LSD 2000 : International Workshop on Limit State Design in Geotechnical Engineering
Melbourne, Australia. 18 November 2000

National Report on Limit State Design in Geotechnical Engineering : Belgium

C. Bauduin
Besix, Brussels; University of Brussels, Belgium

J. Maertens
Jan Maertens BVBA; Catholic University of Leuven, Belgium

A. Bolle
University of Liège, Belgium
M. Devos
Belgian Building Research Institute, Brussels, Belgium

P. Mengé
Geotechnics division, Ministry of Flanders; University of Ghent , Belgium

H. Raedschelders
Formerly Catholic University of Leuven, Belgium

S. Vandemeulebroecke
Technical Control Office Seco, Brussels

1 COUNTRY, REGION AND REGIONAL GEOLOGY

Belgium belongs to the European Union. The Belgian territory is rather flat, with a continuous transition from the plain at the North Sea to the highlands of the Ardennes culminating at about 700 m above sea level. In the North of the country, the stratigraphy has been governed by fluctuations of the coastal line. The deep bedrock is covered by alternating tertiary clay, sand and (occasionally) gravel sediments with thickness up to hundred of meters. The quaternary Pleistocene formations have been influenced by glacial periods, giving rise to the formation of marine, coastal, river, lake or wind deposits sand, clay, peat and silt (loess). Holocene erosion and river sedimentation, as well as human activities, have further influenced the actual subsurface. In the South, the bedrock is often found at rather shallow depths, overlain by colluvium layers consisting of weathered rocks and river sediments.
As a result of the geological history, one can find in the North a wide variety in stratigraphy, with complicated and heterogeneous soil layer patterns. Geotechnical engineering includes design in such different soils, ranging from highly organic peat to overconsolidated clays and sands, loose to very dense sands and silts, all clean or with variable degree of clay admixture.

2 CODES OF PRACTICE

2.1 Current practice

Belgium has no strong tradition in stringent codified design for geotechnical structures. However, there is a body of "generally accepted design practices" in most field of geotechnical engineering. Governmental authorities have some own rules and requirements. Some differences can be noted between the calculation methods imposed and the sets of specifications and provisions.
By tradition, the factors of safety are applied within a deterministic framework, i.e. they are globally applied to the components of the resistance:

$$F \leq \Sigma R_i/s_i$$

Where:

F : effect of the actions (representative values)

R_i : component i to the resistance

s_i : global safety factor applied to component i to the resistance

The resistance term is quite always obtained by using analytical or semi-empirical methods. The resistance is calculated starting from conservative estimates of the soil properties as derived from in-situ test or obtained from laboratory tests or by experience. Load tests, full scale tests etc are rarely used for design. Load tests may be used for control of piles and anchors. For piles, these load tests are usually performed to 1.5 times service load.
Indicative values of safety factors for different types of geotechnical structures are summarized in the table below. The values are indicative for common structures, without abnormal risks and with no abnormal soil or load conditions. Lower values may be used for temporary structures or accidental load combinations.

Table 1: Indicative values of global safety factors as applied in the existing deterministic approach

Type of structure		**Design rule**	**Safety factor**[(1)]
Shallow foundation:	Bearing capacity [(2)]	Analytical,...	2-3
		Semi-empirical: pressure meter	3
Gravity wall:	Bearing capacity [(2)]	Analytical	2-2.5
	Sliding	Analytical	1.5-2
	(Toppling; Only relevant foundation on rock)	Analytical	1.5
Pile compression:	Base resistance	Semi-empirical : CPT	2 [(3)]
	Shaft resistance	Semi-empirical : CPT	3 [(3)]
Pile in tension	Single pile	Semi-empirical : CPT	3-4
Embedded wall:	Passive resistance, final stage Passive resistance, temporary stage	Analytical (Coulomb; Caquot-Kerisel) Analytical (Coulomb; Caquot-Kerisel)	1.8-2 1.4-1.5
Slope stability	Final stage	Bishop simplified	1.3-1.4

When considering the values above, one should pay attention to the following:

1. The values are the result of a long tradition of balance between investment and safety which have proven to be successful, rather than the result of deep-going theoretical analyses;
2. Ultimate bearing capacity analyses are accompanied by settlement calculations, especially when the lower values of the indicated range of the safety factors are applied. Sometimes, when the higher values of the safety factors are applied, the serviceability conditions are not explicitly analysed for routine structures on compact sands for which favourable previous experience exists;
3. The values of the safety factors are considered to cover serviceability limit states for piles in sand and overconsolidated clays under routine structures for which favourable previuous experience exists;
4. The required value of the safety factor for a given project is determined to some extent in relation with the selected value for the shear strength parameters and the extent of the soil investigation. The values of the safety factors may be the lower bound when rather conservative values of the soil shear strength parameters are introduced in the calculation model. When a restricted soil investigation is available, (very) conservative values of soil strength parameters are selected or the higher bound of safety factors is applied;
5. The values of the safety factors are related to the design rule applied: much higher values of the safety factors may apply to crude or very simplified rules.

The design practice for axially loaded piles is reported in detail by Holeyman et al. (1997) in the Belgian contribution to ERTC 3 "Design of axially loaded piles-European Practice : Belgian practice".

As the geotechnical profession uses more "generally accepted rules", or rules imposed by Ministries or other institutions instead of stringent codes, their application is not mandatory by law. However, strong deviations from them need a well demonstrated justification of the design.
In some cases, foreign codes may be used to fill a lack in the existing design practice. Some examples of references frequently used are listed below:

Table 2 : examples of foreign codes applied in Belgian practice for some specific design methods

Type of structure	Reference	Origin
Quay wall	Recommendations of the EAU	Germany
Sheetpile	CUR 166	Netherlands
Pile bearing capacity using pressuremeter	DTU 13.2 or Fascicule 62	France

2.2 Future codes: The Structural Eurocodes

Belgium intends to adopt the Structural Eurocodes (EN 1990 to EN 1998). Most ENV 's are accepted at present as Belgian Norm This will lead to the application of a harmonized set of structural design codes (covering loads and load combinations, structural design of concrete, steel... members and connections, geotechnical design etc). The domain of application of this codes will be the one required by the European Community to allow free trade.
At present, most prEN, among which prEN 1997/1 "Geotechnical Design"(1994) are adopted as Belgian Norm NBN. The boxed values of ENV 1997/1 (1994) are taken over, except for piles where the values are still in discussion and will probably be slightly amended.
Once they will be voted, the EN's will be adopted as Belgian Norms. They will be introduced by a national foreword indicating mainly the values of the partial factors to be used. The EN will also be supplemented by national annexes giving guidelines on the application of some specific clauses of the EN. Especially, EN 1997 will be adopted for geotechnical design.

3 DESIGN METHODS

As explained in section 2.1, the present practice of geotechnical engineering is based on the concept of the global safety factor, which aims to cover all sources of uncertainties (including model uncertainty) in one single global factor applied to the calculated resistance.

3.1 Partial factors and load combinations

The introduction of the Eurocodes in the geotechnical design leads to the use of load and material factors. Although it is expected that the EN 1997 will allow for different approaches, Belgium will probably adopt the approach which is the closest to the existing pr ENV 1997. The partial factors on the loads and on the soil parameters are applied as much as at the source of the uncertainty, i.e. on the loads and on the soil strength parameters c' and φ'. The values as proposed in the present state of the prENV 1997/1 (1994) for the relevant approach are indicated in the table below.. For convenience, the names of the "cases" as introduced in ENV 1997 are not modified. The final values of the partial factors have to be adopted in the National Foreword to EN 1997 and may slightly depart from those indicated in the table below; especially, it is expected that the partial factor on effective cohesion intercept will be reduced to the same value as the one on tan φ and thus taken equal to 1.25.

Table 3: Partial factors according to ENV 1997/1 (1994) (Boxed values)

	Load factors			Material factors			
	Permanent favourable	Permanent unfavourable	Variable	tanφ	c	c_u	piles
Case A	0.95	1.00	1.50	1.1	1.3	1.2	
Case B	1.00	1.35	1.5	1.0	1.0	1.0	1.0
Case C	1.0	1.0	1.3	1.25	1.6	1.4	1.3-1.6

The load combinations and ψ factors (taking account of the lower probability of simultaneous occurrence of several variable loads at their characteristic value) will be applied in accordance with the rules given in EN 1990 for Ultimate Limit states and Serviceability limit states. Characteristic values of loads will be taken from EN 1991.
For temporary situations, the national foreword of EN 1990 will probably allow lower partial factors than those indicated above.
For accidental situations the load and the material factors are taken equal or very close to 1.0
For serviceability limit states the load and material factors are taken equal to 1.0.

3.2 Design method

The main features of the design method are:

- The design has to be checked for static equilibrium by the case A. Examples of checks of static equilibrium are: buoyancy of structures, bottom heave, uplift of a group of piles subjected to upwards tensile force, toppling of a rigid structure on a rock base etc
- The design (geotechnical sizing and structural design of members) has to be checked for two sets of load and material factors: set B and set C. In principle, this requires two calculations: one set of factors will really govern the considered part of the design, the other is to a check for the other set of factors. Where it is clear that one set is critical to the design, it will not be necessary to perform calculations for the other. Often the sizing of the geotechnical structure will be governed by the set of factors of case C. Often the structural design of the members will be governed by the factors of case B. In practice, the dimensions (eg of the geotechnical structure, of the member or its reinforcement etc) will be assessed by using the presupposed "governing case" and checked for the other case. Guidance will be given to identify the governing case, see table below. Where it is obvious which is the governing case, calculations for the other are even not necessary.

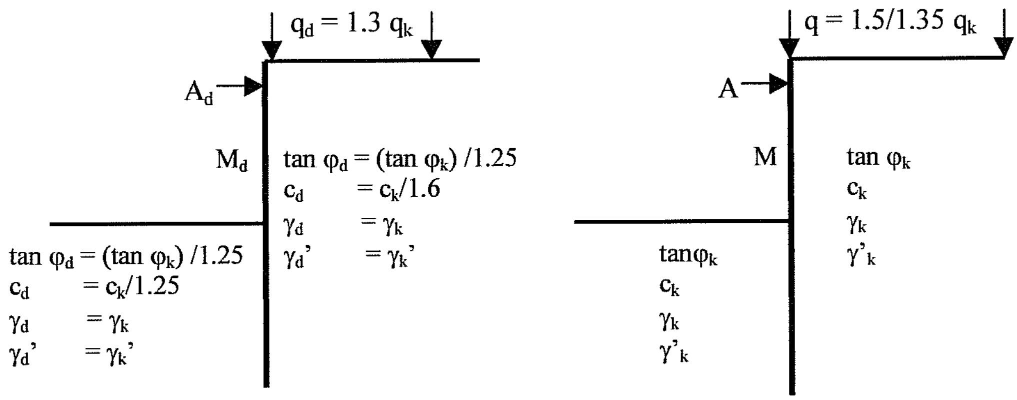
Table 4: guidelines to help the selection of the case governing usually the design for different types of structures; other case only to be checked

Type of structure	Design situation	Case usually governing
Spread footing, vertical or slightly inclined load	Bearing capacity: Sizing of footing	C
	Structural design	B
Axially loaded piles [1]	Geotechnical sizing	C
Axially loaded piles	Structural design	B
Gravity wall	sizing	C
	Structural design	B
Embedded wall	Embedment depth	C
	Structural design	No guidance
Slope	Overall stability	C

[1] For variable load with opposite sign of permanent load and with absolute value larger than 0.7 absolute value of permanent load, case B governs geotechnical design

In both cases B and C (and A), the structural design is performed for the obtained design values of internal members solicitation using the partial factors of the relevant material Eurocode (concrete member: EN 1992; steel member: EN 1993 etc)

Where the application of the load factor could lead to design values which are unreasonable, load factors may be treated as model factors. For example, design values of earth pressures in Case B for retaining structures (embedded walls) are often treated in this way: the case B calculation is performed using unfactored loads (except correction for variable load: 1.5/1.35 = 1.11) and shear strength parameters and the obtained internal forces (and anchor reactions if any) are multiplied by the load factor 1.35 to obtain design values. The figure below illustrates the procedure for an embedded wall.



Case C | Case B

Calculation delivers: Embedment depth
Design value of moment
Design value of anchor force

Calculation checks: Embedment found in case C
Calculation delivers: M, leading to $M_d = 1.35 * M$
A, leading to $A_d = 1.35 * A$

Remark that the ENV 1997 cannot be classified as fully "material factoring approach" or "load and resistance factoring approach": some design situations are treated by MFA and other by LRFA; both may even be mixed. Following general guidance will be used:

- In analytical and numerical calculations, the partial factors will be applied right at the source, i.e. immediately on the shear resistance parameters and on the loads
- When the resistance is obtained as a derived value from some in-situ measurements (bearing capacity of piles or spread foundations deduced from CPT or PMT results using semi-empirical methods, charts etc), the partial factor is applied on the derived resistance instead of on the measured parameter (thus applying the partial factor on the bearing capacity instead of on the measured cone resistance, limit pressure...)

Basic consideration for proposing the above guidance, is that analytical methods give resistances which increase more than the increase of the shear resistance, while semi-empirical methods deliver resistances which increase slower than the increase of the measured parameter.

In accordance with ENV 1997, model factors may be introduced to ensure that the calculation results are at the side of safety or to avoid a too large gap with the results which should be obtained with existing practice. It is considered preferable to use a restricted number of model factors in order to avoid sources of mistakes. However, model factors will be introduced for the design of spread foundations in undrained situation and for the design of the bearing capacity of piles. Their values are not yet finally established, but it is expected that they will be around 1.4.

3.3 Comparison with existing practice

The values of the material factors (and model factors) are calibrated mainly by comparison with the existing design, taking account of the concept of "characteristic values". The following have been observed through the comparative calculations and parametric analyses (characteristic values being the same for the deterministic and semi-probabilistic calculations):

- The ULS design of vertically loaded (or slightly inclined) spread foundations become a little bit more economical than when using a global factor of 2.5; service limit states are not "automatically" covered by the ULS calculations, especially for soils with friction angles lower than 30°
- The ULS design of spread foundations in undrained situation demands for a model factor on the resistance to avoid an unacceptable gap with the existing practice
- The ULS design of gravity walls turns out to be a little bit more conservative than the existing practice, probably due to the fact that as well as the fill material as the bearing soil are factorised together in the ULS calculations
- Although there is a wide variety of design methods and safety concepts for embedded walls, the ULS design according to ENV seems to fit with the existing practice, being in some circumstances slightly more economic
- The ULS design of axially loaded pile foundations is usually slightly more economic, especially when three or more field tests have been performed

No true probabilistic analyses underlay the values of the factors. The safety index for some typical spread foundations has been calculated starting from the indicated partial factors. The obtained reliability index was sufficient and showed to be much more constant through the range of φ values expected for spread foundations than when a global safety factor concept is used.

4 MATERIAL PROPERTIES

4.1 Characteristic values

The current design practice for the choice of the value of material parameter leaves to the discretion of those involved in the design. The value of the soil parameters is usually derived from field test (mainly CPT) supplemented, for larger projects, by laboratory tests. When available, results of tests in the same formations are considered. The value introduced in the calculation is often selected as a conservative estimate of the mean value. The selection of the value for the calculation relies on engineering judgement to account for:

- Knowledge of the local geological conditions, allowing to establish the soil layering and to introduce previous experience;
- Observed or known variability of the ground parameter;
- Amount and reliability of the information: number of tests, type of sampling (local or regional population), previous knowledge;
- Difference between test and behaviour of the soil in the real structure; scale effects between test and real soil mass; time effects which may lead to different behaviour in test conditions and in reality or which may alter the value of the soil parameter;
- Adequacy between the value of the geotechnical parameter and the limit state considered (e.g. compatibility of strains, relation with stress level…);
- Ductility or brittleness of the soil;
- Is the process (and thus the accompanying limit state) governed by an average value or by the lowest values of the soil parameter (think e.g. on compression in layered soil: the deformation is governed by the most compressible layers, not by mean values);
- Does the volume of soil involved in the limit state considered or the strength and stiffness of the structure allow for transfer of stresses from weak to strong spots (in which case a mean value may govern the limit state) or is the limit state governed by a local weak spot (in which case the (stochastically occurring) low or point value govern the limit state);

The Eurocode defines the "characteristic value" as "a cautious estimate of the value governing the behaviour of the soil at the limit state considered". This definition is not in conflict with the existing practice.

The national application document will allow for the use of statistical methods as an objective, but not mandatory tool to support the selection of characteristic values. The statistical formulas will be devised such that "the calculated probability of a worse value governing the occurrence of a limit state is not greater than 5%".

Due to the number of elements to be taken into account, the first step in a statistical approach is to select the appropriate method. The scheme below is an illustration of flow charts to help for the choice of an adequate statistical method, taking account of many of above mentioned items to be considered.

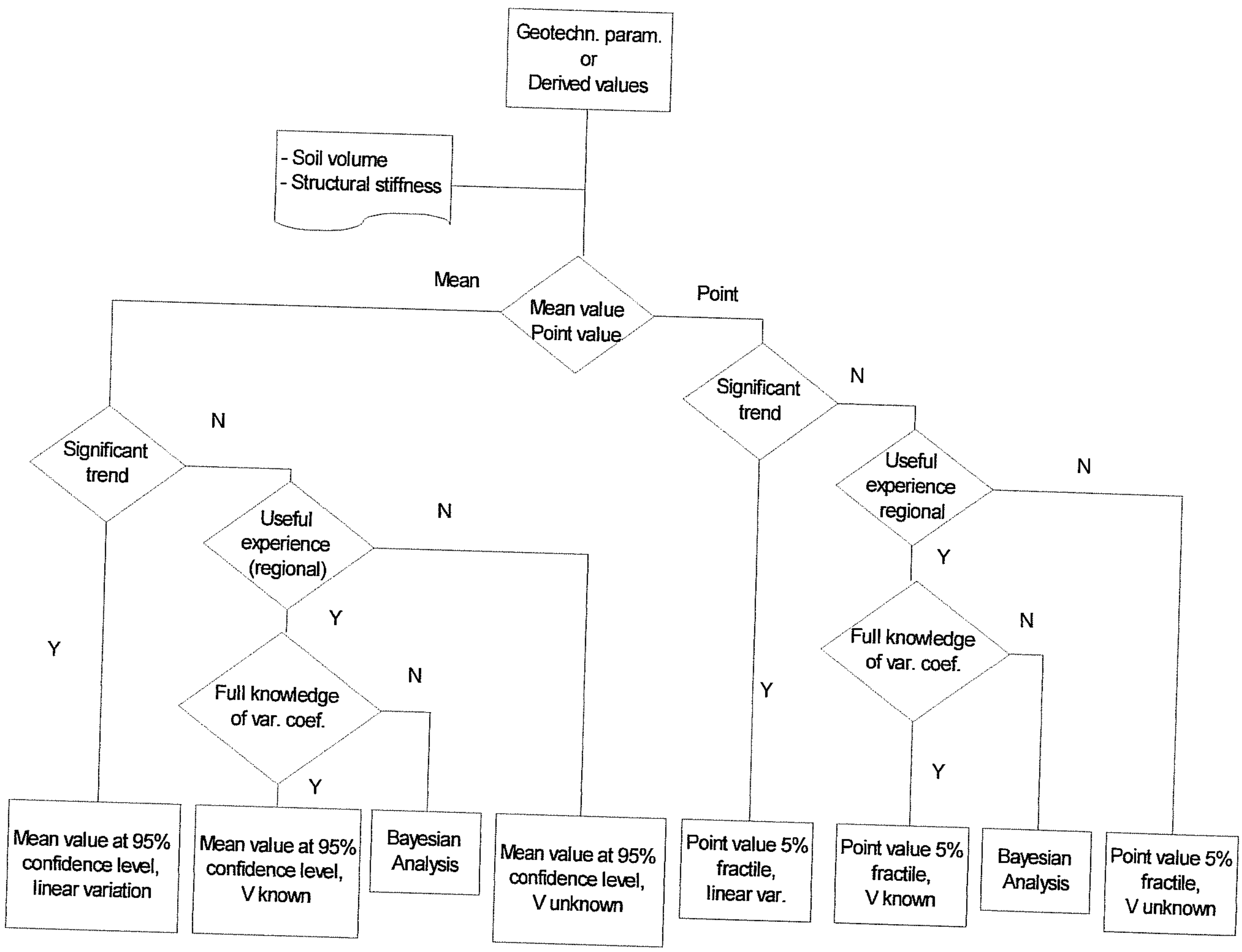


Figure 2: Flow Chart to help selection of the statistical formula in case of a local sampling

As an example, for layers which can be considered as homogeneous (thus without significant systematic trend with depth or on horizontal direction), the characteristic value of a geotechnical parameter can be assessed out of a set of N individual values according to:

$$X_k = X_{mean} * (1 - k_n * V_X)$$

Where:

X_{mean} :arithmetical value of the geotechnical parameter out of the N results

V_X : coefficient of variation of the parameter X

k_n :a statistical coefficient taking account of:

- The number of tests
- The volume of soil involved and the ability to transfer
- The type of sample population: local population only or population and relevant experience
- The statistical level of confidence of the assessed characteristic value (95%)

Two extreme situations may be considered concerning the coefficient of variation:

- The coefficient of variation is not known a-priori and must be estimated from the arithmetical mean and the standard deviation sample as:

$$V_x = \sigma_X / X_{mean}$$

- The coefficient of variation is well known a-priori from pre-knowledge; pre-knowledge might be found from the evaluation of previous tests in comparable situations. What is comparable situation is determined by engineering judgement

When the soil volume in the limit state considered is very large compared to the length of fluctuations of the property, and when it may be assumed that the behaviour is governed by the mean value, one can use the k_n values of the table below:
k_n values for 95% reliable estimate of the mean value (homogeneous population).

N	3	4	5	6	8	10	20	30	∞
V_X known	0.95	0.82	0.74	0.67	0.58	0.52	0.37	0.30	0
V_X not known	1.69	1.18	0.95	0.82	0.66	0.57	0.41	0.33	0

When the soil volume in the limit state considered is very small compared to the length of fluctuations of the property and when it may be assumed that the behaviour is governed by stochastically occurring point values, one can use the k_n values of the table below:
k_n values for 5% fractile of a homogeneous sample.

N	3	4	5	6	8	10	20	30	∞
V_X known	1.89	1.83	1.80	1.77	1.74	1.72	1.68	1.67	1.64
V_X not known	3.37	2.63	2.33	2.18	2.00	1.92	1.76	1.73	1.64

4.2 Characteristic values of pile bearing capacity

According to prEN 1997 (draft version, august 2000), the characteristic value of the axial pile bearing resistance as obtained from pile load tests or derived using semi-empirical methods will be selected as:

$$R_k = \min(R_{min}/\xi_1 ; R_{mean}/\xi_2)$$

Where R_{min} : minimum ULS bearing resistance as obtained from pile load tests or derived from CPT

R_{mean} : mean of the ULS bearing resistance as obtained from pile load tests or derived from CPT

ξ_1 and ξ_2 are statistical factors, depending of the number of pile load tests or the number of tested profiles (number of CPT) and the ability of the structure to transfer loads from weaker to stronger piles. Note that ξ_1 and ξ_2 may be different for pile load tests and for field tests. Their values are established so that when the coefficient of variation of the bearing capacity is lower than about 10 to 15 %, the mean value governs; if it is higher, the lowest value governs.

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