characteristic value might be defined as a load which with a probability of 98% will not be exceeded within a period of one year. However, it is important to understand that there is not one specific answer to the question: Which live load on floors is the correct one to be used in the design of office buildings? The answer depends on the entirety of design in which the live load is goint to be used.

## 2.3 The soil parameters

The shear strength parameters of the soil in question will be determined either from field tests (e.g. vane tests to determine the undrained shear strength of clay), from element tests in the laboratory (e.g. triaxial tests to determine the angle of shearing resistance for sand), or from empirical relations between the shear strength parameters and the standard classification parameters. Let us consider as an example the use of triaxial tests to determine the angle of shearing resistance for sand. A couple of questions now arise: Which diameter and which height/diameter ratio should be used for the sample? Should rough or smooth surfaces of the top and bottom platens be used? Which cell pressure should be applied? Should the angle of shearing resistance be interpreted as the secant or tangent angle of the failure envelope in the Coulomb-Mohr diagram? etc., etc.

All these questions have to be answered in order to determine the test procedure and to evaluate the test results with regard to the angle of shearing resistance.

## 2.4 The calculation procedure

The calculation procedure used for the design of footings against failure is based normally on the plasticity theory. The bearing capacity for the footing is often determined from the Terzaghi bearing capacity formula. Even though this formula is widely accepted and applied, a large number of questions arise concerning the $N_\gamma$ value, the shape-, depth- and inclination factor(s) etc. A comparison by Malcharek and Smoltczyk (1981) for example has demonstrated that a range of numerical values between 8 and 20 can be found for the $N_\gamma$ value for $\varphi = 30°$ in the codes of practice in the - then - eight countries: Czechoslovakia, Denmark, Federal Republic of Germany, France, German Democratic Republic, Poland, USA, and USSR.

## 2.5 The safety elements

The safety factors to be applied to the bearing capacity problem may often be specified in a code of practice. Factors of total safety between two and three will normally be considered adequate. However, this also raises a number of problems. Should the safety factor be applied on load or on material strength? Should the same safety factor be used in an effective stress and in a total stress analysis? etc., etc.

## 2.6 A code: a tool for decisions

From the above discussion it appears that quite a number of questions have to be answered in order to design a footing. Each code of practice will answer the various questions in different ways.

It is thus important to understand that a code of practice at its very best represents a fine balance between the four components mentioned above. A code of practice can not be judged on the basis of an isolated evaluation of for example the calculation procedures that it recommends. These calculation methods can only be judged in combination with the safety factors, the loads and the shear strength parameters which, according to the code, are going to enter into the process of designing the geotechnical structure in question.

A code of practice is not "scientific" by nature. It does not represent "the truth" about the matter in question. It represents a tool by means of which decisions can be made relevant to the design of the geotechnical structure in question.

A good code proves its value if it works in practice; and that means if the right decisions are made by means of the code. Right decisions, again, mean designing structures which are sufficiently safe on one hand and which are economical on the other hand. In a perhaps too oversimplified manner, it could be said that a good code leads to a situation where only a few structures fail from time to time.

## 3. LIMIT STATES DESIGN

### 3.1 The development of Limit State Design

Before World War II codes of practice for foundation engineering were used only in a small number of countries. These codes were aimed at describing good engineering practice and they were not very systematic in their approach to design.

The postwar boom in the construction industry led to a wide-spread rethinking of the whole civil engineering design process. In the early fifties, for example, the Institution of Structural Engineers (1955) in the United Kingdom set up a committee to report on safety in structural design. In their report the committee noted that "the main body of evidence regarding the safety of a structure ... will usually take the form of design calculations" and they proposed that two particular ratios should dominate the discussion:

- "the ratio of the ultimate load to the appropriate working load, known as the ultimate load factor",

- "the ratio of the limiting load to the appropriate working load, known as the limiting load factor".

The "ultimate load" was identified as that causing collapes while the "limiting load" was intended to define the onset of "excessive elastic deflections, limits to which may be set by esthetic consideration or by some resulting interference with the proper use of the structure, (similar) permanent deflections, (and the) development of local defects, such as cracks....."

In 1956 Brinch Hansen used for the first time the words "limit design" in a geotechnical context. He described limit designs in the following way: "In the design of any structure two separate analyses should in principle be made: one for determining the safety against failure and another for determining the deformations under actual working conditions". Brinch Hansen (1956) linked the limit design concept closely to the concept of partial safety factors, and he introduced these two concepts in Danish foundation engineering practice.

During the 60's and 70's a number of European technical associations and committees initiated work on model codes for various building materials.

An early example of the result of this work is a British standard CP110, The Structural Use of Concrete from 1972. This code was coined in the terminology of Limit States. Any condition that a structure might attain, which contravened the basic requirement was designated a Limit State. The most important innovation in CP110 was the explicit use of probability theory in the selection of "characteristic" values of strength which - according to some notional or measured distribution - would be exceeded in at least 95% of standardised samples.

During the 70's and 80's developments similar to that in Denmark and the UK have taken place in many other countries.

In 1978 the Nordic Committee on Building Regulations (1978) issued a report containing "Recommendation for Loading and Safety Regulations for Structural Design" - NKB report No 36. This report laid down the basic principles for Limit States Design. It defines the ultimate and the serviceability limit states. It introduces a concept of safety class and control class. It gives rules for the combination of the various loads, and it also describes the system of partial factors of safety. Finally, the report gives information on the calibration of partial factors of safety.

The NKB report has had a pronounced effect on the codes of practice in Denmark and also in Sweden. During the first part of the 80's all Danish structural codes of practice were rewritten into the NKB format and as a result of this procedure a rather unique set of codes of practice for civil engineering works have been established in Denmark by Dansk Ingeniør Forening (1983 and 1984). All codes, whether they involve concrete, steel, timber, masonry or soil, are based on two basic codes: The Safety of Structures and The Load for the Design of Structures.

Recently Sweden has made a similar, systematic approach to establish a harmonised set of building codes, Boverket (1989).

The limit state concept is today widely accepted as a basis for codes of practice in structural engineering. From the very beginning of the work on the Eurocodes it was a foregone conclusion that the Eurocodes should be written in the limit state design format and that partial factors of safety should be used. It was also clear that a harmonised set of codes should be aimed at.

Consequently it was decided that also those parts of the Eurocodes which will be dealing with geotechnical aspects of design should be written in the limit state format with the use of partial factors of safety. In the early stages of the work this decision caused some concern among the European geotechnical profession - and to some degree still does - since very few European countries have had experience with geotechnical codes written in the limit state format.

Today, however, it looks as if the geotechnical profession have come to realise the advantages of establishing the geotechnical parts of the Eurocode system within the framework of a harmonised set of structural and geotechnical codes.

### 3.2 Geotechnical aspects of Limit States Design

In order to ensure an adequate technical quality of geotechnical structure, it is required that the structure as a whole and the various parts of the structure fulfil certain fundamental requirements of stability, stiffness etc. during construction and throughout the design life of the structure. In a code of practice the fundamental requirements are expressed in specific terms as performance criteria.

Whenever a geotechnical structure or part of the geotechnical structure fails to satisfy one of its performance criteria, it is said to have reached a "limit state". In a code based on "the limit state method" each limit state is considered separately in the design and its occurrence is either eliminated or shown to be sufficiently improbable.

In geotechnical design it is normal practice to consider the possible formation of a mechanism in the ground. However, it is also necessary to consider the possibility that serious damage could occur in the structure due to deformation in the ground without the mobilisation of a mechanism in the ground.

In geotechnical limit state design two main classes of limit states are consequently considered:

- An Ultimate Limit State at which
  - either a mechanism is formed in the ground
  - or a mechanism is formed in the structure or serve structural damage occurs due to movements in the ground.

- A Serviceability Limit State at which deformation in the gournd will couse loss of serviceability in the structure.

Relevant serviceability limit states include settlements which affect the appearance or efficient use of the structure or cause damage to finishes or non-structural elements or vibration which causes discomfort to people or damage to the content of the building.

Durability of the structure during its entire intended life span must be considered when selecting the design parameters. Durability should therefore not be considered a separate serviceability limit state. Inadequate durability may lead to the occurence of serviceability or ultimate limit states.

In practice it may often be difficult to know which type of limit state will govern the design, and it is therefore often necessary to investigate the ultimate as well as the serviceability limit state.

Whether the ultimate or the serviceability limit state will be decisive in any given case will depend on a number of factors, such as type of superstructure, type of soil as well as well as dimensions of the foundation and the load acting on it.

The sensitivity of the superstructure to settlements is of course very important. Both flexible and very rigid superstructures must be considered to be rather insensitive, because a flexible one can follow uneven settlements without servere structural damage whereas a rigid one will settle as a block. Examples of the first kind are steel oil tanks and of the second kind concrete silos.

The sensitive superstructures are those of medium rigidity, especially frame structure. Also the machinery to be placed on the structure will often determine the settlements which can be tolerated. An example of this is a turbine foundation.

With regard to the type of soil it is evident that in dense or medium sand where the settlements are usually small, the settlement criterion will seldom be decisive. In contrast, for foundations on soft clay the settlements will usually govern the design.

Brinch Hansen (1967) has given the following indication as to where the two main types of limit states may become decisive:

| **Ultimate** | **Serviceability** |
|---|---|
| Small footings | Large footings |
| Small unit loads | Large Unit loads |
| Dense or firm soils | Loose or soft soils |
| Insensitive superstructures | Sensitive superstructures |

K. Mortensen (1983) has given a critical review of the use of Limit States Design in geotechnical engineering.

## 4. SAFETY MARGINS

### 4.1 Traditional practice

One important role of a code of practice is to indicate how risks shall be dealt with through the introduction of adequate safety margins. In traditional geotechnical practice there are many different ways in which safety factors are defined and introduced. Very often the way in which the safety margin is introduced depends upon the type of geotechnical problem.

As regards shallow foundation many building codes have until recently used the concept of "allowable foundation pressures" for different kinds of soil. This practice, however, is very unsatisfactory for a rational designer since he will not know the magnitude of the safety factor implied by the indicated "allowable foundation pressures".

In earth pressure problems the traditional practice has been to divide passive earth pressures by a safety factor whereas no safety factor is applied to the active earth pressures or to at-rest pressures.

In slope stability problems the old definition of the safety factor as a ratio between stabilising and overturning moments is still being used. A more widely accepted practice today is, however, to define the safety factor as a ratio between average shear strength and average shear stress in the critical failure surface.

Through the development of the limit state concept it was realised that there are a number of reasons for including a safety margin under many aspects of the design process:

- Soil strength in the field may have both a systematic and a random difference from that assumed in design.

- Geometrical parameters, expecially the interfaces between strata, water levels and ground levels, may differ from those assumed in design.

- Simplified assumptions and equations used to evaluate bearing capacities etc. may lead to systematic or random errors.

- Simplified representatives of loads used for design may introduce systematic or random errors and loads may be applied in a way which is different from the one assumed in the design.

Also the potential consequences of a failure may affect the level of safety selected. The same applies to the type of failure.

It was considerations of this type that led to the development of the concept of partial factors of safety. The concept has been developed in close connection with the development of the limit state concept. The "purest" from the system of partial safety factors has been developed in Europe, and in Denmark almost 40 years of experience with this system have now been gained in geotechnical engineering. In the European format the partial factors are introduced on the load as well as on the strength side.

A different concept has been adopted in North America where the development has especially been centered around the Ontario Highway Bridge Design Code (1983). Here the safety margin is introduced by so-called Load Factors on the load side in combination with a Resistance Factor on the material resistance side.

## 4.2 Partial factors of safety for geotechnical design

In 1953 Brinch Hansen proposed to introduce the principle of partial factors of safety for the design of foundation and earth retaining structures. In Brinch Hansen's concept the partial factors of safety were closely related to the concept of Limit States Design, and it may be worth quoting the following from Brinch Hansen (1956):

> " In the design of any structure two separate analyses should in principle be made: One for determining the safety against failure, and another for determining the deformations under actual working conditions. The failure analysis can in practice be made in three different ways:
>
> Method No 1 ..........
>
> Method No 2 ..........
>
> Method No 3
>
> Multiplying the prescribed loads with certain (partial) safety factors and limiting the corresponding maximum stress to the limit strength of the material divided by another (partial) safety factor.
>
> The third method must be the logical one because the safety factor should be applied to any quantity which is not known accurately and this implies the loads as well as the limit strengths of the material.
>
> In soil mechanics certain loads, earth pressures and foundation pressures depend on the shear strength of the soil and on the deformation of the structure. As shown by the author, a logical consequence of this fact is that a consistent system of safety factors can only be devised by the third method mentioned above."

The principle of partial safety factors was proposed by the Danish structural engineer, A. J. Moe, as early as 1927. It was published internationally by A. J. Moe in 1936. It was partly introduced in the Danish Code of Parctice for Concrete Design in 1949. Brinch Hansen applied the principle to geotechnical structures in his book on "Earth Pressure Calculation" (1953), and he proposed numerical values for the various partial safety factors. The system was rapidly accepted in foundation engineering practice in Denmark, and from around 1955 virtually all foundations and retaining structures in Denmark were designed according to the new principle. In engineering schools the system was taught in all courses in soil mechanics and foundation engineering from 1955.

The numerical values of the factors proposed by Brinch Hansen have undergone minor changes during the past 35 years. In the following a description of the method will be given as it is used in the Danish Code of Practice for Foundation Engineering today, Dansk Ingeniør Forening (1984).

Design values (indices d) of loads are found by multiplying the corresponding characteristic values (indices c) by the respective partial factors.

$$G_d = \gamma_g \cdot G_c \qquad\qquad Q_d = \gamma_q \cdot Q_c \tag{1}$$

Where g refers to dead loads, and q refers to variable loads.

In most foundation engineering problems the dead loads are known with considerable accuracy. Small variations may occur in unit weights and dimensions, and for this reason it might be considered appropriate to use a partial factor of e.g. 1.05. However, not all dead loads will have unfavourable effects, and for those which are favourable, it would be unsafe to mulitply by 1.05. On the contrary, these loads should be divided by 1.05. In this way things can get rather complicated, and it is really not worthwhile to accept such complications for the sake of a margin as small as 5%, taking into consideration all the uncertainties involved in soil strength etc. Consequently, according to Danish tradition, all dead loads of structures and of soils are given the partial coefficient $\gamma_g = 1.0$.

Hydrostatic water pressures are known with the same accuracy as dead loads when the water table is given and for this reason it is logical to use a partial factor of unity for water pressures. Moreover, any other value would lead to similar complications as for dead loads because the up-lift force on a soil mass or a structural element is part of its effective weight.

For a variable load the partial factor $\gamma_q = 1.3$ is normally used in Danish practice. However, in the case of more than one variable load acting, the various loads are normally combined in order to take account of the fact that it is very unlikely that all variable loads will act with their full design value at the same time.

Design values of strength parameters are found by dividing the corresponding characteristic value by the respective partial factor:

$$\tan \varphi_d = \frac{\tan \varphi}{\gamma_\varphi} \qquad\qquad c_d = \frac{c}{\gamma_c} \tag{2}$$

According to the Danish tradition a value $\gamma_\varphi = 1.2$ is used. Similarly, a partial factor $\gamma_c = 1.5$ is used in the case of stability or earth pressure problems, while $\gamma_c = 1.8$ is used in the case of bearing capacity problems of footings.

For the bearing capacity of piles or anchors $\gamma_b = 2.0$ is used to obtain the design balue of the bearing capacity in the case when the bearing capacity has been found by a geostatic calculation or from a pile driving formula. In the case when load tests have been performed, a value $\gamma_b = 1.4$ is used for the piles tested while $\gamma_b = 1.6$ is used for the other piles. The partial factors given for piles are valid only for the bearing capacity determined by the strength of the soil and not for the pile material.

A special problem exists in geotechnical engineering concerning the selection of characteristic properties for the strength parameters $c_c$ and $\varphi_c$ of soil and rock. The ground displays a large range of material behaviour and many different testing techniques are appropriate in order to measure or infer the required material parametres. However, very often it is not possible to obtain a sufficiently large number of test results to derive a characteristic value using formal statistical methods. In geotechnical engineering charecteristic values of soil and rock parameters are therefore normally based on careful assessment of the range of values which might govern the field behaviour during the lifetime of the structure. This assessment must take account of geotechnical and other background information, such as relevant data of previous projects and the results of field and laboratory measurements. For parameters for which the relevant values in the field are well established with little uncertainty, the characteristic value may be taken as the best estimate of the value in the field. Where there is greater uncertainty, the characteristic value is somewhat more conservative.

According to the principles of Limit States Design, the design criterion is simply to design for equilibrium in the design limit state of failure. The design criterion could be expressed in the following way:

$$R_d \geq s_d \tag{3}$$

$S_d$ is the design load effect calculated on the basis of the principles underlaying equations (1). The design resistance effect $R_d$ which in the case of the design of a footing is the design ultimate bearing capacity of the footing, is calculated on the basis of the design soil parameters defined by equations (2).

The use of partial factors of safety requires special attention with regard to the derivation of design values of earth pressures. The magnitude and direction of earth pressure depends on the material properties of the soil. As a consequence, the Danish tradition has been to calculate design values of earth pressures by introducing partial factors of safety for the material properties and not for the earth pressures as such.

It should finally be stressed that the information given above on the use of partial factors of safety relates exclusively to the ultimate limit states. In the case of serviceability limit states it is normal Danish practice to use unity for the partial factors on the material side, and at the same time use partial factors equal to unity on the loads, which should then be taken in their frequent combination.

## 5. SERVICEABILITY CONSTRAINTS

In geotechnical design a constraint is an acceptable limiting value for a particular deformation in order to satisfy the limit state requirements. It may be beneficial to distinguish between two different types of structures with respect to constraints.

The first type of geotechnical structure may be represented by a shallow or piled foundation supporting a multi-storey building. For such a structure it is, of course, important to prevent a ultimate limit state at which a mechanism is formed in the ground, as well as a serviceability limit state at which deformation in the ground will cause loss of serveceability in the super-structure. However, it is also important to consider a limit state at which a mechanism is formed in the supported structure, or server structural damage occurs due to movements in the ground.

Another type of geotechnical structure is represented by for example an earth dam. In this case there is no "supported" structure to be considered and the various limit states to be prevented in the geotechnical design concerns the ground as such.

In foundation design of supported structures constraints will normally be expressed in the form of allowable total and differential settlements, rotations, relative rotations (differential settlement/span), horizontal displacements and vibration amplitudes and accelerations. There may also be constraints concerning structural elements forming part of the foundation, e.g. in the form of allowable cracking width for reinforced concrete basement walls, piles, etc.

The code-writer's problem of establishing constraints is part of a much wider problem of structural interaction. As indicated by Burland, Broms and de Mello (1977), little progress has been made in this global problem for a number of reasons. Some of these are:

- Serviceability is very subjective and depends both on the function of the building, the reaction of the user and owner, and economic factors such as value, insurrance cover, and the importance of prime cost.

- Buildings vary from one another in such features as purpose, structural form, building materials, construction details and finishes.

- Buildings, including foundations, seldom perform as designed because of the many simplifying assumptions that have to be made regarding the properties of the ground and the supported structure.

For these reasons it is obvious that it is difficult for the code-writer to specify acceptable values of the constraints with a high degree of confidence.

The geotechnical engineer has a responsibility to design an economic foundation which will ensure that the supported structure fulfils its function. In doing so he must not only evaluate the behaviour of the ground but he also needs to know how the building will respond to deformations and what the consequences of such deformations will be to its function. Close contact between the structural and the geotechnical engineer during the design process is an essential element in establishing structural and geotechnical serviceability constraints.

The best known study leading to recommendations on allowable differential settlements of structures is that of Skempton and MacDonald (1956), and the constraints given in most of today's Codes of Practice are still based largely on this work. It was concluded that the limiting values of relative rotations to cause cracking in walls and partitions are 1/300 and that values in excess of 1/500 should be avoided. The limiting value of relative rotations to cause damage in the superstructure is 1/150. Subsequently, Bjerrum (1963) supplemented these recommendations.

The above guides concerning limiting settlements apply to simple routine structure. Many Codes of Practice contain similar simple guides. These guides should not, however, be applied to buildings or structures which are out of the ordinary or for which the loading intensety is markly non-uniform.

The above-mentioned work by Burland, Broms and de Mello (1977) still provides the best guidelines as to allowable settlements for geotechnical structures.

# 6. EUROCODE 7 GEOTECHNICAL DESIGN

## 6.1 Background

In 1980 an agreement was reached between the Commission of the European Communities and the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), according to which the society should undertake the drafting of a model code to be adopted as Eurocode 7. The ISSMFE established an ad hoc committee for this task which produced a draft model for Eurocode 7 (1987).

In 1988 the Commission of the European Communities established a small Drafting Panel which was given the task of redrafting the 1987 version of the model code into a Eurocode format.

During 1989 the work on the Eurocodes were transferred to the European Committee for Standardisation, CEN, and a new Technical Committee CEN/TC 250 "Structural Eurocodes" was created. TC 250 met for the first time in May 1990 and established a number of Sub-Committees - among them SC7 being responsible for Eurocode 7 Geotechnical Design.

## 6.2 Scope and parts

The Subcommitte SC7 responsible for Eurocode 7 Geotechnical Design met for the first time in December 1990 in Rotterdam to prepare a work program. N. Krebs Ovesen serves as chairman for SC7, B. Simpson as vice chariman and the Netherlands Normalisatie-Institut, NNI as administrative secretariat for SC7.

The scope of work for SC7 is to establish and maintain European standards in the field of structural design rules for building and civil engineering works covering geotechnical applications, including general design rules, related laboratory testing and field testing and sampling, and additional design rules for specialized geotechnical elements and structures.

SC7 has decided to divide Eurocode 7 into four parts as follow:

**Part 1 Geotechnical design. General design rules**
Standardization of general geotechnical design rules for building and civil engineering works.

**Part 2 Geotechnical design. Standards for laboratory testing**
Identification of existing and development of new standards for laboratory testing on soil and rock materials.

**Part 3 Geotechnical design. Standards for field testing and sampling**
Identification of existing and development of new standards for field testing and sampling of soil and rock.

**Part 4 Geotechnical design. Rules for specialized elements and structures**
Standardization of additional design rules for specialized elements and structures taking into account the general design rules specified in Part 1.

SC7 has also resolved to divide parts 2, 3, and 4 into several subparts in order to achieve a better division of publications of documents.

For each of the parts 1, 2, and 3 SC7 appointed a Project Team to elaborate on a detailed working program and to draft the necessary documents. The following project teams were established:

Part 1 N.Krebs Ovesen, Lyngby (Convener), F.Baguelin, Paris, E.J.L.Maranha das Neves, Lisboa, B.Simpson, London, U.Smoltczyk, Böblingen, T.L.L.Orr, Dublin (Technical secr.)

Part 2 T.Berre, Oslo (Convener), A.Anagnostopoulos, Athens, K.Head, Cobham, E.Lousberg, Louvain la Neuve, B.Schuppener, Hamburg

Part 3 W.J.Heijnen, Delft (Convener), U.Bergdahl, Linköping, R.Frank, Paris, M.Jamiolkowski, Torino

In the following a detailed description of the work on the various parts is given.

### 6.3 Part 1. General design rules

Part 1 of Eurocode 7 will contain the following chapters:

- Introduction
- Basis of design
- Geotechnical Categories
- Geotechnical data
- Fill, dewatering and ground improvement
- Spread foundations
- Pile foundations
- Retaining structures
- Embankments and slopes
- Supervision of construction, monitoring and maintenance

Like all Eurocodes Part 1 of Eurocode 7 will be written in the Limit States Design format and verification of safety and serviceability will be based on the use of partial factors of safety. However, since only limited geotechnical experience with the partial safety factor format has been gained in Europe, a good deal of calibration is needed before definitive numerical values of the partial factors of safety can be established. In the text of Part 1 of Eurocode 7 distinction will be made between "principles" and "application rules" - as in the other Eurocodes.

Even though Part 1 is being prepared as part of a unified system of codes for the various building materials, specific features distinguish Part 1 of Eurocode 7 from the other Eurocodes. For conventional structural design it is a common feature that much of the safety evaluation is centered around calculation models. In contrast, in geotechnical design much more effort is devoted to identification and characterization of the relevant ground masses and the processes taking place in the ground than to apply sophisticated calculation models.

As a consequence a great variety of geomechanical models is needed whereas the feasibility of detailed standardization and codified calculation procedures is limited and frequently not warranted. The Part 1 Project Team has also observed considerable differences in the professional geotechnical parctice in the various European countries. In the opinion of the Project Team it is a consequence of these differences that Eurocode 7 should be a much less detailed code, especially in relation to geotechnical calculation procedures than the Eurocodes covering sush man-made materials as steel and concrete.

Some features of Eurocode 7 deserves further mentioning.

In Part 1 of Eurocode 7 three "geotechnical categories" are defined in order to establish minimum requirements for the extent and quality of geotechnical investigation, calculations and construction control checks. The following factors shall be taken into consideration when determining which geotechnical category is appropriate for each particular design situation:

- nature and size of the structure and its elements, including any special functional requirement,
- special conditions with regard to its surroundings (neighbouring structures, traffic, utilities, vegetation, hazardous chemicals, etc.),
- ground conditions,
- groundwater situation,
- regional seismicity,
- influence of the environment (hydrology, surface water, subsidence, seasonal changes of moisture, etc.).

Geotechnical Category 1 includes small and relatively simple structures for which it is possible to ensure that the functional reguirements will be satisfied on the basis of experience and qualitative geotechnical investigations and with no risk for property and life.

Geotechnical Category 2 includes structures for which quantitative geotechnical data and analyses are necessary to ensure that the functional requirements will be satisfied but for which conventional procedures of design and construction may be used.

Geotechnical Category 3 includes very large og usual structures, involving abnormal risks or unusual or exceptionally difficult ground or loading conditions and structures in highly seismic areas. Eurocode 7's specification for Geotechnical Category 2 forms the lover limits for the extent and quality of the necessary investigations and calculations but apart from this no detailed code reguirements have been formulated for Geotechnical Category 3.

A preliminary classification of a structure according to geotechnical category must be performed prior to the geotechnical investigations. The category may be changed eventually; it is important, however, that it remains well defined throughout the design and construction control process.

In geotechnical design previous experience of the construction and performance of similar structures in similar conditions are frequently quoted. Therefore the concept of "comparable experience" has been introduced in Part 1 of Eurocode 7. This term refers to documented or other clearly established information related to the structure being considered in design, involving the same ground types and for which similar geotechnical behaviour is expected.

One special problem has been identified during the work on Eurocode 1 Part 1. In geotechnical design it is normally difficult to distinguish between favorable and unfavorable effects of the weight of the ground - see section 4.2. For this reason it would be logical to use a partial factor of safety equal to unity for dead loads in geotechnical design. However, this could create problems in that a partial factor $\gamma_g = 1.35$ has been chosen for dead loads in structural design. In order to achieve a harmonised set of codes for structural as well as for geotechnical design it would be practical to use the same mumerical value of the partial factor on dead loads. There is no easy solution to this problem which at present is under close consideration by SC 7.

Two other fundamental problem areas also have been identified during the work on Eurocode 1 Part 1. One area concerns the problems of ground structure interaction, which will be dealt with in a following paper by B. Simpson. Another area concerns the problem of defining characteristic ground properties for geotechnical design. This problem area is dealt with in a following paper by F. Baguelin.

## 6.4 Parts 2 and 3. Standards for laboratory and field testing and sampling

According to the agreement between the Commission and CEN specifications for the construction materials and products used and the methods for testing their performance will not be included in the Eurocodes. However, the agreement states that specifications for laboratory and field investigation methods regarding the ground parameters necessary for design of foundations, retaining structures, and earth works will be included.

If the whole range of geotechnical laboratory and field tests are to be covered, an enormous task lies ahead of the project teams. It may therefore be foreseen that the work will be kept at a minimum by adapting existing national standards as European standards as far as possible. The main amendments that may be needed will be to make them less restrictive on dimension sizes and allow some alternative techniques. The project teams may decide not to aim for a uniform style throughout the standards. If they try to harmonize the style they will be committed to rewriting and therefore debating every word. This will be very time-consuming and could be a wasteful use of what financial resources CEN is able to provide.

It has been proposed to give high priority to the following standards for geotechnical laboratory tests:

- Moisture content
- Dry unit weight
- Specific gravity of solid particles
- Particle size distribution by wet sieving, dry sieving and hydrometer method
- Attenberg Limits
- Triaxial test
- Oedometer test

It has also been proposed to give high priority to standards for the following geotechnical field tests:

- Cone penetration test (CPT)
- Standard penetration test (STP)
- Pressuremeter test
- Dynamic penetration test (DPT)
- Pore pressure measurement
- Permeability test
- Vane test.

## 6.5 Part 4. Rules for specialized elements and structures

It should be noted that this part will deal with additional design rules for specialized elements and structures taking into account the general design rules specified in Part 1. The Eurocodes will cover execution and control only to the extent that is necessary to indicate the quality of the construction material and products and the standard of workmanship on site needed to comply with the assumptions of the design rules. It will consequently be a difficult task for the project teams to define the content of Part 4 and to determine to what detail execution and control should be covered.

Initial discussions on the topics to be treated as special elements in Part 4 of Eurocode 7 have indicated that high priority should be given to:

- bored piles, driven piles, anchors, diaphragm walls, grouting, sheet pile walls, vibroflotation, vibrocompaction, stone columns, dewatering, cut-off walls, reinforced ground including soil nailing, and mini piles.

Discussions have also indicated that low priority should be given to:

- tunnels and underground structures, marine structures, bridges, dams, and pavements.

The Eurocodes are essentially dealing with design rules. During the past 2-3 years the European foundation contractors have taken a strong interest in establishing standards covering the practical aspects of foundation work. This interest has been strongly voiced by the European Federation for Foundation Contractors and has lead to the establishment in February 1992 of a new CEN Technical Committee TC 288 on "Execution of Geotechnical Works".

TC 288 is chaired by dr Manfred Stocker, Germany and its technical secretariat is provided by Association Française de Normalisation, AFNOR.

TC 288 will attempt to standardise the execution procedures for geotechnical works (including the testing and control methods of the procedures) and of the required material properties. Highest priority will be given to work on bored piles, anchors, and diaphragm walls while work on driven piles, grouting, and sheet piles will come next.

Close cooperation will be maintained between TC 288 and TC 250/SC7 in order to avoid overlapping in the work.

## 6.6 Status by May 1992 for Eurocode 7

By May 1992 a first draft of all ten chapters of Eurocode 7 Part 1 has been finalised by the Project Team and the draft has been distributed to all CEN Member Organisations for comments. During the fall of 1992 the Project Team will subject these comments to discussions and decisions taking as far as possible the comments into account. The final draft of Part 1 will then be submitted to SC 7 for decision in early 1993. After formal approval by SC 7 Eurocode 7 Part 1 will then be issued as ENV for experimental use.

Due to lack of funding no progress is at present being made concerning the work on Eurocode 7 Part 2, 3 and 4.

## 7. CONCLUDING REMARKS

The foregoing discussion suggest that Codes and Standards will play an increasing role in the future as regulators for the geotechnical profession. They will serve as important professional reference documents for quality assurance and in pormotion of the free movement of goods and services at the international level.

The following is an attempt to draw some general conclusions on the use of Codes and Standards:

- It is important to differentiate in concept between a Code and a Standard. A Standard is a reference document that assists the designer in achieving an optimum degree of order in a given context. A Code of Practice aims at obtaining a specific technical level of quality. In other words, a code of Practice aims at setting professional standards.

- A Code of Practice represents, at its very best, a fine balance between the four components entering into the design process: Material parameters, loads, calculation methods and sefety elements. A Code of Practice is not "scientific" by nature. It represents a tool for making rational design decisions.

- The Limit States Concept represents a logical design principle. It is not in itself a radically new method compared to earlier design practice, but it represents a clear formulation of some widely accepted principles.

- The ground displays a far greater range of material properties and of heterogeneity than do manufactured materials such as steel and concrete. Even the best methods for obtaining the necessary geotechnical data and the most reliable calculation methods are therefore inadequate to the point that the factors of safety act, to some extent, as correction factors. For that very reason, the best way of determining design criteria to be used in Codes of Parctice is a combination of experience and back-calculation of successful geotechnical structures.

- The system of partial factors of safety is today widely used in Codes of Practice for structural design. During the years to come it will find increasing use in geotechnical engineering for the evaluation of the risk of failure and collapse. It may represent a useful tool for the design of traditional and routine geotechnical structures but it is not a universally applicable system which can readily be used with fixed numerical values for all geotechnical structures.

- The problem of establishing constraints for serviceability limit states for geotechnical structures is a problem of structural interaction. It is difficult for the code-writer to specify acceptable values of such constraints with a high degree of confidence. Close contact between the structural and the geotechnical engineer during the design process is an essential element in establishing such constraints.

## REFERENCES

Bjerrum, L. (1963). Discussion, Proc., ECSMFF, vol.2, p.135. Wiesbaden.
Boverket (1989). New Building Regulations (in Swedish, available in English from 1992), Allmanna Forlaget AB, Stockholm.
Brinch Hansen, J. (1953). Earth Pressure Calculation. The Danish Technical Press, Copenhagen.
Brinch Hansen, J. (1956). Limit design and safety factors in soil mechanics (in Danish with an English summary). Bulletin No 1, Danish Geotechnical Institute, Copenhagen.
Brinch Hansen, J. (1967). The philosophy of foundation design, design criteria, safety factors and settlement limits. Proceedings, Sumposium on Bearing Capacity and Settlement of Foundations, pp.9-13. Duke University, Durham, North Carolina.
Burland, J.B., Broms, B.B. & de Mello, V.F.B. (1977). Behaviour of Foundations and Structures, Proc. 9th ICSMFE, vol.2, p.495. Tokyo.
Dansk Ingeniørforening (1983 and 1984). DS 409 The Safety of Structures. DS 410 Loads for the Design of Structures. DS 4ll Structural Use of Concrete. DS 412 Structural Use of Steel. DS 413 Structural Use of Timber. DS 414 Structural Use of Masonry. DS 415 Foundation Engineering. (All in English). The Danish Technical Press, Copenhagen.
ISO (1986). Guide 2, General terms and their definitions concerning standardization and related activities, The International Standard Organization.
Institution of Structural Engineers (1955). Report on Structural Safety. The Structural Engineer, 33, May, pp.141-149. London.
Malcharek, K. & Smoltczyk, U. (1981). Vergleich Nationaler Richtlinien für Berechning von Fundamenten. Baugrundinstitut, Mitteiling 16, Stuttgart.
Mortensen, K. (1983). Is Limit State Design a Judgement Killer? Danish Geotechnical Institute, Bulletin No 35, Lyngby.
Nordic Committee on Building Regulations (1978). Recommendation for Loading and Safety Regulations for Structural Design. NKB report No 36.
Ontario Highway Department (1983). Ontario Highway Bridge Design Code, Ministry of Transportation and Communication, Highway Eng. Div., Toronto.
Skempton, A.W. & MacDonald, D.H. (1956). Allowable Settlement of Buildings, Proc. Inst. Civ. Eng., part 3, vol.5, pp.727. London.

# EC 7: Characteristic Values for Geotechnical Parameters

# EC 7: Valeurs caractéristiques des paramètres géotechniques

# EC 7: Charakteristische Werte geotechnischer Parameter

**François BAGUELIN**
Scientific Director
TERRASOL Cons. Eng.
Montreuil, France

François Baguelin, born in 1942, is Chief-Engineer of Ponts-et-Chaussées. He worked with Laboratoire Central des Ponts-et-Chaussées from 1967 to 1990, where he conducted research on foundations and in situ soil testing, and was responsible for geotechnical research in the Central and Regional Laboratories. He joined Terrasol, Consulting Engineers in Geotechnics, in 1990.

## SUMMARY

Determining characteristic values for geotechnical parameters is an essential, but difficult task. It cannot be simply the result of a statistical analysis. It must take account of the scatter, but also of the geological context and the mode of interaction between the structure and the ground which is considered.

## RESUME

La détermination des valeurs caractéristiques des paramètres géotechniques est une tâche essentielle, mais délicate. Elle ne peut être simplement le résultat d'une analyse statistique. Elle doit prendre en compte la dispersion, mais aussi le contexte géologique et le mode d'interaction entre l'ouvrage et le terrain considéré.

## ZUSAMMENFASSUNG

Die Bestimmung von charakteristischen Werten geotechnischer Parameter ist eine grundlegende aber auch schwierige Aufgabe. Sie kann nicht einfach das Ergebnis einer statistischen Analyse sein. Es müssen sowohl die Streuung als auch der geologische Zusammenhang und die Art der Wechselwirkung zwischen Boden und Bauwerk berücksichtigt werden.

## 1. DEFINITION AND ROLE OF CHARACTERISTIC VALUES.

### 1.1 Eurocode 1 "Basis of design"

The April 92 draft [1] gives the following definitions:
*"The material properties and their statistical variation should generally be obtained from tests on appropriate test specimens. The tests should be based on random samples which are representative with regard to the population under consideration.*
*A material property is represented by its characteristic value, $f_k$, which in general corresponds to an a priori specified fractile of the assumed statiscal distribution of the particular property in the supply produced within the scope of the relevant material standard."*

The role of the characteristic values is to be a basis for the design values to be used in the verification of limit states. A material factor $\gamma_m$ is used to reduce the characteristic value:
- in serviceability limit states, $\gamma_m = 1$ .
- in ultimate limit states, $\gamma_m$ is usually larger than 1 .

Thus, it is important to notice that the characteristic values are the design values in the verification of serviceability limit states, i.e. they must be somewhat conservative.

### 1.2 Eurocode 7 "Geotechnical design"

In the November 1989 draft [2], the application of the concept of characteristic value to soils or rocks is dealt with in paragraph 2.2.5 "material properties" of chapter 2 "Basis of design", clauses 2 to 4. With the same role in mind as in Eurocode 1, the determination of characteristic values has to be adapted to the particular situation of soils and rocks, which is quite different from the situation of manufactured materials such as steel and concrete.

Clause 3 indicates that *"the assessment shall take account of geological and other background information, such as relevant data from previous projects, together with the results of laboratory and field measurements."*
Clause 4 gives an indication of the degree of conservatism which should be attached to a characteristic value:
*"Characteristic values shall be selected with the intention that the probability of a more unfavourable value governing the occurrence of a limit state is not greater than 5%."*

But in the 'application rule' of this clause 4, the difficulty of using statistics with soils or rocks is pointed out:
*"It might sometimes be helpful to use statiscal methods. However, it is emphasized that this will rarely lead directly to characteristic values since these depend on an assessment of the field situation. The choice of characteristic values is not dependent on the severity of the limit state under consideration. However, the choice is often dependent*

*on the mechanism or mode of deformation being considered."*

During the consultations on the present draft of Eurocode 7, concern has been expressed about this definition of the characteristic value by the 5% risk and its possible legal implications. The drafting group has maintained the present text with two basic ideas in mind:
1) it is necessary to give an explicit definition of the characteristic value for soils and rocks, and therefore it is necessary to specify the fractile (or risk) which is mentionned in the general definition of Eurocode 1.
2) this statistical reference is given as a goal, not as a requirement for application, since statistical methods are rarely applicable in geotechnics.

The peculiarities, and difficulties, in assessing characteristic values for soils and rocks are examined below, with two main aspects being considered: 1) the soil or rock variability and the geological context, 2) the mechanism of interaction.

## 2. VARIABILITY AND GEOLOGICAL CONTEXT

### 2.1 Identifying homogeneous zones

In a statistical analysis, it is necessary to identify an homogeneous population before any attempt to determine its distribution law and to quantify the parameters of this distribution law.

This first task corresponds in geotechnical engineering to one of the main goals of the ground survey, which is identifying the natures and the extents, both in the vertical and horizontal directions, of the various soils and rocks. This has to be considered on a large scale (geological layers or formations, geological accidents) as well as on a small scale (e.g. seams).

In general, much effort is devoted to the detection of possible weak zones, due to some geological, local accident, for instance a fossile, tributary river in a main valley or in a slope, or a fault in a bedrock, or a deeper weathered zone. Cheap, quick investigation tools (geophysical methods, penetration tests) are often used for this purpose, while sophisticated, mechanical tests, which are a direct measurement of strength or deformation modulus, are performed in a limited number.

### 2.2 Using the statistics for soils and rocks

Once the danger of a possible, unknown weak zone has been reasonably kept away, the geotechnical engineer has to deal with 'homogeneous' soil or rock zones for which he has to select 'characteristic' values of strength or deformation modulus. However the use of statistics will

be generally limited, because :
1) the number of measurements will not generally be sufficient.
2) the application of statistics to ground is still in a infantile stage.

It is now recognized that the usual Gaussian law cannot be simply applied to soils, as for manufactured materials. Instead, the spatial variations have to be investigated, and researchers have shown that, for sedimentary soils, the auto-correlation distances are different in the vertical and horizontal directions.

To counter-balance these two defects: limited number of data, difficulty of applying statistics, the geotechnical engineer may draw upon two other sources to arrive at reasonable characteristic values:
1) general correlations on soil or rock properties; in particular, he may then be able to use the results of the quick, simple tests performed during the survey to detect the weak zones.
2) comparable experience, i.e. the experience gained in other sites on similar soils or rocks.

## 3. MECHANISMS OF INTERACTION

The soil or rock behaviour is complex: for instance, the strength of soils may depend on the type of loading and on the size of the loading. This is true for soil tests as well as for the structures.

Hence the need for calibration of design methods used in geotechnical engineering.

This also affects the choice of the characteristic values. Examples are given in the application rule of clause 4 : because of the size effect, the variability in the soil strength will affect more the ultimate limit bearing capacity of an end-bearing pile than the one of a friction pile in the same soil. On the other hand, the soil strength will be reduced by pile installation to a larger degree for the friction than for the end-bearing. Another example is the case of a shear surface in a fissured soil or rock; the characteristic strength to be used might be different, depending on whether the shear surface is free to follow the fissures or is constrained to intersect the intact material.

## BIBLIOGRAPHY

1. CEN TC 250 / Coordination Group , "Basis of Design", Third draft, April 14, 1992.

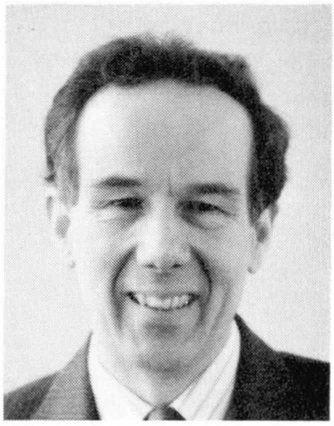
2. Eurocode 7 Drafting Panel, "Eurocode No.7 : Geotechnics, Design". First, incomplete draft, November 1989. Chapters 1,2,3,4,5,6,9 and 10. Prepared for the Commission of the European Communities.

# EC 7: Ground-Structure Interaction

## EC 7: Interaction entre sol et structure

## EC 7: Wechselwirkung zwischen Boden und Bauwerk

**Brian SIMPSON**
Director
Ove Arup & Partners
London, UK



Brian Simpson, born in 1947, obtained his MA and PhD degrees at Cambridge University. Since that time he has worked for the geotechnical department of Ove Arup & Partners, a company of structural and civil consulting engineers. He was made a director in 1984.

**SUMMARY**

The limit state format provides an ideal framework for the study of geotechnical design problems, including those involving significant ground structure interaction. When the partial factor method is applied to geotechnical problems, however, it is necessary to reconsider some of the simplifications conventionally adopted in structural engineering design. In this paper, three particular problems which have been studied by the Eurocode 7 drafting panel, are described and some solutions are proposed.

**RESUME**

Les principes du calcul aux états limites offrent un cadres idéal pour l'étude des problèmes de géotechnique, y compris les problèmes importants d'interaction sol-structure. Toutefois, lorsqu'on applique la méthode du coefficient partiel aux problèmes de géotechnique, il est nécessaire de revoir les simplifications conventionelles utilisées pour le calcul des structures. Cet article décrit les trois problèmes particuliers qui ont été étudiés par l'équipe rédactionelle de l'Eurocode 7 ainsi que les solutions proposées.

**ZUSAMMENFASSUNG**

Das Konzept der Grenztragzustände bietet einen idealen Rahmen zur Behandlung geotechnischer Bemessungsprobleme, einschliesslich erheblicher Boden-Bauwerk-Wechselwirkung. Bei der Anwendung der Methode der Teilsicherheitsbeiwerte müssen jedoch einige Vereinfachungen, wie sie im konstrucktiven Ingenieurbau üblich sind, neu überdacht werden. Der Beitrag beschreibt drei ausgewählte Bemessungsaufgaben, die von der Bearbeitergruppe des EC 7 studiert wurden, und schlägt einige Lösungen vor.

## 1. INTRODUCTION

Soil-structure interaction occurs when the deformation of the ground affects the distribution of stresses at the ground/structure interface or within structures supported by the ground or which support it. The limit state method provides an excellent framework within which to carry out both the structural and geotechnical aspects of design, but it may be necessary to reconsider some of the simplifications conventionally adopted when the method is applied to structural design. In this paper, three problems which have been considered at length by the drafting panel of Eurocode 7 Part 1 are described and their treatment in the present draft of EC7 is presented. The topics are also under debate in a Eurocodes ad hoc group on Soil-Structure Interaction, and a summary of the preliminary conclusions of that group will be presented at the Conference.

The three questions to be considered are as follows:

a) In a limit state, partial factor system, what load factors are appropriate for the geotechnical design (sizing) of foundations, particularly for gravity loads?

b) How should design proceed in situations where soil strength and stiffness significantly affect internal structural forces and bending moments?

c) How should design proceed for retaining structures in which the load to be born by the structure reduces as deformation occurs?

## 2. LOAD FACTORS FOR GEOTECHNICAL DESIGN OF FOUNDATIONS

### 2.1 Consistent system for geotechnical design

The 'geotechnical design' of foundations is taken to mean the determination of the size of foundation elements required to transmit loads from a structure into the ground. The internal strengths of these members is not the main concern here, but will be considered briefly at the end of this section.

Geotechnical design is concerned with foundations, earth retaining structures, slope stability and similar problems. Frequently these aspects of design overlap: for example, foundations may be placed on a slope or on ground supported by a retaining structure. Despite this, few countries, if any, have codes which present a philosophy of design which is consistent for each of these situations. Eurocode 7 attempts to achieve this.

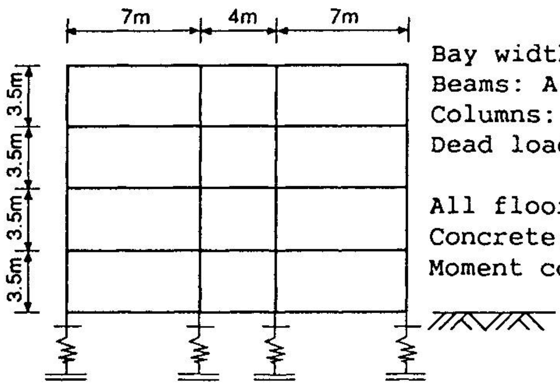
Geotechnical design is often concerned with the balance between the weights of large bodies of material and with soil strength derived from frictional properties; slope stability problems are an example of this. It is often not obvious which weights are favourable and which are unfavourable in a calculation. Even weight which, by its location, acts in a generally unfavourable manner increases the shear resistance of frictional soil. Fortunately, the density of the ground is usually fairly well defined so it is reasonable to apply unit load factors to the weight of soil and, implicitly, allow for minor variations in density within the partial factors on the strength components of the soil. In exceptional cases where there is genuine, major uncertainty about the distribution of weight of materials in the ground, special procedures are needed, probably using parametric studies.

If a slope supports a building or other structure, the weight of the structure is involved in the stability calculations in the same way as the weight of the ground. There is no reason to treat its weight differently in design, unless there is a genuine fear that it could be significantly heavier than its nominal weight. The same applies when soil and a supported structure together load a retaining wall.

## 2.2 Foundations of structures

The Eurocode 7 panel have understood that a reasonable factor to allow for the uncertainty in weight of structures would be about 1.1 to 1.15. Further components of the factor of 1.35 used on gravity loads in structural design are related to uncertainty in the distribution of forces between individual structural elements, that is, uncertainty in the loading model. The uncertainty about structural weight is therefore little more than that of the density of the soil and it is convenient to treat it in the same way, particularly as it may be unclear whether it acts favourably or unfavourably in geotechnical calculations.

Problems of bearing capacity are fundamentally similar to those of slope stability, and the two often merge into each other. It is therefore desirable to use a consistent approach to both. Furthermore, the loads on the ground at individual foundations are often governed by the stiffness of the ground itself.




Bay width in third dimension = 6m
Beams: A = $0.675m^2$ I = $0.046m^4$
Columns: 400mm x 400mm
Dead loads: Floors: 7.5 kPa
Roof: 5.0 kPa
All floors suspended
Concrete: E = 28 $kN/mm^2$ $\nu$ = 0.15
Moment continuity at all connections

Fig. 1 2D frame used as example

Figure 1 shows a simple 2-dimensional frame which has been used to study this, using an elastic analysis. Spring stiffnesses were first determined to give equal settlements (20mm) of all four foundations and individual spring stiffnesses were then varied by factors of 2. The changes of settlement were in the range -7.5mm to +8.4mm and the forces transmitted to the foundations varied by -38% to +44%. The 'poorer' foundations, represented by softer springs, received lower loads and the stiffer foundations received higher loads. Differential settlements were not serious in this case, but would have been reduced if the stiffer foundations had yielded slightly in response to the higher loads imposed on them.

This example illustrates how ground stiffness may have a large influence on the loads transmitted to each foundation supporting a reasonably stiff redundant structure. Significantly, there was no possibility that the poorer foundations would be pushed towards failure by forces greater than the nominal values for which they would normally have been designed. A foundation which yields because it is on poorer ground tends to shed load rather than to become overloaded. The same applies to a foundation which settles more than others because of its location, at the centre of a loaded group, for example. For structures which have little redundancy, the only uncertainty in the internal forces comes form the actions at source. The load factors applicable to these are thought to be less than the values adopted for structural design in general.

The yielding or failure of the ground beneath a foundation is a different limit state from that of structural failure of the foundation or the structure it supports. The forces which would be in the structure as the limit state approached would be different.

## 2.3 Load factors in Eurocode 7

These considerations have led the drafting panel of Eurocode 7 to propose that the load factor on the weight of supported structures should generally be unity for ultimate limit state design. Exceptions would occur where there is genuine, major uncertainty about the actual weight of the structure or about the distribution of load transmitted to the ground, for reasons other than ground stiffness itself.

Similar considerations apply to variable actions, but it is likely that these are more uncertain at source than is structural weight. A factor of 1.3 has therefore been proposed for these in Eurocode 7, though it might be argued that the ratio between factors on permanent and variable loads should be the same as in other Eurocodes.

The structural design of the foundation elements themselves should allow for the possibility that they carry significantly more than the nominal load. It is anticipated that structural design will therefore use the load factors given in the structural Eurocodes.

# 3. SITUATIONS WHERE SOIL STRENGTH AND STIFFNESS AFFECT STRUCTURAL STRESSES

The bending moments in laterally loaded piles are greatly affected by the strength and stiffness of the ground. This problem has been addressed in Eurocode 7 and will be used here as an example of a broader class of situations.

In many design calculations, two sets of variables and their uncertainties are involved, usually actions and structural strength. Values of partial factors have been derived for these situations. Generally the stiffness of the structure has a fairly small influence on action effects and mean values can be used for its properties without serious error. For laterally loaded piles, and similar design situations, the strength and stiffness of the ground greatly affect the action effects (bending moments) and are also significant uncertainties. Eurocode 7 proposes that it is not sufficient to assume mean values for these, but it would be unduly severe to assume extreme design values for the actions, the ground properties and the structural resistance simultaneously.

The wording adopted in Eurocode 7 requires that two separate design checks should be made. In the following extract, the 'characteristic' soil properties are to have a target probability of 5%, as discussed by in the Conference by Baguelin.

> It shall be demonstrated that the piles can withstand the stresses and bending moments derived by both the following methods:
>
> - Use design actions and soil parameters as specified for geotechnical design in Chapter 2 of this Eurocode.
> - Use design actions as specified in the relevant material Eurocodes for structural design, together with characteristic soil properties, as specified in Chapter 2 of this Eurocode.
>
> In both methods, the design parameters for structural materials shall be as specified in the relevant material Eurocodes.

It is foreseen that an approach of this type could be used in other situations in which ground properties significantly affect structural stresses. In design of retaining walls, it is proposed that factored values of soil properties and structural resistance should be used, with unit factors on all gravity actions.

## 4. SITUATIONS WHERE THE LOAD ON THE STRUCTURE REDUCES AS DEFORMATION OCCURS

Unlike most of the other forms of loading on structures, disturbing earth pressures on retaining walls generally reduce as the wall deflects. Thus the loads on a wall in service often exceed those as collapse approaches, to an extent which depends heavily on the type of soil and its geological or construction history. In these circumstances, the definitions of ultimate limit states require careful attention.

A wall yielding in bending might deflect more, allow the soil to mobilise more of its strength which would ensure stability. Failure of a concrete wall in shear would be more immediate and serious. However, even a failure in bending could lead to very large displacements, which would be more severe than a serviceability limit state. Eurocode 7 needs guidance from colleagues in the other Eurocodes on the significance of attainment of ultimate resistance in bending in situations where this leads to larger movements, but not to collapse. This topic is under review in the ad hoc group on ground-structure interaction.

Leere Seite
Blank page
Page vide