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Autor: Simpson, Brian
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Basis of design in Eurocode 7

Brian SIMPSON
Director
Ove Arup & Partners
London
UK

Brian Simpson, born 1947, obtained MA and PhD degrees at Cambridge University. He combines general geotechnical design and trouble-shooting with an interest in numerical modelling. He has been a director of Ove Arup & Partners since 1984.

Summary

Eurocode 7 provides rules for Geotechnical Design for all structures designed to the Eurocodes. Its development has required very careful definition and application of the concepts of limit state design, partial factors and characteristic values of material properties. The code is consistent with Eurocode 1 and the other Eurocodes, whilst remaining within the principles of sound geotechnical engineering. Examples are presented of applications to foundations and retaining structures.

1. Introduction

Eurocode 7 (EC7) provides rules for Geotechnical Design for all structures designed to the Eurocodes. The geotechnical community, and in particular the Project Team for EC7 Part 1, have accepted the challenge to provide a code which is consistent with Eurocode 1 (EC1) and the other Eurocodes, and also with the principles of sound geotechnical engineering. This has required very careful definition and application of the concepts of limit state design, partial factors and characteristic values of material properties. The code allows design by calculation, by prescription, by testing and by the Observational Method.

EC7 Part 1 provides general rules for design. Following the introductory Section 1, Basis of Geotechnical Design is considered in Section 2. Sections 3 to 5 deal with the investigation of natural ground and requirements for construction, monitoring and maintenance both of engineered fill and of structures supported by the ground. The remaining sections consider particular types of structures in more detail: shallow foundations, piled foundations, retaining structures, slopes and embankments. These later sections necessarily add details to the basis of design for specific cases. In this paper, clause numbers are noted in [square brackets].

EC7 Parts 2 and 3, which have not yet reached ENV status, deal with the use of laboratory and field testing, respectively, in geotechnical design.

2. EC7 Section 2 - Basis of Geotechnical Design

2.1 Recognition of complexity - Geotechnical categories

All terrestrial construction has a geotechnical component. However, the difficulty and complexity of this component varies greatly from one situation to another, and the degree of expertise and attention required therefore also varies. An attempt has been made to classify this requirement by defining *Geotechnical Categories*. Category 1 includes only small structures in simple, well understood situations and may not require the involvement of a civil engineer in the design. Category 2 is for normal structures, requiring at least a qualified civil engineer, whilst Category 3 is for particularly difficult situations where experienced specialists are required.

The code is directed primarily at Category 2. It is foreseen that quantitative measurement and calculation may be unnecessary for Category 1, whilst Category 3 may require procedures beyond the scope of the code. No attempt is made to vary the values of safety factors between the categories; it is considered that safety in geotechnical engineering is governed more by the quality of geotechnical investigation and workmanship than by precision in calculation models and partial factors [2.4.1(2)].

In the writer's opinion, it is unclear whether this system of categorisation can be applied successfully to typical projects in which some geotechnical elements are very straightforward whilst others present considerable difficulty.

2.2 Limit state design

The concepts of limit state design are applied to geotechnics in EC7. Four features which have caused much discussion are noted here.

a) Limit states are generally defined in terms of *damage* to *structures*. Damage to the ground is rarely of significance in itself. Traditional geotechnical calculations have related either to pseudo-elastic states or to plastic mechanisms, but these do not necessarily correspond directly to serviceability and ultimate limit states. In particular, structures are sometimes brought to ultimate collapse by ground movements when the ground itself is far from a state of plastic mechanism. Examples of this include heave due to swelling clays and negative skin friction (downdrag) on piles.
b) The concept of an 'action' requires careful definition, particularly in earth pressure problems and other situations of ground-structure interaction. The key definition is considered to be *a force (or imposed displacement) which is a known quantity at the start of the current calculation* - that is, a force which is not a reaction [2.4.2(1)P]. This allows the possibility that partial load factors can be applied to actions, which would be difficult, or impossible, for reactive forces. It also implies that some forces will be reactions in one calculation but will be classed as actions in a later calculation.
c) Even in ultimate limit state design, it is essential that all design strengths can be mobilised simultaneously. Thus compatibility of strains and displacements must be considered, though very large displacements may sometimes be allowed.
d) In the design of retaining walls, it is common that the earth pressures on the wall reduce as deformation proceeds. Thus the earth 'load' on the wall may be lower in ultimate limit

state conditions than in service. This is an unusual situation for structural design.

e) Serviceability limit states, involving displacements, are often critical to the design of structures in geotechnical situations. However, displacements are difficult to calculate and traditional designs have largely covered this difficulty by use of appropriate factors of safety in analyses of plastic mechanisms. Since the values of partial factors are partly based on experience of successful designs, it can be expected that this will remain the case, though it involves a dilution of the concept of limit state design.

2.3 Characteristic values of ground material properties

EC1, Section 5, requires that characteristic values of material properties shall have "a prescribed probability of not being attained in a hypothetical unlimited test series". It also requires that "a conversion factor shall be applied where it is necessary to convert the test results into values which can be assumed to represent the behaviour of the material in the structure or the ground". It proposes that characteristic values should be defined as a 5% fractile for strength parameters and as the mean value for stiffness parameters. Attention will be concentrated here on strength parameters, whilst the 'mean value' requirement for stiffnesses will be discussed later.

These requirements of EC1 are not rigorously consistent. However, an attempt has been made to apply them, with due regard to the following special features of geotechnical design.

a) In geotechnical design, the designer usually is in possession of site-specific information which gives him much more knowledge of the uncertainties of material properties than the code drafter could possibly have. This is the reverse of the normal situation in structural design.
b) Poor performance of a small element of the ground, of the size involved in a field or laboratory test, is usually of no consequence to the performance of a structure. This is sometimes not the case in structural design. Thus geotechnical characteristic values will often be mean values spacially, though they are not to be means in a probabilitic sense.
c) It is often good practice for the designer to consider in combination several sets of data in order to derive an appropriate characteristic value. These will have varying degrees of relevance and reliability and will often include some conflicts. Observation of the performance of other structures is one important source of data, together with geological and other background information.
d) The operative properties of the ground may be changed by construction activities.

EC7 requires that the designer takes all of these factors into consideration in assessing characteristic values. It then requires "a cautious estimate of the value affecting the occurrence of a limit state" [2.4.3(5)P]. EC7's approach means that characteristic values are somewhat subjective, depending on the knowledge and experience of the designer. However, an alternative approach in which these were disregarded in order to attain uniformity in the assessment of characteristic values would involve dangerous neglect of vital information. The writer has presented an example of the assessment of a typical, but complex set of field and laboratory data for one site (see Krebs Ovesen (1995)).

An application rule points out that statistical analysis may be used provided that proper account is taken of *a priori* knowledge. Where statistical methods are used, "the characteristic value should be derived such that the calculated probability of a worse value *governing the occurrence of a limit state* is not greater than 5%". Schneider (see Krebs Ovesen(1995)) has proposed a statistical approach which covers Item (b) above, taken alone, when deriving a characteristic value from normally distributed results of tests on small samples. He suggests that, to give a value with a 5% probability of *governing the occurrence of a limit state*, the characteristic value should be ½ a standard deviation from the mean of the test results. In simple cases, this is probably consistent with typical engineering assessments of test results, such as that described as moderately conservative by Padfield and Mair (1984).

2.4 Actions - Cases A, B and C

EC1, Table 9.2 requires that designs should be verified for Cases A, B and C 'separately as relevant'. The unfortunate words 'as relevant' are copied into EC7, but succeeding clauses make it clear that all designs must satisfy all three cases *in all respects* - geotechnical and structural. The bracketed values of partial factors are shown in Table 1; they are varied for the three cases for geotechnical material strengths as well as for actions.

Case	Actions			Ground Properties			
	Permanent		Variable				
	Unfavourable	Favourable	Unfavourable	tan ϕ'	c'	c_u	q_u [1)]
Case A	[1.00]	[0.95]	[1.50]	[1.1]	[1.3]	[1.2]	[1.2]
Case B	[1.35]	[1.00]	[1.50]	[1.0]	[1.0]	[1.0]	[1.0]
Case C	[1.00]	[1.00]	[1.30]	[1.25]	[1.6]	[1.4]	[1.4]
1) Compressive strength of soil or rock.							

Table 1. EC7 Table 2.1: Partial factors — ultimate limit states in persistent and transient situations

In EC7, Case A is used only to check against buoyancy. This is a geotechnical equivalent to the 'stability' check of structural design, since it involves a balance of actions with no, or little involvement of the strength of materials. In practice, however, it is sometimes the case that the strength of ground plays a partial role in preventing buoyancy problems. It was therefore considered appropriate to provide partial material factors for use in these cases; these could, perhaps, be made equal to those of Case C. The writer questions whether the value of 0.95 applied to beneficial weight is sufficiently low to give safety in buoyancy problems.

Case B originated from structural engineering considerations and Case C from geotechnical, thus in Case B safety is derived from load factors and in Case C from ground material factors. During the drafting process it was realised that both cases had merit; no logic has been found to suggest that a case considered necessary for geotechnical stability can be disregarded in checking structural strength, or *vice versa*. It was considered that if partial factors were applied simultaneously to structural materials, actions and ground materials the results would be more pessimistic than those of other aspects of the structural design. The two cases therefore have factors applied to structural material strengths (as in other Eurocodes), together with either factored actions or factored ground materials. Researchers working for

The European Sheet Piling Association have suggested that this procedure leads to unnecessarily conservative designs. This will be considered later in an example.

EC7 acknowledges that application of Case B may sometimes lead to physically unreasonable design actions. This is particularly true when factors are applied to water pressures. In these cases EC7 allows the same factors to be used as model factors, applied directly to structural internal forces and bending moments.

2.5 Design by prescriptive measures, testing and the Observational Method

EC7 recognises that design may be based on three procedures other than use of normal calculations. Use of combinations of the four approaches is encouraged.

Prescriptive measures are, in effect, commonly recognised conservative details. They may be used, for example, in design for corrosion or frost protection, drainage requirements or even safe bearing pressures. *Design by testing* is particularly relevant to piling and ground anchors. An example will be given below.

The Observational Method is an approach in which the design may be modified on the basis of the observed performance of the partly completed structure. It can be used to permit construction to start, using a less pessimistic design than normally required, provided that contingency plans are clearly established in case the optimism is not justified by observation. The requirements of this approach are given in some detail, with the intention that design on this basis will have no greater risk of failure than on the basis of normal calculations.

3. Examples

3.1 Shallow foundation

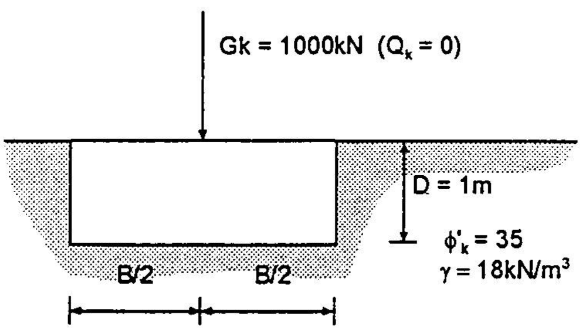

Figure 1. Centrally loaded footing.

Figure 1 shows a centrally loaded square footing to be constructed in sand for which the characteristic angle of shearing resistance ϕ_k is 35° and the characteristic unit weight γ_k is 18 kN/m^3. Section 6 of EC7 provides bearing capacity factors and also states that "For rigid foundations, the bearing pressure may be assumed to be distributed linearly. More detailed analysis of soil-structure interaction may be used to justify a more economic design,..." [6.8].

For this problem, Case A is irrelevant. A calculation for Case B leads to a required footing width of 1.05m with a maximum bending moment of 177kNm assuming a linear distribution of bearing pressure (169kNm/m across the 1.05m width of the square footing). Case C requires a width of 1.29m with a maximum bending moment of 161kNm (125kNm/m). The footing width is therefore governed by Case

C and must be at least 1.29m. If it is now assumed that the bearing pressure is distributed linearly for the loading of Case B, and the width is 1.29m, the maximum bending moment becomes 218kNm (169kNm/m across the 1.29m width). This is the ULS *design* moment for structural design, unless a more complicated calculation is carried out to justify a non-linear distribution of bearing pressure.

Calculations for the serviceability limit state, in terms both of crack widths and settlement, follow traditional lines.

3.2 Piled foundation

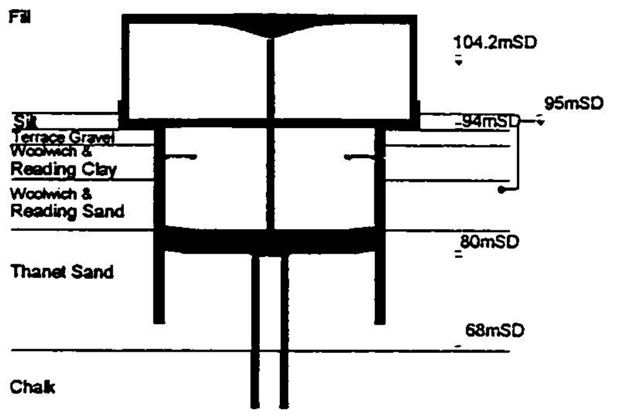

Figure 2. Underground metro station

Case	A	B	C
Water level mOD	110	104.2	110
γ_G (favourable)	0.95	1.0	1.0
γ_{water}(unfavourable)	1.0	1.35	1.0
Design tension force F_d kN	4823	5595 *Structural design*	4379
Conversion factor ξ	1.5	1.5	1.5
Partial factor γ_m	1.4	1.0	1.6
$\xi\,\gamma_m$	2.1	1.5	2.4
Required test result R_{tm} kN	10128	8393	10510 *Critical*

Table 2. Calculations for tension pile.

Figure 2 shows a cross section through a proposed underground metro station. Tension piles are required to hold down the base against buoyancy forces, which depend on the likely future piezometric level of the water in the aquifer, which is uncertain. The design is to be based on a load test.

Table 2 shows the results of calculations. For cases A and C the 'worst credible' water level of 110mSD (site datum) has been used. However, in Case B design actions are generally derived by factoring characteristic values, so a 'worst probable' level of 104.2mSD has been used as a characteristic value. The table shows that in this case the ULS *design* tension force in the pile is given by Case B as 5595kN, which should be used for structural design.

Section 7 of EC7 requires that where only a single load test is used, giving a measured resistance R_m, the characteristic resistance of the pile be taken to be R_m/ξ, where ξ=1.5. (The section also requires that the result of the load test be shown by calculation to be within reasonable expectations.) Section 7 gives partial factors for pile design, different from those of Table 1 above, which convert characteristic values to design values.

In this situation it is found that the structural design is governed by Case B, whilst the required result of the pile load test is governed by Case C. Further calcualtions, not included in Table 2, show that if the dead weight of the structure is increased, the case governing the load test requirement becomes first Case B and then Case A.

3.3 Sheet pile wall

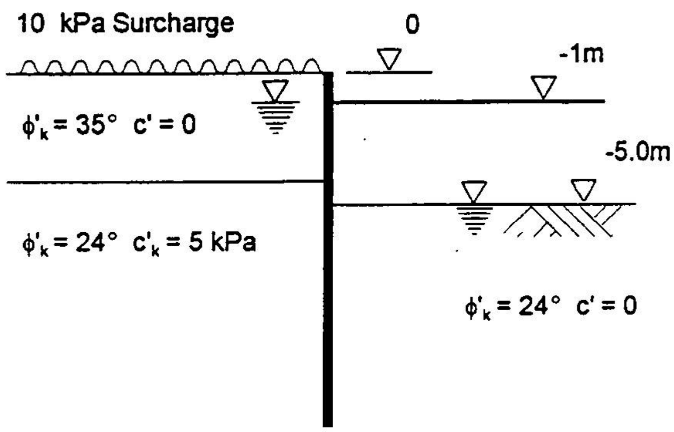

Figure 3. Sheetpile

Figure 3 shows a proposed sheet pile retaining wall for which Table 3 shows some calculated results. Section 8 of EC7 requires that when walls are supported by passive pressure, a top layer of the passive material, in this case 0.4m thick, be disregarded in the calculation. In other respects, the calculation follows the requirements of Table 1 above. EC7 does not specify how the calculations are to be carried out, except that equilibrium must be achieved.

* computed # assumed	Length (m)	BM
Case B - simple active / passive pressures	9.6 *	371
Case C - simple active / passive pressures	11.3 *	431
Case C - SAFE finite element program	11.3 #	350
Case C - FREW pseudo-finite elements	11.3 #	327
Case C - SPOOKS plasticity solution	11.5 *	376

Table 3. Computed lengths and ULS design bending moments (kNm/m) for sheet pile wall.

In rows 1 and 2 of Table 3, simple active and passive earth pressures are assumed. Case C governs the design of wall length and gives a ULS design bending moment of 431kNm/m, compared with 371kNm/m for Case B. The length obtained from this calculation is in line with conventional calculations, or slightly shorter. However, concern has been expressed that the design bending moment for Case C is too high; the option of disregarding this and using the Case B moment has been proposed. Table 3 also shows the results of three calculations which use the same Case C material parameters, but allow for soil-structure interaction in deriving the earth pressure distribution. This is specifically permitted by EC7 and leads to considerably smaller bending moments, more in line with conventional design.

4. Serviceability limit state

Foundation failures involving ultimate collapse are rare, but serviceability failures are too common. The limit state approach of EC7 may help to identify separately the ultimate and serviceability limits, but the code does not provide much guidance on the assessment of serviceability, which is usually related to deformation and, therefore, ground stiffness. Partial factors of unity are applied in SLS design, and EC1 states that the characteristic values of material stiffnesses should be mean values. This appears to imply that 50% of all constructions should be expected to exceed serviceability limit states, which is not acceptable.

5. Concluding remarks

The current draft of Eurocode 7 achieves a large degree of consistency with Eurocode 1, using sound geotechnical principles. The following points have been found to be critical, and are still somewhat controversial.

a) Characteristic values for ground properties are defined as cautious estimates of the values actually occurring in the ground in such a way as to govern the occurrence of a limit state. To obtain these, adjustments may be needed to results of soil tests and allowance is to be included for the effects on soil parameters of construction activities. All relevant information is to be included in the designer's assessment of characteristic values. A strict statistical approach will often be unhelpful.

b) The final design, with the geometry as it will be built, is to be verified for all three cases A, B and C. Calculations are not needed for cases which, by inspection, will not govern the design.

c) EC7 gives rules for derivation of design paramter values, but allows the use of any means of analysis consistent with basic principles of mechanics, including compatibility of displacement. Studies of soil-structure interaction may therefore be used to improve efficiency in design of both foundations and retaining walls.

d) The requirement in EC1 that characteristic stiffnesses should be mean values appears to imply that 50% of all designs would be expected to exceed the serviceability limit states of displacement. This is not acceptable.

6. References

Krebs Ovesen N (1995) Eurocode 7 for geotechnical design. Proc. Bengt. B. Broms Symposium on Geotechnical Engineering, Singapore.

Padfield, C.J. & Mair, R.J. (1984) Design of retaining walls embedded in stiff clay. CIRIA Report 104.