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Eurocode 1, Basis of Design, Background Information

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Summary

This paper gives a review of the background philosophy of Eurocode 1, Basis of Design. The main ideas behind the various parts of the document are discussed. The emphasize is put on the reliability aspects and the partial factor method. Recommendations for future developments are given.

1. Introduction

Eurocode 1, Basis of Design [1], like most present day codes, is formulated in terms of the Limit State Approach and the Partial Factor Method. The main objectives of the code are to prevent the structure, with appropriate degrees of reliability, to reach states of collapse or inserviceability during its intended life time. In this paper we will address a number of the key words mentioned in this introduction. For more extended background information the reader is referred to [2] and [3]. A general introduction can be found in [4].

2. The limit state approach

The Eurocodes are based on the limit state approach. A limit state is the demarcation between desired and adverse states of the structure. Two main categories of limit states are distinguished: Ultimate limit states and Serviceability limit states.

Ultimate limit states are associated with collapse, either on the level of structural members or, in particular for accidental limit states, on the level of a structural system.

Serviceability limit states are associated with the usefulness of the structure. A further distinction is possible into reversible and irreversible limit states [5] [6]. In the first case the limit state is no longer exceeded when the actions are removed, as for instance may happen for large elastic deflections and excessive vibrations. For irreversible limit states the exceedance will remain even when the actions are removed, as is for instance the case for most cracks.

By definition, structural design is concerned with situations which may cause the exceedance of the limit states. In principle these situations may be classified as follows:

1. Extreme unfavourable combination of actions and structural properties,
2. Abnormal but foreseeable actions like collisions or explosions
3. Errors during design, execution or use
4. Influences that are not foreseen.

The first two categories are the situations that normally are considered in standard design. The usual procedure is to define a set of load cases and to verify that the limit state is not exceeded using design values for actions and resistance parameters. Guidance for this procedure can be found in Basis of Design and the other Eurocodes. In some cases, however, if the calculations are extremely difficult or if the theory is not well developed, the calculations may be replaced by empirical "deemed to satisfy rules". Examples are explosions for ordinary buildings or some durability effects.

The last two categories of hazardous situations are much more difficult to tackle. The possibility of errors should be reduced by a proper quality assurance system. The details of such a system are outside the scope of Basis of Design. Unforeseen actions require some kind of robustness of the structure. This vague requirement may be relieved for cases when in the normal design all kinds of hazard scenarios are taken into account explicitly.

3. Reliability aspects

According to Eurocode 1, Basis of Design, Section 2.1 limit states requirements "*shall be fulfilled with appropriate degrees of reliability*". This means that the degree of reliability should be adopted to suit the type and use of the structure as well as the design situation under consideration. The choice of the level of reliability should take into account both the possible consequences of failure and the amount of expense and effort required to reduce the failure probability.

Obvious examples of reliability differentiation are the distinction between the ultimate and serviceability limit states or between failure modes with warning and without warning. The statement in Basis of Design may also mean that one accepts other reliability levels for live load as for accidental and seismic actions or that the failure probability for an agricultural building is higher than for a large span bridge. Section 2.2 of the present version of Basis of Design offers these possibilities in principle. However, the idea is not elaborated and it is not clear where it has been applied in the Eurocodes or where should have been applied.

Once a level of reliability has been specified it may be achieved in various ways:

-prevention of hazards
-protection against hazards,
-ductile and redundant structural system behaviour
-structural strength on a member level

The Eurocodes deal primarily with the structural strength on a member level. Nonstructural measures and structural system behaviour are seldom considered in conjunction. According to Basis of Design, Annex A, the standard reliability corresponds formally to a life time reliability index $\beta = 3.8$ for single mode, single member failure. This corresponds to a life time failure probability of 0.0001. For irreversible serviceability limit the target index $\beta = 1.5$, corresponding to an exceedance probability of 0.07. These values have been established on the basis of example calculations in various countries. Human errors and unforeseen actions were not taken into account. The consequences of these numbers will be considered in section 5.

4. Working life

The working life is an important design parameter. It makes quite a difference whether a structure is meant for 100 years or only for a few months. Fatigue, corrosion and other deterioration mechanisms may play a role only in the long term. Further, in the case of purely economical optimisation, it can be proven that the design values of variable actions may depend on the intended working life. This aspect is fully neglected in the present version of Basis of Design. It should be noted that the dependence of the design loads on the intended working life time is no longer valid when human safety aspects are dominant.

In general, present knowledge is insufficient to enable a sharp prediction of the structural condition during its design working life. The behaviour of materials and structures over extended periods of time can only be estimated roughly. The best option to deal with this uncertainty is to design in such a way that inspection and repair can be carried out. Lack of sufficient durability may then be conceived as a serviceability problem. Consequently one may adopt the corresponding lower reliability levels. If, however, an important structural member cannot be inspected, the effects of deterioration should be included in the Ultimate Limit State analysis [7]. In EC3, Part 2, the fatigue rules have been based in these ideas.

5. The Partial factor method

According to the partial factor method, design values for loads and resistance's are determined via respectively:

$$X_d = \gamma_F \Psi_i X_k \quad \text{and} \quad X_d = X_k / \gamma_m$$

X_d = design value
X_k = characteristic value
γ = partial factor (if present)
Ψ_i = reduction factor for variable actions (if present; i may be 0, 1 or 2)

In the case of the resistance, X may be a single variable or the resistance of a whole element. The characteristic value is normally aimed at the 5 percent lower (and sometimes upper) fractile.

Characteristic values of permanent actions are equal to the mean value. If the variability is large, upper and lower fractiles may be defined. For variable actions one usually takes values with a return period equal to a reference period of 50 years. The ψ values are intended to give reductions for the design values of variable actions to be used in the various ULS and SLS load combinations. For more information see [2], [3] and [8].

According to the theory of structural reliability [9] [10], design values should follow from

$$P(X < X_d) = \Phi(-\alpha\beta)$$

Φ = distribution function of the standardised normal distribution
α = probabilistic influence factor
β = reliability index.

As an example, for a Gaussian distribution this leads to:

$$X_d = \mu(X) - \alpha\,\beta\,\sigma(X)$$

Similar expressions exist for other distributions. Once the design value has been established, the partial factor and ψ factors may follow from $\gamma\ \psi = X_d / X_k$ for loads and $\gamma = X_k / X_d$ for the resistance.

The mean μ and standard deviation σ in the expression for X_d should follow, as far as possible, from experiments and field observations. This seems to be the only rational background for structural design decisions. Unfortunately, in most cases only a limited and not fully adequate set of data is available. In those cases data need to be supplemented by engineering judgement.

In the absence of a more detailed risk study, the value $\beta = 3.8$, mentioned earlier, may be used. According to ISO-2394 the values of α may follow from Table 1. These values are also presented in EC1, Basis of Design, Annex A. Of course one might take all α-values equal to 1.0, but that is extremely conservative. In Table 1 a distinction is made between load and resistance parameters and between dominant and non other variables. In the case of combining time variant loads the above theory is not enough. One needs a more refined analysis on the basis of stochastic processes.

Table 1: Influence coefficients α for design values [9]

	dominant	others
resistance parameters	$\alpha = +0.8$	$\alpha = +0.32$
action parameters	$\alpha = -0.7$	$\alpha = -0.28$

As a simple example, consider a building element having a coefficient of variation σ/μ = 0.10. If we may assume a normal distribution, the design value follows from:

$$X_d = \mu(X) - 0.8 * 3.8 * 0.10 * \mu(X) = 0.70\ \mu(X)$$

The 5 percent characteristic value is equal to:

$$X_k = \mu(X) - 1.64\,\sigma(X) = 0.84\,\mu(X)$$

This leads to $\gamma_m = X_k / X_d = 0.84 / 0.70 = 1.20$.

In Eurocode 1 the values of γ_F and Ψ_0, however, have not been determined according to the above theory. Most values have been found by (rough) calibration to earlier design methods that have proved to be successful, e.g. the Allowable Stress Methods. Theoretical studies [2] give the impression that the γ values presented in Basis of Design are conservative as far as permanent loads and live loads are concerned. The partial factors for wind and snow on the other hand, are relatively low: theoretical calculations indicate values above 2.0 where Basis of Design prescribes 1.5. Another way of looking at these results is that the target $\beta = 3.8$ is not met for structures where wind or snow is dominant. From the economical point of view this might be an acceptable safety differentiation. Finally the Ψ_0 factors are in general conservative. A detailed discussion is presented in [2].

6. Load combinations according to Eurocode 1, Basis of Design

According to EC1, Basis of Design, Table 9.1 one has to check the Ultimate Limit state for the persistent/transient design situation and for the accidental and seismic design situations. For each design situation the table gives the design values to be used for the permanent load, the variable loads and the accidental actions. As far as the variable loads are concerned there is a subdivision into "dominant" and "other" actions. As each action may be dominant for some design aspect, each variable load should be considered in turn as the dominant action. As an example, for the persistent/transient design situation we have:

permanent load:	$G_d = \gamma_G \; G_k$
prestressing:	$P_d = \gamma_P \; P_k$
dominant variable load:	$Q_d = \gamma_Q \, Q_k$
other variable loads:	$Q_d = \gamma_Q \, \psi_0 \, Q_k$
accidental loads:	$A_d = 0$

In a similar way, according to Table 9.4, one has to check the reversible and irreversible Serviceability Limit State by means of the frequent and characteristic combinations respectively. The two combinations typically have different reliability levels. The quasi permanent combination is intended for long term effects.

The γ and ψ values necessary to determine the design values for the loads, can be found in the Tables 9.2 and 9.4. Especially Table 9.2 has been the subject of many lively discussions. Note that Table 9.2 is only relevant for the Ultimate Limit State as for the Serviceability Limit states all γ values have been put equal to 1.0. Note further that Table 9.2 is intended for buildings and silos only, and not, for instance, for bridges. The information for other structures will be implemented later.

In the Table a distinction is made between the cases A, B and C.

<u>Case A</u> deals with the check for *static equilibrium*. The essential characteristic of this verification is that no strength properties of either building elements or soil is involved. Examples are cantilevering beams if supports cannot take tensile forces (see Figure 2, upper case) and underwater structures like tunnels or docks. The analysis of these limit states require a more detailed and subtle analysis than the usual case with failure. The point is that a part or the total of the structural weight in those cases acts as resistance. So it is important to sort out which part acts as a resistance and which part acts as a load. The

partial factors in these cases should reflect the fact that only the relative differences between these parts are of importance, and not the overall deviations.

Case B is the "normal case" where the *strength of a structural element or cross section* is verified for a certain load case. Note that according to the footnote 3 in Table 9.2 no distinction is made between favourable and unfavourable parts of the permanent loads (e.g. the self weight) as long as it belongs to one source. So in the case of Figure 1, when designing the field AB, it is not necessary to distinguish between the favourable effect of part BC and the unfavourable effect of part AB. One simply takes 1.35 (or 0.9) for both parts.

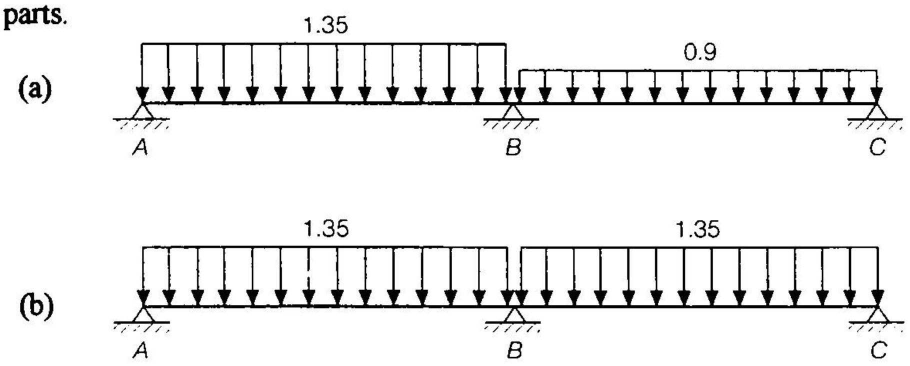

Figure 1: Factors to be used in case B for the permanent loads.
(a) permanent loads from two sources; (b) permanent load from one source

Basis of Design, however, gives no guidance on what should be regarded as "one source" and what should be regarded as two sources. One source obviously is:

- self weight from one material
- ground water pressures from ground water in one soil layer
- water pressure from water in one hydraulic system

What, on the other side to do, if the load effect results from favourable and unfavourable contributions of different building materials. Hans Denver from DGI asked this question to a number of European Colleagues. The differences of opinion were astonishing.

Another important issue in case B is the choice of the partial factor for the unfavourable permanent loads in combination with extreme other permanent and variable loads. In the present version this factor equals 1.35. For some countries this requirement was found to be too strong and a note to equation (9.10) was added. According to this note one may reduce the factor for permanent loads by $\xi = 0.85$ if one of the variable loads is dominant. As a simple example consider the case of one permanent and one variable load, both unfavourable. According to the standard text of Basis of Design one should check for:

$$1.35\, G_k + 1.5\, Q_k$$

If we include the note attached to (9.10) we have:

$$1.35\, G_k + 1.5\, \Psi_0 Q_k \quad \text{and} \quad 1.15\, G_k + 1.5\, Q_k$$

It turns out that for most structures the effective reduction is about 10 %. Note that the factor ξ may be interpreted as a Ψ_0 value for permanent loads.

Case C finally, is relevant if failure in the *soil* plays a role. The difficulty in soil is that it may contribute to the load as well as to the resistance, but in an even more complicated way than the self weight in case A. In addition, soil may have a relatively high degree of uncertainty. According to the theory behind the Partial Factor Method one should then increase the partial factors for soil (make them "dominant", see Table 1) and reduce them on the loading side (make them all "others"). This is what effectively has been done in Table 9.2. However, proper backing from reliability calculations is lacking at the moment.

Note

The distinction between the cases A, B and C may lead to some difficulties in practice. For instance, consider the case of the cantilever beam (Figure 1). In case A one should use a factor 1.10 for the cantilever part and a factor 0.9 for the resisting part. This may lead to the conclusion that the structure is not sufficiently stable and an anchoring device is required. However, if one makes a verification for the strength of the anchor according to case B, one should use the same factor of 1.35 for both parts of the structure. As a result, an anchor of zero strength would be sufficient. This of course is a contradiction. Similar problems may exist between the cases B and C. A possible solution could be to consider cases A and C also in the verification of structural strength. This item needs definitely some more attention before the code is transformed to an EN.

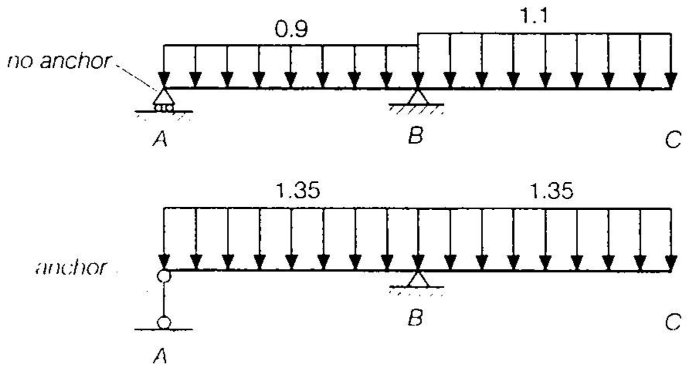

Figure 2: Cantilevered beam with and without anchor at A.

7. Conclusions

The present version of Eurocode 1, Basis of Design, has achieved an important goal. The basic principles of structural design have been harmonised for a large number of countries and (maybe even more important) for a large number of materials (concrete, steel, masonry, timber, aluminium) and disciplines (fire, geotechnics, earth quake, bridge design).

Of course, the harmonisation is not perfect:

- In many cases there still is a lack of uniformity between the various disciplines which cannot be justified from any theoretical point of view.
- Some important items have not been touched, for instance: non-linear calculations, geometrical imperfections, serviceability limit state requirements, durability and working life, reliability differentiation, relation with nonstructural safety measures, influence of working life on design loads, fatigue design, etc.
- Some load cases need to be further developed.

- The Load Combinations and Partial Factors for other structures than buildings (bridges, towers, agriculture structures) should be included.
- Numerical values are, in many cases, more the result of a compromise than of rational and scientific thinking on the basis of experiments and observations. The background of many numbers cannot be properly justified in technical terms, they are an unclear mixture of engineering judgement and experience.

This means that there is still work to be done, first for the transposition to EN and also later. This is especially of importance as also the building industry is not a static one. New materials will be put on the market and new types of buildings will be designed. This means that is will be more and more difficult to rely on intuition and experience built up in the past. In the years to come Basis of Design has to move into a more rational ad professional direction.

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