Panel discussion: Limit state design – A South African perspective

Débat de spécialistes: Le dimensionnement aux états-limites – L'optique sud-africaine

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ABSTRACT: This discussion contribution describes the progress made towards implementation of limit state design techniques in the field of geotechnical engineering in South Africa. It identifies some of the impediments to the acceptance of the Eurocodes and how South African engineers are addressing these issues.

RESUME: Cette contribution à la discussion décrit les progrès faits vers l'éxècution des téchniques de conception d'états limites dans le domaine du génie géotechnique en Afrique du Sud. Elle identifie certains des obstacles à l'acceptation des Eurocodes et la façon dont les ingénieurs sud-africains sont en train d'aborder ces problèmes.

1 INTRODUCTION

Being situated as it is at the southern tip of the "dark continent", change is sometimes slow in coming to South Africa. This has certainly been the case with the introduction of limit state design in geotechnical engineering.

In 1995, a decision was taken by the Geotechnical Division of the South African Institution of Civil Engineers to adopt Eurocode 7 as a standard for limit state design in geotechnical design. The intention was that engineers should use this code in parallel with existing design methods for a trial period of three or four years much the same as is happening in Europe at present. However, two years down the line, the profession is still debating the merits of the Eurocodes and very few designers are implementing Eurocode 7 in practice.

This paper explores some of the reasons for this situation.

2 EUROCODE 1: THE LOAD FACTOR DEBATE

In the early 1980's, the Structural Division of the South African Institution of Civil Engineers set up a working group to make recommendations on limit state formulations in South African structural codes. The objectives of this working group were to select a limit states model, to identify the principles and assumptions on which future materials codes should be based, to define partial load factors and load combinations which would be common to all materials codes and to identify the criteria for calibration of the materials codes (Kemp, 1986).

During their deliberations, this working group studied the limit states proposals being developed in Europe, the United States, Canada and Australia at the time. Particular attention was paid to the selection of partial load factors which would yield load effects with a uniform "probability of being exceeded" over the full range of load combinations. On the basis of this work, the load factors given in Table 1 were adopted and incorporated into the 1989 version SABS 0160 (Milford, 1986a).

Probably the most striking feature of the SABS 0160 approach is the relatively low factor applied to dead load namely 1,2. The reason for this selection is illustrated in Figures 1a and 1b which chart the variation in "load index" across the range of load combinations for the load factors given in SABS 0160 and for those given in the then-current British concrete code, CP 110. (The load index is the negative logarithm to base 10 of the probability of exceeding a given load. A load index of 2,0 corresponds to a 1% probability.)

Table 1. SABS 0160 partial load factors and comparable values from EC 1.

Type of Load	SABS 0160	Eurocode 1
Maximum self weight acting in isolation	1,5	
Maximum self weight acting in combination	1,2	1,35 (Case B)
Minimum self weight	0,9	0,90 (Case B)
Wind - general	1,3	
- freestanding	1,5	1,50 (Case A) 1,35 (Case B)
Floor loads and other imposed loading	1,6	1,30 (Case C)

From these figures, it can be seen that the load factors given in SABS 0160 result in a load index of approximately 2,0 over the full range of load combinations with the exception of high dead loads combined with low wind loads. Apart from this exception, the variation in load index is lower than that exhibited by the load factors given in CP 110. In the South African code, the load factors for ultimate and serviceability limit states were chosen to achieve 50 year probabilities of exceedence of 1% and 10% (load indices of 2,0 and 1,0) respectively. Although these probabilities were derived using statistical distributions for loading pertaining mainly to buildings, they were considered to be sufficiently representative for use in the design of other types of structures.

The South African working group went on to show that the uniformity of the safety index had advantages for the calibration of the materials codes. A target reliability index of 3,0 (as is appropriate for most ductile structures) can be achieved by adopting partial material factors which yield design values with a 1% probability of a worse value occurring. This conclusion was shown to be valid for material properties with coefficients of variation ranging from 10% - 30% (Milford, 1986b & 1988).

After laying of such a solid foundation for the calibration of materials codes, South African engineers are understandably

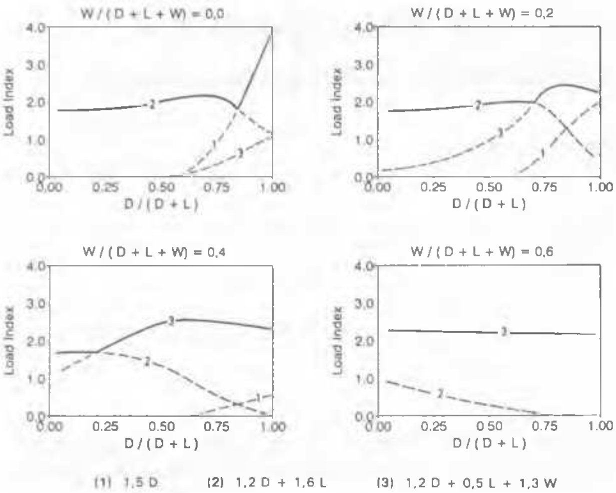


Figure 1a Load index for various load combinations given in SABS 0160.

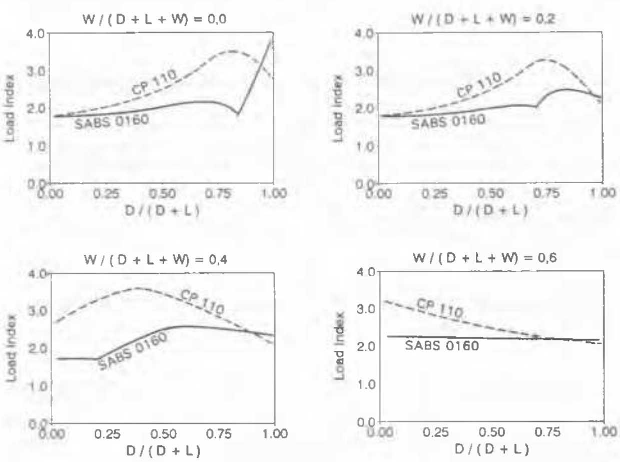


Figure 1b Comparison of load indices - SABS 0160 and CP 110.

reluctant to abandon their loading code. The low dead load factor does, however, prohibit the use of the load factors given in SABS 0160 in conjunction with materials codes from abroad such as EC 2 (Design of Concrete Structures) particularly where brittle modes of failure are concerned.

3 LOAD COMBINATIONS A, B AND C

One of the greatest impediments to the introduction of limit state design in geotechnical engineering is that the self weight of the soil governs not only the loading on a "geotechnical structure" but also its design resistance. This is in contrast to many other forms of structure where the resistance (as defined in the materials codes) is independent of the loading (determined by the loading code).

In order to address this problem, Eurocode 1 has introduced a third load case. The first two load cases deal with loss static equilibrium and failure of the structure respectively. The third load case deals with failure in the ground. For a structure subjected to a dead load (permanent action, G_k) and a single live load (variable action, Q_k), the three load cases are as follows:

Case A
(Loss of static equilibrium) $0,9\ G_k + 1,5\ Q_k$

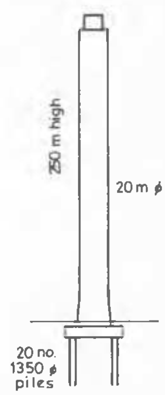
Case B
(Failure of structure) $1,35\ G_k + 1,5\ Q_k$

Case C
(Failure in ground) $1,0\ G_k + 1,3\ Q_k$

Even without the benefit of statistical analysis, it is evident that the load indices attached to these load combinations will differ significantly. As a result, different partial material factors have to be applied to the soil properties in each cases to achieve a uniform target reliability. This situation is reflected in Table 2.1 of Eurocode 7.

Apart from the variation in the load index, the consideration of these three load cases gives rise to problems with the analysis of certain structures, earth retaining structures in particular. For example, the depth of embedment of a cantilever retaining wall is likely to be governed by Case C. The structural strength of the wall should, according to EC 1, be analysed using Case B. However, the application of load Case B to the depth of wall derived from Case C results in a wall which is no longer in static equilibrium under the action of active and passive pressures.

A further point, possibly on the pedantic side, is that structural integrity is not always governed by Case B. For example, consider the case of the 250m high chimney shown in Figure 2. The critical design condition for this chimney was that of cross-wind oscillation during construction with the windshield at a height of 225m. The table on the right hand side of this figure gives the maximum and minimum pile loads, bending moments and the required area of reinforcement for Cases A and B. In this instance, load Case A is shown to govern the structural design of the piles. Under the loads exerted by the completed chimney, Case B was more critical (this time with the piles in compression) but the reinforcement requirements were still governed by Case A during construction.



Load Case	Case A	Case B
During Construction		
Max. Pile Load	19,1MN	22,3MN
Min. Pile Load	-6,43MN	-3,25MN
Pile Moment	1,60MN	1,60MN
Reinforcement	m	m
	1,5%	1,1%
Completed Chimney		
Max. Pile Load	20,1MN	25,2MN
Min. Pile Load	0,31MN	2,76MN
Pile Moment	1,17MN	1,17MN
Reinforcement	m	m
	0,3%	0,7%

Figure 2. Pile loads and reinforcement requirement for 250m high chimney.

Upon reflection, it is logical that Case A should be critical particularly during construction as this is a situation of potential loss of static equilibrium (i.e. overturning). The only "unusual" aspect of this situation is that the resistance to overturning is provided by structural elements, i.e. the piles, rather than by the ground. In the light of this example, the impression should not be created in EC 1 that the strength of the structural elements is insignificant for Case A (See Table 9.2) or by EC 7 that Case A applies only to problems of buoyancy (See Clause 2.4.2[15]).

4 CHARACTERISTIC MATERIAL PROPERTIES

There has been considerable debate over the years about the selection of characteristic values for material properties. The consensus appears to be that the characteristic value for a material property should be a cautious estimate of the value likely to govern the occurrence of a limit state. The selection of this value should take into account not only the variation in

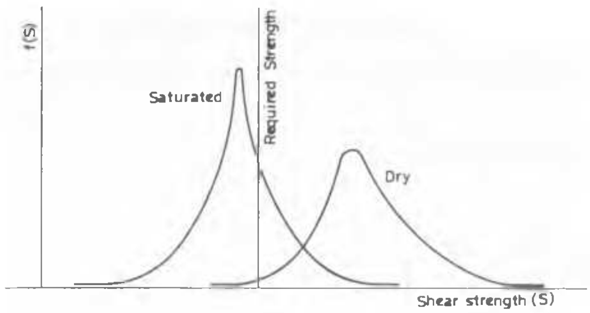


Figure 3. Shear strength of residual granite in saturated and partially saturated conditions.

material properties but also the volume of soil through which the failure surface passes and the averaging effect which this will have on the material properties (Simpson 1984).

In the case of relatively uniform soil profiles where an adequate site investigation has been undertaken, the selection of characteristic values should not pose an undue problem. However, under typical South African conditions, this is seldom the case. Frequently, geotechnical engineers have to contend with a grossly inadequate number of test results (or no tests at all), poor testing procedures and the added variability introduced by partial saturation and relict structure of many typical South African soils.

The issues of inadequate testing and poor test procedures are nothing new and are not unique to the limit state design approach. Similar problems would be faced with any other design method. The problems with partially saturated materials and materials with relict structures are slightly more fundamental.

The extent to which South African Geotechnical Engineers rely on a partial saturation is clearly illustrated by their approach to excavation stability. For example, numerous temporary excavations, some as deep as 9m, have been excavated into the partially saturated residual granites of the northern suburbs of Johannesburg at inclinations of 70° to the horizontal. When tested in a saturated condition, the residual granite has a friction angle of 36° and a cohesion of zero. It is thus evident that the stability of these excavations depends largely on the strength derived from a partial saturation.

This poses a problem for the selection of characteristic values to be used in the design process. Figure 3 depicts the "shear strength" of the residual granite under dry and saturated conditions. If the designer chooses to use the strength of the partially saturated material, he could not doubt show that the excavation is stable. However, he runs the risk that the material may become saturated during the design life of the excavation. The shear strength would then drop to the saturated strength probably resulting in the failure of the excavation.

In this example, it is not the variability of the material property (mainly the shear strength) which governs the selection of the characteristic value. It is the probability of an occurrence of an external event, namely saturation of the soil, which determines the reliability of the structure. Obviously such external events cannot be taken into account by the application of a partial material factor.

5 WHERE TO NOW?

From the debate which has been taking place in South Africa over the past twelve months, there appear to be two issues which need to be resolved before meaningful progress can be made. Firstly, is it necessary to apply limit state design methods to all types of geotechnical problems? Geotechnical engineers are acutely aware of the variability of the parameters used in the design process - perhaps more so than engineers from other branches of civil engineering. They thus appreciate one of the major advantages of limit state design, namely, taking account of the uncertainty associated with each individual parameter at the input stage of the design process rather than merely factoring the end result.

There is generally no problem designing pile groups or spread footings using limit state design methods. This is little more than an extension of the structural design process to which the limit state design method is ideally suited. However, when it comes to the design of a geotechnical structure (eg. slopes, embankments, etc.), many geotechnical engineers see the application of limit state design as somewhat contrived. They would prefer to take account of the variability of the input parameters in a more direct manner by, for example, a sensitivity analysis, the point estimate method or a fully-fledged statistical analysis.

The second issue to be resolved is the selection of load factors and the formulation of load combinations. Do we follow the Eurocode route, stick to SABS 0160 or look for yet another code? In order to address this problem, the South African Institution of Civil Engineers has arranged a seminar on loading to be held in October 1998. This seminar will be addressed by representatives from Europe, North America and Australia as well as by advocates of local practice.

It is hoped that by the end of 1998 South African civil engineers will have achieved consensus between disciplines on the direction to go and can then commence the arduous task of implementing this decision.

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