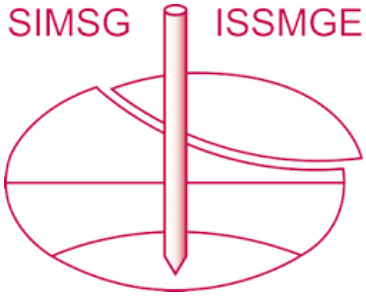
INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Theme lecture: Some questions about Limit State Design in geotechnical engineering

Exposé sur le thème: Quelques questions concernant les calculs aux états limites en géotechnique

B. Simpson – *Arup Geotechnics, London, UK*

ABSTRACT: This contribution will ask some questions, and for some of them it can give partial answers. Some of the other contributors to the session will also consider these questions. The questions may be summarised as follows:
1. What do we mean by 'Limit State Design'? 2. Are we wise to concentrate on limit states rather than working states? 3. Are we wise to concentrate on ultimate limit states rather than serviceability limit states? 4. How are we to define characteristic values? 5. Must we have three cases A, B and C - or just one case? 6. What sort of factors of safety are acceptable? 7. How do we view temporary works and the use of the Observational Method?

RESUME: Cette note soulèvera quelques questions et fournira des débuts de réponse pour certaines d'entre elles. Plusieurs des participants à la séance considéreront également ces questions. Les questions posées peuvent être résumées comme suit:
1. Que veut-on dire par "Calcul aux Etats limites"? 2. Est il prudent de se concentrer sur les états limites plutôt que sur les états fonctionnels? 3. Est il prudent de se concentrer sur les états limites ultimes plutôt que sur les états limites de service? 4. Comment définir les valeurs caractéristiques? 5. Doit-on avoir 3 cas A, B et C ou simplement un? 6. Quels coefficient de sécurité sont acceptables? 7. Comment considérons nous les travaux provisoires et l'utilisation de la Méthode de l'Observation.?

1 WHAT DO WE MEAN BY 'LIMIT STATE DESIGN'?

The term 'Limit State Design' is sometimes taken to mean design based on partial factor methods, or on probabilistic thinking. Whilst both of these might be helpful, I do not believe they are fundamental to the definition of limit state design. I suggest that limit state design simply means 'a design procedure in which attention is concentrated on limit states, with the intention of preventing them occurring'. Limit states are simply states in which something has gone wrong. This may be catastrophic or merely inconvenient, or somewhere between the two. The alternative to 'limit state design' is 'working state design'. In working state design, attention is concentrated on the expected working state of a structure, in which it is performing well, with much less attention on what might go wrong. The adoption of limit state design means that our concepts and calculations are principally directed to situations which we hope will never occur. This implies that the parameter values we will use in calculations will always be more cautious then the values we consider to be most likely.

2 IS IT WISE TO CONCENTRATE ON LIMIT STATES RATHER THAN WORKING STATES?

In the past we have learnt a great deal about the behaviour of structures by measuring performance, back-analysing, and then trying to predict the actual performance (the working state) of the next structure to be designed. In British practice, this process has been carried out with great success for deep excavations in London. Data has been accumulated over many years, such as that published by Burland and Hancock (1977), who measured the ground deformations around the underground car park at the Houses of Parliament. Much understanding of ground behaviour has been developed from that set of measurements and from others which have followed. It is far more difficult to take measurements in a planned way for limit states, which are states which will occur rarely and unintentionally. So limit state design may not help us to develop understanding of ground behaviour.

On the other hand, when failures occur it is usually because a situation arises, or a combination of situations, which had not been foreseen. This is most likely to happen when designers concentrate only on what they hope will happen and are not encouraged to consider systematically what might go wrong. So I believe that limit state design offers the best way of avoiding failures, but we need to consider how to make good use of the measurements we can make in the working state.

3 ULTIMATE LIMIT OR SERVICEABILITY LIMIT STATES?

3.1 *Which state really governs design?*

Most codes of practice based on limit state design concentrate more on ultimate limit states than on serviceability limit states. In fact, it is still a very common mistake, at least in Britain, to think that the term 'limit state' only refers to ultimate limit states.So my third question is: "Are we wise to concentrate on ultimate limit states rather than serviceability limit states?" Are the criteria which govern design really associated with uncertainty about parameters which affect ultimate limit state, or are they really related to serviceability requirements?

It is often said that the limiting requirements for foundation design are really serviceability requirements, and that ultimate limit state requirements do not govern the designs. I am not completely sure that this is true, but it largely depends on how we distinguish 'Ultimate' and 'Serviceability' limit states. For example, in one recent problem some of the piles beneath a house failed due to a combination of inadequate bearing and negative skin friction. The house needed significant repairs, was possibly becoming dangerous and so was evacuated, and it had to be underpinned to give it a new piled foundation. Clearly this was well beyond a normal maintenance procedure: the piles had failed and structural brickwork was broken. This seems to me to be an ultimate limit state. On that basis, I can think of very few, if any, serviceability limit state failures which have been caused simply by the weight of a building acting on the ground below. The situation might be different in bridge design.

In another example, quite severe cracks developed in the construction joints of a building. Although these were not dangerous, they will require more than normal decoration to repair them. However, in my experience, this type of serviceability damage is most often caused by other effects in the ground, not simply by the weight of the building. These other effects include nearby construction, for example of tunnels or excavations, or the effects of tree roots causing settlement in plastic clays or heave when trees are removed. Besides causing serviceability failures,

tree roots can also cause severe damage which constitutes ultimate limit states.

3.2 *BS8002 - mobilisation factors*

The British Standard Code on Earth Retaining Structures, BS8002, published in 1984, was based on a limit state approach. But it also adopted the opinion that the most important limit state for retaining wall design is the serviceability limit state. It was recognised that calculations of displacement are extremely difficult and often unreliable, so the recommended form of calculation was based on limit equilibrium with a reduction factor applied to soil strength. The idea is shown in Figure 1. The design must show that equilibrium can be obtained whilst using no more than about 80% of the soil strength. This means that there is no need for large strains to develop in the soil and hence displacements are reasonably small.

In many ways, the calculations required are similar to those used in other codes for ultimate limit states with partial factors applied to the parameters. However, the justification for the factor applied here - called a 'mobilisation factor' - was the need to prevent unacceptable strains and displacements, rather than to provide an allowance for uncertainty in the strength of the materials. It seems to me that this approach to serviceability calculations might be worth further development, though I think BS8002 is not really successful in forming the link to structural design codes.

4 CHARACTERISTIC VALUES

4.1 *Statistics or human assessment?*

Other contributors will discuss in some detail the issue of characteristic values, which raises the question "How are we to define characteristic values for geotechnical design?"

In the development of Eurocode 7, the definition of characteristic values has led to more debate than any other issue. It seems to me that this is not a new problem - simply an old problem more clearly stated. The question we need to ask is: in our traditional practice, how do we select parameter values to use in calculations? How do we instruct a young engineer to proceed when he is given a typical confusing and incomplete set of information? I believe engineers very rarely use genuine best estimates or probabilistic mean values in the design process; they always lean to the side of caution. After discussing important issues involved in selecting characteristic values, Eurocode 7 defines then, perhaps rather weakly, simply as "a cautious estimate of the value affecting the occurrence of the limit state". Some useful work has been done recently by Schneider, Bauduin, Kavvadas and others, showing that in a restricted, but fairly typical range of circumstances the characteristic value could be derived by statistical processes. Typically it might be taken to be about half a standard deviation from the mean value

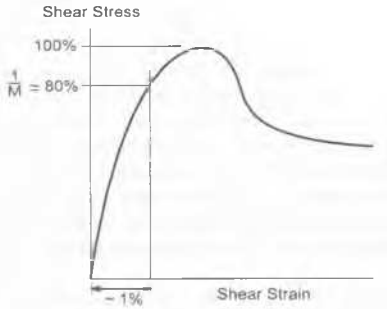


Figure 1. Concept of mobilisation factor used in BS8002. Strain and displacement limited by mobilising only 8% of the strength.

of a set of test results, provided the mode of failure is one which averages the properties of a large mass of soil (see Krebs Ovesen (1995) and Schneider's contribution to this discussion). But here I have a major concern. The human mind, particularly with some engineering training, is able to fuse together diverse, or even conflicting information and draw reasonable conclusions. We must make sure that we don't adopt statistical processes unless they can perform this task equally well.

4.2 *An example*

For example, Figure 2 shows the results of a series of undrained shear strength measurements in a stiff clay. The measurements are made using unconsolidated undrained triaxial tests. A mean line has been drawn through the data and it is clear that undrained strength increases with depth. But where is the characteristic line? The answer to that question should depend on how the characteristic value will be used - what is the limit mode being considered? For example, if we need undrained strength for calculation of ground movements around the retaining wall, we might use a value such as the 'cautious (average)' value shown on the figure. However, for a problem in which failure might take place in a small zone of soil, such as an isolated foundation placed at a deep level, we should use a more cautious value - the 'cautious (local)' value. From these boreholes, we also had results from standard penetration tests, as shown in Figure 3. In London Clay, we know that there is usually a constant factor between standard penetration and undrained shear strength results; the factor is about 4.5 to 5. So if we transfer the mean line from the SPT results onto the undrained strength plot we obtain Figure 4, and find that something strange has occurred - our normal correlation appears not to work. In fact, the measured undrained strengths are remarkably high: they are consistent with very low water contents, which were measured, but this might simply mean that the samples had dried out on the way to the laboratory, though there was no reason to suspect this. Figure 4 also shows lines representing mean values through data from other nearby sites, both for undrained shear strength and SPT results. The usual close correlation applies to these, and it is clear that the undrained strengths for the new site are remarkably high.

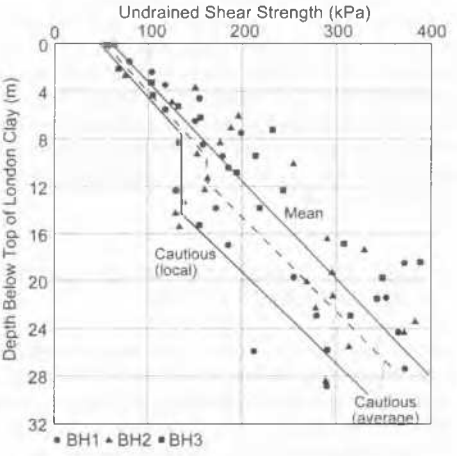


Figure 2. Undrained shear strengths measured at one site using 100mm diameter UU triaxial tests.

So what value should be used as the characteristic undrained strength? We should not ignore the values measured in the triaxial test, but we should also take account of the SPT results and the data from adjacent sites. The value proposed for this data is shown on Figure 5. This is less than the initial assessments in Figure 2, which were based on the triaxial results only, and is closer to a lower bound of this particular set of triaxial results.

This is the sort of process engineers often need to follow when trying to interpret real data. It may be that statistical methods could trace a similar process. However, this would require quite advanced

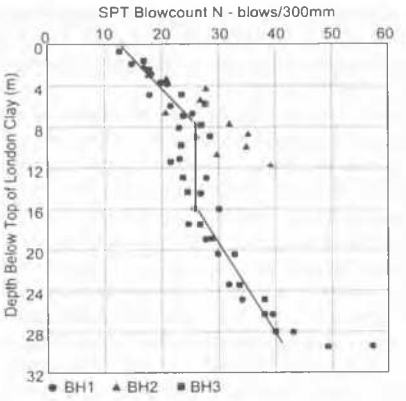


Figure 3. SPT results from some sites at the same site.

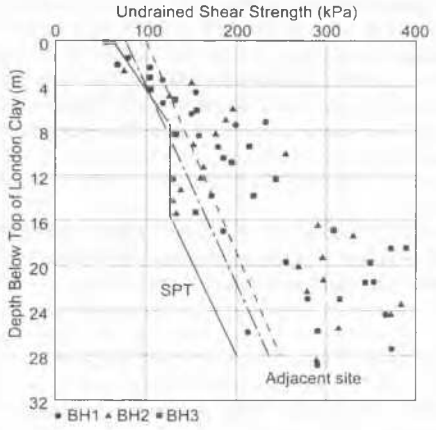


Figure 4. SPT results and results from adjacent sites compared with UU triaxial results.

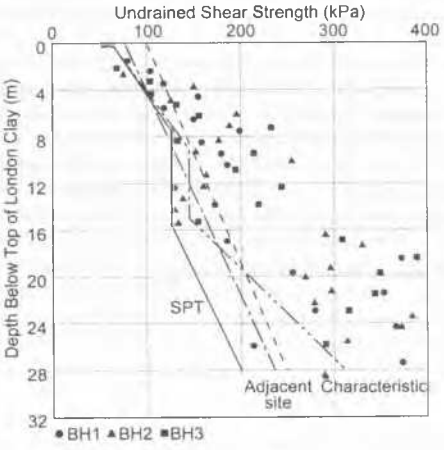


Figure 5. An assessment of a 'characteristic' profile of undrained strength.

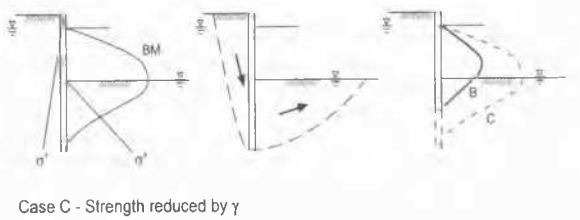


Figure 6. A retaining wall design for Cases C and B.

methods and any statistical approach which failed to take account of the diverse array of data, typically available, would be harmful to the design process.

5 CASES A, B AND C

More recently in the work on Eurocode 7, another issue has become very prominent - the use of Cases A, B and C. The partial factors which are to be applied in these two cases are shown in Table 1. The factors for Case B were originally derived from structural analysis, and those for Case C from geotechnical design. But the question is often asked: "Must we have three cases A, B and C - or could design be based on just one case?" Eurocode 7 defines these two cases, and in my opinion makes the following rule clear: 'All structures must comply with both Cases B and C in all respects, both for structural design and geotechnical design.' Case A should also be included in this statement, but it is not usually the critical case. Some examples of the application of this system are presented by Orr and O'Brien(1996) and Simpson (1996).

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

Case	Actions			Ground Properties			
	Permanent		Variable	$\tan\phi$	c'	c_u	q_u 1)
	Unfavourable	Favourable	Unfavourable				
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.							

This system is said to have two problems:

a) It requires two calculations, where it would be more convenient to have only one.

b) Arguably, it leads to uneconomic design, especially for sheet pile walls which have been studied in some detail.

Various schemes have been put forward to try to avoid these problems, and these are discussed on more detail by other contributors to this discussion. One of the main suggestions is that the geometry of the structure should be determined by geotechnical calculations taking account only of Case C, whilst the strength of the structure is determined by structural calculations, only for Case B.

I am unhappy with this suggestion for the following reason. Figure 6 shows a retaining wall designed for the situation represented by Case C. It will generally be agreed that Case C determines the length of the wall in order to prevent a failure of the type shown in Figure 6b. But if this length is necessary, as the Case C calculation indicates, then it is unavoidable that the bending moment in the wall will be large, as shown in Figure 6a, when the wall is doing the job of preventing the Case C failure. If the wall has only the strength required for Case B as shown in Figure 6c, then it cannot prevent a Case C failure. A proper limit state analysis requires that a design situation is set up (such as Case C - or Case B) and that the structure is shown to be able to withstand that situation, in all respects. If it is true that the wall does not need the strength calculated for Case C, then it must also be true that it does not need the length for Case C. Length and strength must be compatible.

The strength of sheet pile walls is a further significant issue in this discussion. Eurocode 3 Part 5 indicates that full plastic moments of resistance may be used for ultimate limit state calculations for most sheet pile sections. It may be that the rather larger bending moments derived for Case C can be matched with the larger moments of resistance of plastic sections with the result that sheet pile walls need to be no stronger than they have been in traditional practice. At least, this seems to be likely from a British perspective.

6 WHAT FORM OF FACTORS OF SAFETY?

Sixth question: "What sort of factors of safety are acceptable?" Four alternatives are considered here.

a) Factors for parameter uncertainty. Most designers would accept that part of the purpose of the factor of safety is to deal with uncertainty.

b) Mobilisation factors. The use of mobilisation factors to control serviceability, as noted for BS8002 above, is an intriguing possibility. This is still much discussed within the Eurocode 7 team, but a satisfactory working approach has not yet been found.

c) Model factors. Eurocode 1 defines these as 'Partial factors associated with the uncertainty of the resistance or action or action effect model'. For example, this slide shows a fairly common situation in which a bearing capacity calculation is required but the ground profile and the variation of water levels means that standard bearing capacity factors cannot be used. Instead, a slope stability calculation has been adopted. However, it is found that in very simple situations, for which bearing capacity factors are appropriate, the slope stability calculation tends to give a rather higher factor of safety. It was therefore concluded for this problem that a model factor should be applied to the slope stability calculation so that its results were equivalent to bearing capacity calculations. This seems to me to be a genuine model factor, dealing with the uncertainty, or perhaps inaccuracy, of the calculation model itself.

d) LRFD - Load and resistance factor design, as used in American codes. This differs from the type of limit state design used mainly in Eurocodes in that the factor is not applied directly to soil strength but to a resistance effect, such as bearing capacity. In fact, Eurocode 7 uses this approach directly for pile design. In my opinion LRFD is very sensible in those situations where the link between material strength and the resistance of a structure, or structural element, is difficult to establish. For example, in steel members it is probably true that the uncertainty about the properties of a cross-section is greater than the uncertainty in the material strength. In the geotechnical field, this is probably applicable to pile design, but not to most other aspects of geotechnical design. A factor on resistance of an element of a structure is, in effect, quite similar to a model factor, at least for geotechnical work. Besides allowing for uncertainties, the values adopted in current codes may provide some control over mobilisation of strength, and hence of displacements.

7 TEMPORARY WORKS AND THE OBSERVATIONAL METHOD

The points made at the beginning of this contribution about limit states, working states and the value of monitoring behaviour on site are particularly relevant to temporary works and the use of the Observational Method.

Eurocode 7 does not allow any reduction in safety levels merely because of the temporary nature of works or design situations. In relation to partial factors, however, it says that 'where it can be justified on the basis of the possible consequences, less severe values may be used for temporary structures or transient situations'. This paragraph refers to partial load factors, and it is perhaps unfortunate that Eurocode 7 does not have a similar sentence referring to partial material factors. However, the principle applied here is that the justification for less severe safety requirements lies in the possible consequences, not the temporary nature of the situation. On a construction site, there are several features which reduce the consequences of failure. The two main ones are:

a) Engineers and experienced construction workers are present on site and are able to observe both the actual design situations which apply and the behaviour of the geotechnical structure. In some cases, the behaviour might be being monitored precisely by measurement, but often simple observation by experienced personnel is all that is needed to check that there is no imminent limit state.

b) If observation leads to the conclusion that the structure is near a limit state, then the personnel on site often have the knowledge and authority to take action to prevent a failure, or at least to move people and equipment out of the area of danger.

It is therefore often reasonable to argue that the consequences of failure are lower during construction than they would be for the completed structure when untrained, unobservant members of the public are relying on its safety.

Eurocode 7 has a specific clause about the Observational Method, which has received a great deal of support from the geotechnical community. In the United Kingdom, a report has recently been completed for the British Construction Industry Research and Information Association, which sets out in some detail the philosophy, safety and technical implications of the Observational Method and also discusses the contractual framework in which it can be used (Nicholson et al (1997)). It aims to show that design by Observational Method is no less safe than conventional design (termed prescriptive design) provided that the requirements of the method are rigorously followed. One important technical conclusion in the report relates to calculations carried out at various stages during the application of the Observational Method. The authors conclude that it is preferable to retain the same nominal factors of safety in these calculations as in prescribed design, but to apply those factors to parameter values which are less cautious than the characteristic values of Eurocode 7. Hence the main change they make to supporting calculations is to start from best estimate values rather than the "cautious assessments" of Eurocode 7. This may either be regarded as a decision not to adopt characteristic values for this purpose, or it could be said that characteristic values are defined differently when used with the observational method. This approach is different from that taken in the current draft of Eurocode 7.

In the discussion session, some of the questions noted above were taken up by panellists. Open discussion related mainly to problems of interpretation and application of Eurocode 7.

REFERENCES

BS8002. 1994. British Standards Institution Code of Practice for Earth Retaining Structures.

Burland J.B. & Hancock, R.J.R. 1977. Underground car park at the House of Commons: Geotechnical aspects. Struct. Engr., vol. 55, 87-100.

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

Nicholson, D. P., Tse, C-M & Penny, C. 1997. The Observational Method in ground engineering: principles and applications. CIRIA Funders' Report CP/49. Construction Industry Research and Information Association.

Orr, T. L. L. & O'Brien. E. J. 1996. Examples of the application of Eurocode 1. IABSE Colloquium: Basis of design and actions on structures - background and application of Eurocode 1, Delft.

Simpson B. 1996. Basis of design in Eurocode 7. IABSE Colloquium - Basis of design and actions on structures, Delft.