



Reliability background of the Eurocodes

*Support to the implementation,
harmonization and further development of the Eurocodes*

Vrouwenvelder, T., Dimova, S., Sousa, M.L., Marková, J., Mancini, G., Kuhlmann, U., Taras, A., Jockwer, R., Jäger, W., Schweckendiek, T., Franchin, P., Skejić, D., Sørensen, J.D., Spehl, P., Stacy, M., Feldmann, M., Croce, P., Zhao, B., Tanner, P., Knippers, J., Mollaert, M., André, J., Steenbergen, R., Kohler, J., Bakeer, T., Maljaars, J., Allaix, D., Spross, J.

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Abstract

EN 1990 “Eurocode: Basis of structural design” defines structural reliability as the ability of a structure to fulfil the specified requirements during the service life for which it has been designed; the notion covers safety, serviceability and durability. According to this definition reliability is the concern of all parts of the whole suite of Eurocodes. In a more narrow sense of the word, reliability refers to the way uncertainties are dealt with in design and assessment by the use of partial factors or more advanced probabilistic methods. This document deals primarily with this more narrow definition, it describes both the probabilistic and semi-probabilistic approach and gives special attention to their underlying relationship. Chapter 2 should be considered as a general introduction to this report and introduces relevant basic notions such as uncertainty, probability, risk, member and system failure, time dependency, effects of deterioration and inspection, etc. In chapter 3 the focus is on the basic reliability requirements in EN 1990 related to the persistent as well as the accidental and seismic design situations. Some special attention is given to the new materials: glass, fibre reinforced polymers and membrane structures included in the second generation of the Eurocodes. In chapter 4, attention is given to the special problems that may arise in assessment of existing structures. Important is the treatment of several types of observations (inspections, monitoring, proof loads etc), the various decision options (further inspection, repair, and demolition) and of course typical reliability issues (updating, reduced targets). The explicit risk and full probability methods described in Chapter 5 are intended to support these approaches where allowed and meaningful. It also gives background for code calibrations. Annex A gives an overview of probabilistic models for actions, material properties and model uncertainties. It may be considered as a reflection of the present state of knowledge in the various Eurocode subcommittees, working groups and horizontal groups. Annex B shows a number of example applications related to various parts of the theory.

Foreword

In building and civil engineering, most structures are designed and assessed on the basis of codes and standards. These documents describe the structural calculation models, the values for the load (action, demand) and resistance (strength, capacity) parameters, the safety margins, the way to cope with accidental and extraordinary design situation, etc. The intention is to arrive at structures that are both safe and economic and serve implicitly or explicitly sustainability goals like limiting emissions and consumption of natural resources. In several countries the codes may be mandatory in a legal sense on the basis of a building act.

The suite of Eurocodes (EN 1990 – EN 1999) has been written with the above intentions to harmonize the various national rules within Europe. Its field of application are all relevant structures like buildings, bridges, towers, etc. including their foundations for all kind of materials, loads, geometry and models. The various parts of the Eurocodes deal with one or more of those topics. Within this spectrum of codes the aim of EN 1990 Basis of Structural Design is to harmonize the reliability concepts over the various structures and design situations. It should be emphasized that as far as safety levels are concerned, every member state within CEN has the right to make individual choices and modifications. Actually, that is the role and duty of every national code committee as explained in document CEN/TC 250 N1250.

The present report is a Background Document to the EN 1990:2023 standard mentioned above. It has been written by a special Ad Hoc Group that was installed on the basis of the following formal decision by Subcommittee 10 of CEN/TC250.

DECISION 13 taken by CEN/TC 250/SC 10 on 2018-12-06

Subject: Formation of a CEN/TC 250/SC 10 Ad-Hoc Group to prepare a JRC Science and Policy Report on 'Reliability Background in the Eurocodes'

The CEN/ TC 250/SC 10 EN 1990 – Basis of structural design, considering the requirement to engage efficiently with progress of SC10 related outputs from Mandate M/515

– agrees to the formation of an Ad-Hoc Group to prepare a JRC Science and Policy Report on 'Reliability Background in the Eurocodes' with a convenor (TBC), maximum 1 representative from each of CEN/TC 250 subordinate group (SCs, WGs, HGs), 3 members from SC 10 as key members and 2 members from JRC, noting the possibility for the SC Chairman to enlarge the group in the future as needed;

– agrees the following terms of reference for the Ad-Hoc Group:

- Draft an explanatory report on the assumptions in EN 1990 for the most representative set of partial factors so as to avoid doubts and possible confusion both at national level and for practical users,
- To provide guidance to NSBs for their decision on partial factors in Annex A.1 of EN 1990 in each country, illustrating common methods, procedures, etc. also by means of examples, taking into account the work of SC 10/WG 1,
- Provide guidance on stochastic modelling on the most representative failure modes for different materials (e.g. concrete, steel, timber, masonry) and for different loads (e.g. imposed loads, wind actions, snow loads),
- Provide basic assumptions, methods and provide examples to serve as a reference for the implementation of full probabilistic methods and, potentially, of risk-based methods, in line with EN 1990 guidance given in its informative Annex C,
- To report regularly on the progress of the work to SC 10, thus enabling SC 10 to report to TC 250,
- To deliver within the time frame for the publication plan for EN 1990.
- SC 10 requests TC 250 to encourage each subordinate group to appoint a member to the Ad-Hoc Group.

The decision was taken by unanimity. The appointed members of the Ad Hoc Group can be found on page 8 of this report. The group started their activities in the summer of the year 2019 and has met about 12 times, mostly in the form of electronic meetings. Subcommittee SC10 was informed on a regular basis and in the summer of 2021 a request for feedback under all relevant subcommittees and workgroups was organised. A summary of the main comments received and the reactions of the Ad Hoc Group can be found in chapter 6.1.

It should be noted that the report is intended to support those engaged in the application of reliability concepts for the calibration of National Annexes to the Eurocodes and/or for specific projects. The report has therefore been drafted for a target audience that is familiar with this technical field and it also assumes an understanding of the fundamental concepts of reliability analysis. Such 'text book' material is not included in this report. References is made to e.g. Melchers and Beck 2007, Madsen et al. 2006, Schneider and Vrouwenvelder 2017. It is also assumed that the reader is familiar with methods and terminology in EN 1990.

It is helpful to acknowledge that CEN/TC 250 unanimously agreed the approach to be taken in seeking to enhance ease of use of the second generation of the Eurocodes. Two aspects of this approach are of particular relevance to this report.

First is the aim defined for the primary target user group of competent practitioners which states: 'We will aim to produce standards that are suitable and clear for all common design cases without demanding disproportionate levels of effort to apply them'. This aim inevitably results in some degree of simplification and conservatism in certain design situations.

Second is the principle that that Eurocodes should include 'state-of-the-art material the use of which is based on commonly accepted results of research and has been validated through sufficient practical experience'. Standards such as the Eurocode are reviewed on a regular basis and it is expected that they will be periodically updated. Whilst this report focusses on the background to EN 1990, it also discusses some matters that have not been incorporated into the Eurocodes because they were not yet felt to sufficiently satisfy this principle. However, it is valuable to include them in anticipation that, with further research and application, they could potentially be included in the future and therefore to encourage that such research is undertaken.

Finally, it is appropriate to recognise that during the development of the Eurocodes there has been significant debate on some of the matters covered in this report before arriving at a consensus position. Inevitably, such a consensus does not necessarily represent the first preference of all parties.

Steve Denton

Chair, CEN/TC 250 "Structural Eurocodes"

Paolo Formichi

Chair, CEN/TC 250/SC 10 "EN 1990 Basis of structural design"

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Authors

This report is the result of the work performed by the CEN/TC 250/SC 10 Ad-Hoc Group on the Reliability Background in the Eurocodes. The group had the following members:

Ton Vrouwenvelder	Convenor CEN/TC 250/SC 10 Ad Hoc Group Reliability	
Silvia Dimova	formerly Joint Research Centre of the European Commission, Ispra-Italy	
Luisa Sousa	formerly Joint Research Centre of the European Commission, Ispra-Italy	
Jana Marková	CEN/TC 250/SC 1	(Actions on structures)
Giuseppe Mancini	CEN/TC 250/SC 2	(Concrete structures)
Ulrike Kuhlmann	CEN/TC 250/SC 3	(Steel structures)
Andreas Taras	CEN/TC 250/SC 4	(Composed structures)
Robert Jockwer	CEN/TC 250/SC 5	(Timber structures)
Wolfram Jäger	CEN/TC 250/SC 6	(Masonry structures)
Timo Schweckendiek	CEN/TC 250/SC 7	(Geotechnical structures)
Paolo Franchin	CEN/TC 250/SC 8	(Earthquake resistance)
Davor Skejić	CEN/TC 250/SC 9	(Aluminium structures)
John Dalsgaard Sørensen	CEN/TC 250/SC 10	(Basis of structural and geotechnical design)
Pierre Spehl	CEN/TC 250/SC 10	(Basis of structural and geotechnical design)
Mungo Stacy	CEN/TC 250/SC 10	(Basis of structural and geotechnical design)
Markus Feldmann	CEN/TC 250/SC 11	(Structural Glass)
Pietro Croce	CEN/TC 250 HGB	(Horizontal Group Bridges)
Bin Zhao	CEN/TC 250 HGF	(Horizontal Group Fire)
Peter Tanner	CEN/TC 250 WG2	(Existing Structures)
Jan Knippers	CEN/TC 250 WG4	(Fibre reinforced polymer structures)
Marijke Mollaert	CEN/TC 250 WG5	(Membrane structures)
João André	CEN/TC 250 WG6	(Robustness)

The AHG would like to thank the following experts for their contributions as co-authors to some specific parts of this document:

Raphael Steenbergen	TNO, Delft, the Netherlands	Section 3.4 and annexes B1 and B2 on partial factors
Jochen Kohler	NTNU, Trondheim, Norway	Section 3.4 and annexes B1, B2 and B3 on partial factors
Tammam Bakeer	TU Dresden, Germany	Section 3.18 and annex B4 on numerical analysis
Johan Maljaars	TNO, Delft, the Netherlands	Annex B6 on fatigue
Diego Allaix	TNO, Delft, the Netherlands	Annex B7 on deterioration of concrete
Johan Spross	KTH, Stockholm, Sweden	Annex B8 on risk analysis and design by observation

1 Introduction

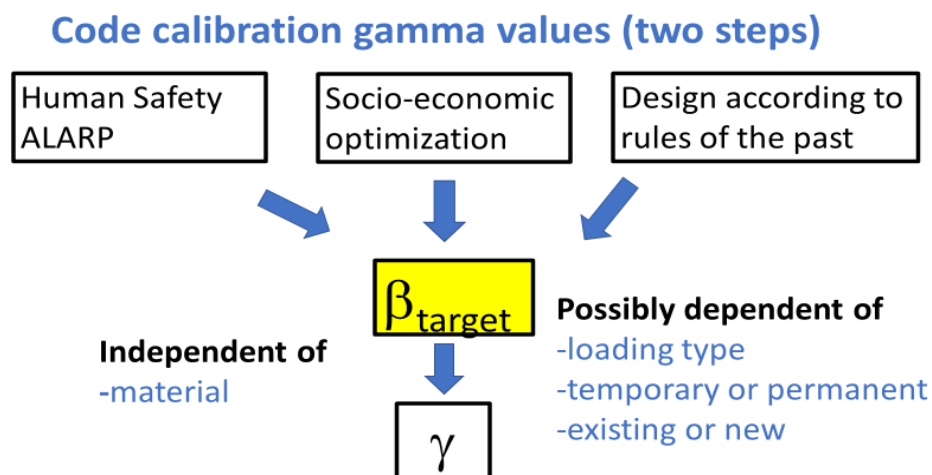
Society expects that buildings and other structures are safe for the people who use them or are near them. They should also protect against adverse economic consequences. The failure of a building or structure is expected to be an extremely rare event.

Structural engineers devote their effort to meeting society's expectations efficiently. Engineers and scientists work together to develop solutions to structural problems. Given that nothing is absolutely and eternally safe, the goal is to attain an acceptably small probability of failure for a structure. Reliability analysis is part of the science and practice of engineering today, not only with respect to the safety of structures, but also for questions of serviceability and other requirements of technical systems that might be impacted by uncertainties.

The elaborations of these tasks may take place on several levels often referred to (ISO 2394) as (1) the risk-based approach, (2) the reliability-based approach and (3) the semi-probabilistic or partial factor-based approach. Appropriate calibration procedures should lead to the various levels giving consistent results (see section 2.3). In addition to this internal calibration of the various design approaches there is also a need to compare the results externally to design and assessment methods used in the past (see Figure 1.1). Where these methods were successful, they should not be changed substantially. On the other hand, unsafe or uneconomic situations should be corrected when encountered, as has been done systematically at every code revision in the past decades.

On the highest level of abstraction, the design as well as the assessment of a structure (or structural system or part of it) comes down to a decision analysis. The question is which lay out, which material and what dimensions are needed to give a structure that satisfies all preferences, taking into account possible constraints of several types. Only in special cases the total optimization is part of the individual design process; in most cases this process is solved in several steps. Authorities often prescribe the minimal required reliability by balancing implicitly or explicitly on the one hand the use of resources needed and on the other the consequences of unsatisfactory behaviour. This includes, of course, also aspects of human safety. The resulting required reliability may be presented in terms of target failure probabilities, but in daily practice use is made of a translation of probabilities into demands on the level of characteristic values and partial (safety) factors, the so-called partial factor method or semi-probabilistic approach. In this document the full risk and reliability-based design and assessment procedures are treated in Chapter 4, the semi-probabilistic method is treated in Chapter 3. Chapter 2 treats the information that is general for all levels.

Figure 1.1. Basic scheme of calibration procedure: the first step is to determine a target reliability for an appropriate class of structures based on a combination of several criteria, the second step is to find corresponding partial factors.



2 Principles of structural reliability

2.1 Steps in structural design and assessment

The design and assessment of structural systems comprise many aspects. In this document we focus on structural reliability related issues. In this context the word *structural* includes geotechnical and other relevant disciplines. This chapter 2 deals, by way of introduction, with a number of the general aspects of structural design and assessment and does not give specific background information to clauses and formulas the Eurocode. This will be done in chapter 3.

The basic question is, whether for a given structure (planned or existing) the level of reliability for an intended (remaining) service life is optimal or at least sufficient. This is called a reliability verification procedure. This verification procedure may formally be subdivided into a number of steps either explicitly in the case of risk based approaches or implicitly using the partial factor approach:

1. In the first stage the system to be considered is defined and an inventory of all relevant external *threats and hazards* is made; the threats can be from any source of mechanical, physical, chemical or biological origin. Note that even so called “unforeseen” actions may need attention.
2. Next the internal structural processes that may lead to failure need to be described, usually referred to as *mechanisms* or *failure scenarios*, including the planned or achieved effects of quality assurance, inspection, monitoring and maintenance. The combination of hazard and failure scenario is often referred to as a design situation.
3. The third step describes the possible suite of *consequences* of the resulting structural behaviour.

In all the above steps “models” are used. The models reflect our scientific and empirical knowledge, usually in the form of mathematical expressions and algorithms like for instance a structural Finite Element Model. Models can be defined as relations between variables describing the state of input and output of physical processes. Because of the *uncertainties* involved, the outcome of the structural behaviour as well as the net benefit of all design and assessment decisions are uncertain. The best we can do is to find the decision that optimizes the expectation of the benefit, using the concepts of probability theory. Actually, that brings us to the next steps in our procedure:

4. Quantification of relevant uncertainties in probabilistic terms.
5. Finding solutions leading to an optimal expectation of the benefit (or a minimum of costs), meeting at the same time all societal preferences and demands related to human safety, environmental and other intangibles.

Given the fundamental role of the notions uncertainty and probability it makes sense to discuss these issues first before going into further elaboration in structural engineering.

2.2 Uncertainty, probability and frequency

2.2.1 Types of uncertainty

Like in everyday life, uncertainties of multiple different nature are encountered in decisions concerning the design and assessment of structures. The most obvious uncertainties are related to the *natural variability or randomness* of actions, material properties and geometric data. However, as soon as we want to establish the statistical properties of those variables, we have to deal with many unclear sets of subpopulations and often a substantial shortage of data. This leads to the important class of so-called *statistical uncertainties*. In addition, we have to admit that our theories and models about environmental and structural behaviour are approximations of reality and make predictions with some higher or lower degree of accuracy. The related uncertainties are referred to as *model uncertainties*. Additionally we have to deal with uncertainties present in *measurements* and *observations*. Finally, in practice uncertainties can arise from human *errors* of several kind as well as *unforeseen* (or even unforeseeable) events.

Notes:

Different kinds of uncertainty may require different means of counteracting; for instance, partial factors may not be very effective against gross errors.

Uncertainty is not a (constant) property of a structure, but depends on the state of knowledge, which can change over time, e.g. during the process of design, building and use; it is one of the reasons that existing structures can have to be judged in a slightly different way (see chapter 4).

In reliability literature uncertainties are often subdivided into *aleatory* uncertainties resulting primarily from (natural) variability and *epistemic* uncertainties resulting from incomplete knowledge. The idea is that epistemic uncertainties can be reduced by investigations while aleatory cannot, although the exact subdivision to some extent is arbitrary. It is of course useful to know whether further investigations or research activities can reduce uncertainty or not. Sometimes reduction of uncertainty is possible in theory but not in practice. The basic question, however, is whether, in structural engineering, we want to make a distinction in treatment between the two types of uncertainty if they exist. It turns out that there are two schools of thought.

In the *classical (frequentist)* school of statisticians a clear distinction in the treatment is made. The basic philosophy is that the quantification of epistemic uncertainties in principle requires subjective estimates and should be kept carefully separated from the objective scientific part. In the alternative so called *Bayesian* school, the notion of probability is understood as a “degree of belief” which makes an equal treatment of objective and subjective uncertainties a logical next step. In applications in structural engineering the Bayesian approach has been adapted from the beginning as the most appropriate one. In short, the idea is that as subjective estimates of likelihood simply cannot be completely excluded anyhow, the best we can do is to deal with them and make sure that these estimates reflect the view of a large group of professionals, thereby making them as objective as possible. This will not make every individual estimate fully reliable. The only claim is that, if carried out properly, the probabilities will be correctly averaged over a large number of decision situations. The requirement to fulfil that claim, is that the purely intuitive part is neither systematically too optimistic nor systematically too pessimistic. Checking whether the theoretical results match reality can only be done by long term observations over a wide variety of structures.

A pronounced difference in application is visible in the statistical evaluation procedures for design by testing. For example, in the frequentist formulas for the 5% characteristic values we first find the best guess for the 5% fractile (natural uncertainty) and an additional say 10% confidence interval (statistical uncertainty). In the Bayesian approach (see EN 1990:2023 Annex D) the 5% fractile is found including the statistical uncertainty and no confidence interval is given.

2.2.2 Model uncertainty

Uncertainties are not only present in variables like loads and material properties but also in the action and resistance models used in structural analysis. The models usually are based on a combination of first principles and (semi) empirical relations. Depending on the material and the type of mechanism (e.g. bending, shear, buckling, corrosion) models will predict the real behaviour with lower or greater accuracy. The main sources of model errors are related to influences or parameters neglected in the models and approximations, simplifications or errors in the sets of equations. To incorporate these uncertainties, the outcome of the model Y_{model} can be represented as:

$$Y_{\text{model}} = f(X_1, \dots, X_n, a_1, \dots, a_m, \theta_1, \dots, \theta_k) \quad (2.1)$$

Often reduced to:

$$Y_{\text{model}} = \theta_1 f(X_1, \dots, X_n; \mathbf{a}) \quad (2.2)$$

The variables $X_1 \dots X_n$ describe the relevant physical (loading, material or geometrical) parameters; the additional random variable θ accounts for the model uncertainty, $\mathbf{a}$ is a vector containing the parameters that can be assumed as deterministic.

As far as possible models are compared with and calibrated on measurements and experimental tests using a best fitting criterion and estimates of the degree of uncertainty, usually respectively the bias $\mu(\theta)$ and the coefficient of variation $V(\theta)$. Measurement errors and statistical uncertainty due to a limited data set should be taken into account. Note that well calibrated advanced models sometimes are used to calibrate simplified

engineering models. In those cases it is important to realize that model and statistical uncertainties are present on both levels and should be taken into account simultaneously.

In the absence of experimental evidence expert judgement (Bedfort and Cooke, 2003) is sometimes considered as an option to estimate the model uncertainty parameters, although utmost care is needed.

2.2.3 Observed failure frequency in practice, notion of hidden safety

In the light of the above discussion the question is often raised whether or not the theoretical probability of failure corresponds or should correspond to the frequency of failed structures in practice. In this discussion the notion of hidden safety plays an important role, among other factors such as gross/human errors or negligence during design, execution or operation. Gross/human errors are supposed to be covered by quality control and operational requirements (e.g. execution standards); they cannot be adequately covered by partial factors calibrated based on engineering models and physical and model uncertainties related to uncertain parameters. Excluding these type of errors, the observed failure rates (e.g. ULS exceedance) appear in general to be small compared to the estimations with engineering models.

This section addresses different types of hidden safety, illustrated by examples, and the implications for the safety philosophy in the Eurocodes. It is noteworthy that there are also situations where unconservative choices or assumptions are made, however this section focusses on the more common safe-sided elements.

Generally speaking, assumptions and choices in design and the related engineering modelling tend to be conservative, or in other words, on the safe side. In most cases such conservative choices are very rational, taking into account the asymmetry in consequences from errors or deviations on the safe and unsafe side. The list below contains several (maybe interconnected) examples of hidden safety that can arise in construction design:

1. Resistance modelling: We may opt for elastic analysis instead of plastic analysis. Model choices are often based on the wish to make simplifications (ease of use). The simplified models are often relatively safe-sided and robust approximations of more advanced models.
2. Load modelling: The load patterns assumed for modelling the spatial distribution of imposed and climatic loads are often chosen with very safe assumptions, or the seismic action is described through a uniform hazard spectrum, rather than a less conservative conditional spectrum (See.3.5.7).
3. Design type or structural system: In many design situations, a structure is designed according to its critical section or member, while the rest is overdesigned. Sometimes this is done in order to enforce a desired favourable plastic mechanism (Žižmond et al, 2016). Also, it is quite common that ULS does not govern the structural dimensions, but rather SLS does. In these cases, the safety margin can be considerable. Finally there may be robustness consideration, for example, anchors of retaining walls are designed such that neighbouring anchors could cope with the failure of an in-between anchor.
4. Structural behaviour modelling: The structural behaviour is simplified using conservative assumptions. For example, a hinge is assumed where in fact an encasement providing at least partial continuity exists. Such system redundancies are often not explicitly exploited.
5. Material strength modelling: For example, the usual steel design is for yielding, while strain hardening and the margin to the ultimate strength is typically neglected (unless covered by bias in model factors in calibration of partial factors); another example are the conservative assumptions with respect to geometrical imperfections.
6. Constructional type: For many designs there are minimum values for dimensions and additional often prescriptive safety rules. For example, in steel construction, there is a minimum weld thickness given even if not needed. Besides there are explicit robustness rules and rules like “one bolt is no bolt”, also providing extra safety. Minimum values for member sizes and detailing in Eurocodes compliant structures have been shown to lead to reliability, with respect to seismic action, higher than the target one by orders of magnitude in low-seismicity areas (Iervolino, et al. 2018).
7. Practical issues rounding off structural dimensions to the safe side and producers aiming at better quality than required to avoid a negative acceptance test.
8. The limited number of safety elements in the safety format intended for a large set of applications. This may lead to a subset of applications where already the theoretical reliability level is not in accordance with the target.

The list above is not exhaustive but meant to provide examples of types of hidden safety, many of which are related to modelling choices. It should be clear that the estimation of actual, observable failure rates in reality using engineering models is difficult, given the unavoidable bias introduced by hidden safety elements, even though some types can be covered by taking model bias explicitly into account.

When considering hidden safety, it is meaningful to make a distinction between the following two categories:

- Hidden safety which is actually 'hidden' for the normal user of a code but can be modelled as model uncertainties based on comparisons between physical tests and the engineering models in the codes. Hence, this type can be accounted for in calibration of the partial factors, though it can be difficult to determine model uncertainties accurately. If physical test results are available, model uncertainties can be quantified using the method in EN 1990:2023 Annex D.
- 9. An important implicit conclusion of this type of hidden safety poses limitations to using advanced analysis methods other than those the code was calibrated to. For example, a user may think that an increase of the characteristic resistance can be obtained using an advanced numerical method (e.g. FEM) instead of the engineering model in the code but the conservative bias of the engineering model in the code may already be accounted for in the choice of the material factor. A reduction of the scatter, however, still may be possible.
- Hidden safety related to aspects like conservative assumptions in design. These aspects can be substantial but there is no guarantee that they always are present. Some structures are optimized just to the limit of the requirements in the code.

Related to the first category the transparency is of great importance: if hidden safety is made explicit by comparing model predictions with experiments or real behaviour, the implication for design mostly depends on the starting point for the reliability target, which has been and probably will be treated differently in the different Eurocode member states. The basic options, as stated earlier, are:

- Design rules for new structures are calibrated to similar existing structures as long as they exhibit satisfactory performance (essentially ensuring that roughly the same structural dimensions are obtained by new design rules).
- Designs of new structures meet a reliability target based on considerations of human safety and economy of design, verified by reliability analysis.

In the first case there is almost no chance to remove a (useful) hidden safety, in the second case there is with possible unintended consequences. For completely new types of structures and materials the second option is the only one available.

In addition to hidden safety there might also be *hidden unsafety*. Sometimes models may be based on incorrect assumptions that are not detected in underlying experiments and tests. As an example, one might think of a change of failure mode from ductile to brittle when reaching the tail of the distribution. One observes always ductile modes in even a large number of tests, but at the relevant low fractiles the mode may be brittle having large consequences for the system behaviour and consequences to human safety because of the lack of warning.

However, even if we could fully exclude the issues of hidden (un)safety, the nature of probability itself would still give rise some careful considerations. The failure probability calculated is always the result of the chosen physical models for the relevant mechanisms and the appropriate corresponding statistical properties for the random variables. These choices are based on a *particular state of information*. As a consequence, the failure probability for a certain building may change if we get/obtain more information e.g., on the concrete quality actually delivered instead of the concrete quality in general. The state of information is to some extent a choice in itself that can be optimized.

This ambiguity is also one of the reasons that we should not interpret a failure probability as an absolute attribute of the physical structure or structural system. This would only be the case if no further information could be obtained at all and in structural engineering that can hardly ever be the case. In the context of risk informed decision making, the failure probability is mainly a comparative measure and should always be seen and communicated conditional to that.

However, the question of the relation between the outcome of a reliability analysis and the failure frequency in the real world will remain. Notions like optimization and updating are meaningful only if the statistical numbers

have a meaning in the world where optimization and updating is performed. So the need to have numbers as realistic and up to date as possible for that aspect is there.

2.3 Approaches to structural decision making

2.3.1 Safety and Economy

In section 2.1 the general decision-making approach with respect to the verification of structural reliability has been presented and in chapter 1 the possible levels of risk based, reliability based and semi-probabilistic methods were introduced. In this section these three levels and in particular their step wise coherence will be discussed. For all levels the result should be a sufficiently safe as well as economic structure. The notions safe and economic are not fully clear in themselves and need some further elaboration.

“Safe” has primarily to do with the safety of people who use the structure or are in the vicinity of the structure. Attention should be given to the risk of individuals, to the so-called societal risk (the possibility that a number of persons gets killed or hurt) and the fact that people should feel safe in a built environment. To some extent *“safe”* is also understood as a requirement of the structure to protect economical value. The central attention, however, is usually given to the notion of *“individual risk”*. The individual risk follows from the combination of the probability of structural failure and the probability that a person is killed in the event of structural failure. The socially accepted value for the individual risk level related to structural failure level is usually considered to be at least one order lower than for other accidents. Maximum acceptable individual risk values are generally in the order of 10^{-6} to 10^{-5} (often industrial) per year (see also Vrijling et al, 2005). In many countries values for the maximum individual risk are also often depending on regulations for specific types of hazards such as industrial accidents, flood defence, traffic safety, etc. Usually, they are expressed in terms of annual probabilities.

“Economy” should be interpreted as *“socio-economic”*, including all types of direct and indirect costs related to building, use, damage and demolition of a structure. It includes monetary values as well as intangibles like environment issues and sustainability. In other words: one should not build structures that require large amounts of resources that could be used in better ways. On the other hand, where with little effort a higher reliability level can be achieved this should be done to avoid unnecessary socio-economic costs of failure. So, we are looking for a balance between safety, sustainability and economy. This is a responsibility for every code committee when developing new codes or when rewriting existing ones. The past in this context is a guidance, not a must. The aim must be to combine new developments with proven technology.

2.3.2 Risk based approach

In the full risk-based approach the above decision process is carried out explicitly, including the safety constraints mentioned. In its most simplified way, the optimization can be written down as a minimization of the sum of the direct costs of the design decision and the risk due to failure. That is, leaving out for the time being discounting and other complicating factors like maintenance, we want to minimize the total cost expectation C_{tot} :

$$\min C_{tot} = C_{con} + P_f C_f \quad (2.3)$$

Under the safety constraint:

$$P_f < P_{ft} \quad (2.4)$$

where C_{tot} are the expected total costs, C_{con} the cost of the (design) decision, C_f the costs of the consequences of unsatisfactory performance and P_f the probability of unsatisfactory behaviour during the design service life. The extra condition $P_f < P_{ft}$ is based on ethical and societal aspects of human safety where relevant. In many cases the so called ALARP principle is used (As Low As Reasonably Practicable). The risk-based approach is generally used for individual structures, not for a portfolio.

The approach implicitly assumes that the structural behaviour can be subdivided into a satisfactory performance and an unsatisfactory performance or failure. The border line between the two possibilities is

called the limit state. To enable a further numerical approach a limit state function $g(\mathbf{X})$ is defined, where $\mathbf{X}$ is the vector of all random variables (actions, material properties, etc). By definition the domain $g(\mathbf{X}) < 0$ corresponds to failure and $g(\mathbf{X}) > 0$ to non-failure. In order to formulate the function $g(\mathbf{X})$ use is made of well-known action and structural models. This means that the function $g(\cdot)$ might be much more complex than it looks at first sight. The failure probability can now formally be calculated from:

$$P_f = P(g(\mathbf{X}) < 0) = \int_{g(\mathbf{x}) < 0} f(\mathbf{x}) d\mathbf{x} \quad (2.5)$$

where $f(\mathbf{x})$ is the joint probability density function of all the random variables, which means that for all variables involved we need estimates of their mean values, their uncertainty and also their mutual dependencies. Note that $\mathbf{X}$ in equation refers to the random variables themselves and $\mathbf{x}$ to particular realisations of $\mathbf{X}$.

Notes:

The term “failure” in reliability theory is not restricted to collapse but can refer to any form of unintended behaviour. It can refer to the system level as well as to the level of members or (sub) systems. In some cases, the binary subdivision “failure versus nonfailure” is of course a simplification as there can also be a gradual transition from satisfactory to non-satisfactory behaviour like in service-ability limit states. In other cases, the real limit state can often be difficult to model and use is made of approximate or condition limit states.

Random entities (like for instance the wind velocity) can vary continuously in in time and space; in those cases they are usually referred to as random processes and random fields respectively

2.3.3 Reliability-based approach

In the reliability-based approach the optimization (2.3) to (2.5) reduces to:

$$\min C = C_{\text{con}} \quad (2.6)$$

Under the condition

$$P_f < P_{ft} \quad (2.7)$$

The minimal required reliability P_{ft} now includes (next to immaterial constraints like human safety) also the economic considerations that were left to an explicit optimization procedure in the risk-based approach. This in fact is the first step in Figure 1.1. Note that this way minimum human safety requirements and economic average targets are mixed up. It is the price of simplification.

The required minimum or target reliability levels (in fact the maximum acceptable failure probability) usually are proposed by the engineering community (in particular code committees) but are in the end the responsibility of the relevant authority and given in the National Annex. Required reliability levels are generally presented as a classification system of the consequences of failure expressed in terms of human safety, economy and environment. Recourses needed to achieve a higher safety level may play a role, explicitly or implicitly. In many code calibrations also a comparison to the design procedures applied in the in the past is made. The idea is that the design rules in the past on the whole turned out to be quite satisfactory and should only be modified to achieve more consistency or to enable a wider field of application. There is, in other words, no need for major shifts. For completely new types of structures or materials the challenge is somewhat different. Also new requirements, for instance relating to sustainability matters, can lead to the need for reconsideration of target reliabilities.. The expectation is that this will be even more in the future.

Note that (2.7) may refer to the whole system or to individual members of failure modes (say ductile versus brittle). If used for a portfolio the safety constraint could in (2.7) refer to the highest failure probability or to an average value. See also sections 2.3.5 and 2.5.

2.3.4 Semi-probabilistic approach

For the semi-probabilistic approach, used in daily practice, the requirements (2.6) and (2.7) are transferred to:

$$\min C = C_{\text{con}} \quad (2.8)$$

Under the condition:

$$g(\mathbf{X}_d) > 0 \quad (2.9)$$

The $\mathbf{X}_d$ are the so-called design values of the random variables. In terms of reliability theory, the design values represent the values of $\mathbf{X}$ for which the likelihood density of failure (after a transformation to a set of independent standard Gaussian variables) is maximal. If X has a normal distribution the value X_d can (see also the Design Value Format method, chapter 2.6) be found from:

$$X_d = \mu_X - \alpha_X \beta_t \sigma_X \quad (2.10)$$

where μ_X and σ_X are the mean and standard deviation of X respectively, α_X is the (FORM) importance factor of the variable X with $0,0 \leq \alpha_X \leq 1,0$ for resistance properties and $-1,0 \leq \alpha_X \leq 0,0$ for actions, β_t is the target reliability index corresponding to the target failure probability P_{ft} via $P_{ft} = \Phi(-\beta_t)$, where $\Phi(\cdot)$ is the distribution function of the zero-mean normal distribution with unit standard deviation, see further Table 2.1. For other than normal distributions the value of X_d may be found from $F(X_d) = \Phi(\alpha_X \beta_t)$ with $F(\cdot)$ being the cumulative distribution function of the variable X . As the result must be conservative for a large group of structures, the selected α_X values will usually be conservative (uneconomic) in individual cases.

Table 2.1. Relationship between the failure probability P_f and the reliability index β .

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1,3	2,3	3,1	3,7	4,2	4,7	5,2

Source: Excel

In the Eurocodes the design values are derived by multiplying (or dividing by) the characteristic values $\mathbf{X}_k$ with some appropriate partial or combination factor.

$$X_d = X_k / \gamma \quad \text{for resistance} \quad (2.11a)$$

$$X_d = \gamma X_k \text{ of } \psi X_k \quad \text{for actions} \quad (2.11b)$$

These partial factors have been calibrated (second step in Figure 1.1) in such a way that the design or assessment decision for a large class of structures fulfils with some acceptable level of accuracy the reliability requirements (see Chapter 3 for more details). The basic problem is that the alpha-values α_X vary from design situation to design situation. For that reason, the semi-probabilistic method is implicitly or explicitly based on a conservative (weighted) choice of the alpha-values. This is not only the case for the Design Value Format (DVF) method (section 2.6) but implicitly also holds for the procedure where partial factors are derived using an optimization procedure.

Note: both in EN 1990:2002 and EN 1990:2023 the DVM method is recommended and often used on the resistance side; the proposed partial factors for variable loads, however, are often lower than would follow from the DVM method. See section 3.4 and Annex B.2 for further information.

2.3.5 Member versus system behaviour, robustness

The analysis of reliability and risk should in principle be applied to the total structural system, including its interactions with the environment. In practice however the verification of the reliability is often performed on the level of single members (beams, columns, connections), single modes (shear, bending) or even on the level of a single cross section or individual point. The verification rules in the Eurocodes are in most cases formulated at the member level and also the target reliability is used for the individual failure mode at member level. This might be considered as the outcome of a trade-off between correctness and complexity of the analysis. We shall discuss it below in somewhat more detail.

Even if collapse modes on the structural level are considered (like in non-linear (e.g. buckling) analysis), the verification is actually carried out on an individual *failure mode* basis; the fact that the presence of several (partially dependent) potential modes may lead to an increase of the failure probability is usually neglected. The JCSS probabilistic model code, however, recommends a more system-based approach and a relatively simple application in practice may be found in (Manen et al., 1998).

In general, however, the extension of member to actual system reliability is a very challenging process. System effects may be load path and load history dependent (including the initial conditions). System effects involve concepts such as failure modes, damage progression, robustness, fragility and system reliability. As a result, the level of uncertainties that influence the determination of the reliability of a system are larger than the one applicable to members.

The general belief is that due to a combination of redundancy and ductility most structures have a (substantial) higher reliability than the individual members. However, ductility requirements are not always linked explicitly to target reliabilities as mentioned above and for the possible effects of low ductility (brittle behaviour) there is little attention from a reliability point of view. Also, in cases of low redundancy (e.g. a statically determinate structure) comprising multiple element the failure probability of the system is typically higher than the maximum member failure probability.

Typically, in design codes, system effects are dealt with predominantly by indirect design methods (i.e. prescriptive design and detailing rules), envisaged as a second layer of safety, additional to the direct design methods used to check the reliability of members. For example, interstorey drift limits are used to define performance-based levels in seismic design while tying rules and ductility requirements are specified to provide baseline levels of robustness performance.

The details of the indirect methods depend on the severity level of the (direct and indirect) consequences due to failure of a structural member, structural part or the entire system. Usually, the severity is not considered as a gradual scale but as a rough classification of Consequence Classes, actually the same as the classification used for differentiation of partial factors. It should be noted here that controlling the member reliability, does not, in general, cause a change of the potential (direct and indirect) consequences that can occur in the event of failure, but rather concern the control of the expected probability of occurrence of such events (and this way the expected risk level).

The supplementary indirect design provisions for system behaviour (also referred to as design for robustness) are intended for the uncertain and unknown adverse triggering events where the failure consequences are not negligible. The rules for robustness are usually presented as a set of rules that cover indirect and direct design methods:

Indirect design methods are usually provided in terms of minimum tying forces, which attempt to indirectly simulate the structural effects caused by a loss of a column or foundation support,

Direct design methods may involve alternative load paths, identification and design of key members, structural segmentation.

While indirect design methods apply primarily to hazard independent scenarios (i.e. only the structural effects of a set of unidentified hazards are simulated, most commonly by notionally removing of a key member), direct design methods apply to both hazard independent and dependent scenarios. Indirect design methods and alternative load paths often use catenary action effects to mobilise additional resistance. Reference is made to the work done by (WG6.T2).

Another type of relevant system behaviour is the micro system behaviour inside a member. This may result in volume effects as for instance are relevant for timber (Rouger, F. 2017) and ground (see chapter 3.6.1). For brittle materials the resistance of a member may also change with the volume (size effect in concrete) due to energy balance effects

A very special cases in this respect is the material glass. Given its very brittle behaviour it is very sensitive to all kinds of influences that cause extra stresses normally not accounted for. It is not always possible to design for adequate reliability of all individual elements which makes it essential that fail safe system behaviour is considered, given the failure of a single element. See also chapter 3.

2.4 Time effects

2.4.1 Bath tub curve

Time effects are almost always present. Even in a standard design we have to deal with a reliability requirement $\beta = 3,8$ for 50 years and a time varying load for which the characteristic value is defined on a 2% exceedance probability for the period of 1 year. The given time periods are essential. The point is that the fluctuations in time of the load leads to a dependency of the failure probability on the period of time under consideration. In addition, a structure may lose resistance due to time dependant degradation processes like creep, corrosion, surface or internal damage, fatigue, etc. Both effects should be taken into consideration regardless of whether we adopt a probabilistic or semi-probabilistic approach.

If we include the time, the random variables $\mathbf{X}$ as well as the g function itself may become time dependent. If we subdivide the total period t_L in a number of subintervals (for instance periods of one year) we may write:

$$P_f(t_L) = P_1 + P_2 + P_3 + \dots + P_n \quad (2.12)$$

where P_i is the probability of failure *exactly* in year number i , which may be written in terms of limit state functions as:

$$\begin{aligned} P_1 &= P(g_1 < 0) \\ P_2 &= P(g_2 < 0 \text{ and } g_1 > 0) \\ P_3 &= P(g_3 < 0 \text{ and } g_1 > 0 \text{ and } g_2 > 0) \\ &\text{Etc.} \end{aligned} \quad (2.13)$$

where g_i is the (minimum) value of the limit state function in year i . The logic is that failure in the second year is feasible only if the structure has survived first year.

Using the definition of conditional probability, we may also write (2.13) as:

$$\begin{aligned} P_1 &= P(g_1 < 0) \\ P_2 &= P(g_2 < 0 \mid g_1 > 0) P(g_1 > 0) \\ P_3 &= P(g_3 < 0 \mid g_1 > 0 \text{ and } g_2 > 0) P(g_1 > 0 \text{ and } g_2 > 0) \\ &\text{etc.} \end{aligned} \quad (2.14)$$

The vertical bar is the symbol for “conditional on” or “given”. Note that:

$P_i = P(g_i < 0 \text{ and } g_1 > 0 \text{ and } g_2 > 0 \dots)$ are referred to as the *unconditional* annual failure probabilities.

$P_i = P(g_i < 0 \mid g_1 > 0 \text{ and } g_2 > 0)$ are referred to as the *conditional* annual first failure probabilities.

The conditional failure probabilities are larger than the unconditional ones. As long as the failure probabilities are small like in most structural reliability applications, the difference between the two is negligible, as in that case all survival probabilities $P(g_1 > 0 \text{ and } g_2 > 0 \dots)$ are almost equal to 1,0.

For the sake of completeness:

$P_i = P(g_i < 0)$ is called the failure probability for year i (irrespective of other points in time).

Example 2.1. Time varying load, no deterioration

As an example, take a member with time independent resistance R loaded by self-weight G and a yearly independently varying stationary climatic load Q . In that case:

$$g_1 = R - G - Q_1$$

$$g_2 = R - G - Q_2$$

$$g_3 = R - G - Q_3$$

Let now R and G (for the case of simplicity) be deterministic; in that case:

$$P_1 = P(R - G - Q_1 < 0) = P$$

$$P_2 = P(R - G - Q_2 < 0 \mid R - G - Q_1 \geq 0) = P(R - G - Q_1 > 0) = P(R - G - Q_2 < 0) = P(R - G - Q_1 \geq 0) = P(1 - P)$$

Similarly:

$$P_3 = P(1 - P)^2$$

In that case the annual probabilities P_i will decrease although very slowly in the course of time; if R and G are random the decrease will go faster, depending of course on the ratio of the standard deviations of the time dependent part (Q) and the time independent parts (R and G).

Example 2.2. Deterioration

As another example, consider the same case as above, but now the resistance R is subject to mechanisms like corrosion or fatigue:

$$R_i = R_0 - \sum \Delta R_j \quad (j=1..i-1) \text{ where } \Delta R_j \text{ is assumed to be larger than } 0.$$

For an unprotected steel element all ΔR_j may be a set of (correlated) random variables with constant statistical properties each year

For a corrosion protected steel element (e.g. bars in a reinforced concrete structure) ΔR_j may be zero for some random time and non-zero later on.

In the case of fatigue, the resistance at year i is a function of all the load cycles that have occurred in all previous intervals from 0 to $i-1$.

Figure 2.1 shows a possible resulting course of the conditional annual failure probabilities. The curve first goes downwards because of the correlation between the subsequent years. Later it goes upwards because of degradation processes, if present. The total curve is called the bathtub curve. The sum of all conditional failure probabilities may be larger than 1,0.

The unconditional annual probabilities are slightly smaller in the beginning but the difference may increase later. The sum of the unconditional annual failure probabilities to a certain point in time is equal to the total failure probability of the period of time under consideration (see equation (2.12)). The sum of all unconditional probabilities (from one to infinity) is equal to 1,0.

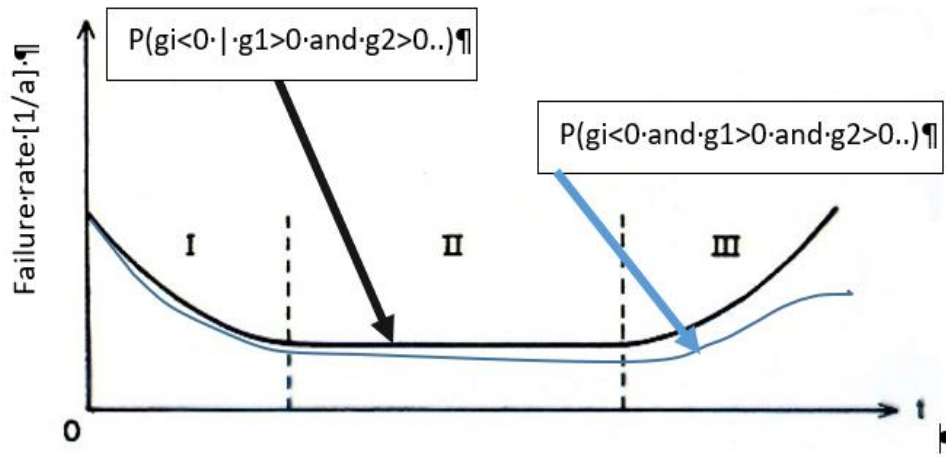
Note that the failure probability for the total period from year 1 to year n is given by (2.12) and can be found by adding the unconditional failure probabilities. It is an increasing (strictly speaking a non-decreasing) function of time.

In practical applications the use of (2.13) and (2.14) is quite cumbersome. The calculation can be simplified by using the so-called *outcrossing approach*:

$$P_i = P(g_i < 0 \text{ and } (g_{i-1} > 0 \text{ and } g_{i-2} > 0 \dots \text{ and } g_1 > 0)) \approx P(g_i < 0 \text{ and } g_{i-1} > 0) \quad (2.15)$$

So instead of taking all past survivals from 1 to $i-1$ on board, we limit ourselves to only the previous year $i-1$ (or event). Whether this is accurate or not depends on the correlation properties. In literature refinements of this method and elaborations for continuous cases can be found.

Figure 2.1. Bathtub curve (black line) showing the conditional failure probability as a function of time, with zones of (I) increasing reliability (dominant time independent uncertainties like resistance and self-weight), (II) constant reliability (dominating time dependent uncertainties, usually loads) and (III) decreasing reliability (dominating deterioration). The blue line shows the unconditional failure probabilities. Note that the annual reliability index has just the opposite profile: it first goes up, is constant and then goes down.



In present practice (also in the Eurocodes) the combination of cumulative deterioration and failure of the reduced resistance under some kind of peak load is often neglected. Both aspects are often considered as separate phenomena with independent safety criteria without considering that the reliability requirement (say a total life time beta of 3,8 or an annual beta of 4,4 or 4,7 for every year) holds for the combination of effects. In chapter 3 this item will be discussed further.

2.4.2 Inspection, monitoring and maintenance

As counterweight to the loss of reliability due to deterioration there is usually a program of monitoring, inspection and maintenance. Given for instance the result of an inspection at a certain point of time t we might update the probability of failure:

$$P_f = P(g(X) < 0 \mid h(X) = 0) \quad (2.16)$$

The event $h=0$ represents the outcome of the inspection; depending on the type of inspection this can also be in the form of $h>0$ or $h<0$. If inspection took place at time t , the updated failure probability is in principle derived for some period following this point in time.

Example 2.3 Inspection for fatigue

As an example, we can rewrite the above general equation in the case of fatigue, assuming no crack was found in year k :

$$P_{fi} = P(a_i > a_{crit,i} \mid a_k < a_{det})$$

where P_{fi} is the failure probability in one of the following years i , a_i is the length of the fatigue crack in year i , $a_{crit,i}$ is the critical crack length in year i , a_k is the length of the fatigue crack in the year of inspection k and a_{det} is the detectable crack given the method of inspection. Note that the critical crack length depends among others on the height of the extreme load in year i . Further examples may be found in Annex B.

Important notions to be included in the analysis are the statistical properties of the inspection method like the probability of detecting a defect and the accuracy with respect to location, nature and size.

Inspection planning should in fact already be taken into account in the design stage. The first inspection should be scheduled at latest at the point in time where the (usually annual) failure probability reaches its maximum acceptable level. Given the result of the inspection a new point in time for inspection can be derived or direct maintenance or repair can be necessary or optimal. The other way around is to start from a given inspection plan and then verify that it the combination of design and inspection meets the target.

In equation. (2.14) we might also use the observation that the structure has survived until the point of inspection (proven strength) or even more data if available. In geotechnics often data already gathered during the building process are included (observational method) enabling sometimes a more economic design.

2.5 Target Reliability

2.5.1 Safety and economy

For the reliability-based approach and the partial factor approach the target reliability is a prescribed quantity.

Usually, the target reliability level is proposed by the engineering community (often the code committee) and endorsed by the National Authorities who finally are responsible. The common starting point is that a structure should be safe as well as economic during the design or remaining service time. The notions safe and economic have been discussed in section 2.3.

In general, one may state that the accepted reliability level with respect to ULS-failure depends on:

- The consequences of failure which may depend on the failure mode (see 2.5.6)
- The resources needed for structural dimension, in particular the costs related to additional safety
- The anticipated service life,
- The variability and uncertainty in resistance and load, including the likelihood of mistakes (as far as not covered by quality control).

In most codes only the first factor is used for explicit reliability differentiation in terms of different levels of target reliability levels. It is actually the connection between reliability-based design and risk analysis. The consequences of failure, however, are not elaborated in detail, but only a rough classification of structural types (dwellings, high rise buildings, grand stands, etc.) is given. There usually is also a link to robustness requirements and QA-procedures.

2.5.2 Reliability differentiation

Reliability differentiation depends on the consequences of failure (related to economy losses, environmental effects and losses of human lives) but may also be used as a tool to apply the available resources of society in an optimal way. As such it is one of the means to contribute to more sustainability in building industry.

For various reasons it does not seem appropriate to distinguish in reliability requirements between various materials. All structures or structural elements within a certain class of use and failure consequences should have the same reliability, whatever material present.

However, there is no reason why the same value (as for instance the in the Eurocode commonly accepted value of 3,8 for 50 years as derived over 30 years ago for CC2 buildings under variable loads only. see chapter 3.1) should automatically hold for other limit states like deflection, for other structures like masts, towers, silos or for loads with a higher degree of uncertainty (e.g. fire, impact, earth quakes). The wish for transparency in the field of reliability requires primarily the same target for comparable consequences. Differentiation, however can be accepted, provided a careful consideration is given to all aspects discussed before (life safety, consequences of failure, economical and historical aspects). The topic in relation to the Eurocode will be dealt with further in chapter 3.3.

A very special application of the reliability differentiation concerns the field of assessment of existing structures. Strengthening an existing structure in general requires a lot more efforts than strengthening a structure that only exists on paper. Based on these considerations it can make sense to lower the reliability targets for existing structures. Have in mind that when assessing an existing structure also the (remaining) design service life might be (much) shorter than 50 years. The further elaboration in the Eurocode is given attention in chapter 4.

2.5.3 Annual versus life time targets, influence of design life

Reliability targets always have to be specified for a certain event or a certain period of time (the so-called reference period). Annual values are commonly in use when dealing with human safety; in that case the value is usually independent of the design life time. The life time option is convenient if one only considers economic optimization. In practice the life time option was connected directly to a design service life of 50 years. Rules for structures for a different design service life (temporary structures), large civil engineering works) were never very clear. It seems logical to make a difference between economy and human safety dominated targets.

Confusing in this respect is also what is meant by an annual value criterion. Does it mean that during the design service life the *average* or the *maximum* yearly failure probability is decisive? As the operational difference between the life time approach and the annual average is small it makes sense to set a target for the *maximum* annual value during the design or remaining service life. This also simplifies calculations. Usually only the first and the last year need to be checked.

If failure events in various years are independent and have small similar probabilities, the relation between an annual and a 50-year target is simply approximated as (see also example 2.1):

$$P_{50} = 50 \times P_1 \quad (2.17)$$

The formula still holds for moderate correlation but complex relation arises if there is a large degree of correlation (for instance due to dominating permanent loads or large resistance uncertainties or the presence of degradation processes giving rise to the increasing failure probability at the end of the life time. This also means that there is no one to one correspondence for a large class of different structures. One can only claim an equivalence for instance between $\beta = 3,8$ for 50 years and (depending on the degree of correlation) $\beta =$ between 3,8 and 4,7 for one year. Some structures however will have higher dimensions in the one system and lower in the other. These differences may also be present in corresponding α -values and/or γ -factors.

Note by the way that there is no need to make a choice between the two approaches: one can also adapt the life time target for design and add constraints for annual failure probabilities.

2.5.4 Minimum versus average reliability

As already pointed out in chapter 2.3.3, the target reliability can be formulated for a portfolio of structures (say all bridges). In that case it may be questioned whether the target refers to the average reliability of the portfolio or to the minimum reliability (average or maximum failure probability respectively). If aiming at an economic optimum the average value makes sense. However in order to fulfil requirements related to human safety the minimum reliability level seems to be the most appropriate. Of course, such differentiation would make the derivation of design rules more complicated. A compromise could be to take the average but to add a limit to the scatter of the reliability levels in each portfolio.

2.5.5 Member versus system

This issue was already discussed in chapter 2.3.5. In most practical applications target reliabilities are used for designing individual members (including connections) for separate failure modes. However, consequences are usually related to system failures and the system failure probability may be the result of contribution of a set of system failure modes. For a part this has to be considered as the state of the art. However, in some cases we encounter exceptions as in the design of laminated timber beams or multiple pile foundations.

2.5.6 Brittle versus ductile failure

Brittle failures on the member level have in many situations more serious consequences for the structural system integrity than ductile failure. This might be a reason to use a higher consequence class and higher

reliability targets for brittle failure modes or to force the structure into ductile modes such as in capacity design used in seismic design situations. An additional aspect is the degree of warning: ductile failure modes may give some kind of pre-warning and thus time for people to leave the building in time (see also section 3.7).

2.5.7 Role of Quality Assurance

Quality Assurance (QA) procedures (for more information see EN 1990:2023 Annex B) may be carried out for design, fabrication and execution. The intention of those procedures is to ensure that the reality is in accordance with the assumptions in the verification (like schematisation and properties of materials and geometry). As far as the overall design is concerned the best way to reach this goal is to make use of independent checking procedures.

There obviously is a direct relation between QA and reliability verification. One often heard opinion is that the use of a higher level of QA leads to a lower uncertainty in the material properties and therefore provides a justification for a reduction of partial factors. Although there is some logic in this reasoning there is no recommendation (also not in the Eurocodes) to do so for new structures. For existing structures this may be different. QA in that case is very close here to the actual measuring the actual properties of the existing structure ; one may then follow the corresponding rules from statistical inference

2.6 Derivation of partial factors for semi-probabilistic design

2.6.1 Design value method

The theory of structural reliability offers several methods to find partial factors given a prescribed target reliability β_t level. The simplest one is the method using standardised importance (sensitivity, weight) factors α as already introduced in formula (2.10):

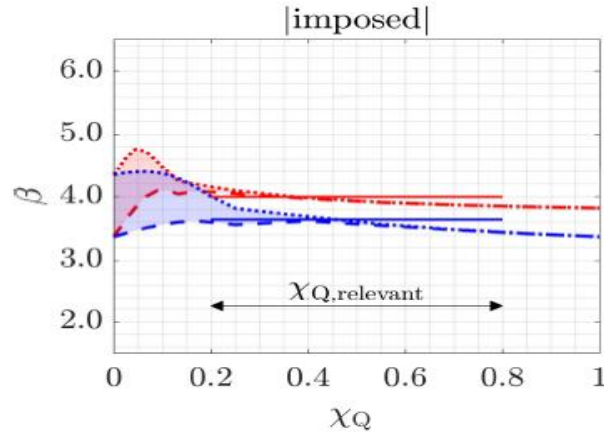
$$X_d = \mu_x - \alpha_x \beta_t \sigma_x \quad (2.10)$$

This formula holds for the normal distribution. Given the design values the partial factors can be derived using (2.11).

In EN 1990:2002 the choice was made that for $\beta_t = 3,8$ and a reference period of 50 years, the value $\alpha_x = 0,8$ for the dominant resistance variable and $-0,7$ for the dominant load variable; for non-dominant variables the values are $0,4 \times 0,8 = 0,32$ and $-0,4 \times 0,7 = -0,28$. (See also section 3.1.3). These choices are also the basis for the load combination rules. Where relevant the various loads have to be considered as dominant one by one. The practical advantage of the design value method is that all load and resistance design values can be derived individually, assuming a consistent application on both load and resistance side and taking care of limits as mentioned in EN 1990:2023 Annex C.

The proposed α -values, of course, can be varied to improve the result in a certain application. Usually, the results of a specific choice are checked by considering a set of structural members and load combinations and calculating the corresponding β -values (like for instance as presented in Figure 2.2 taken from Meinen and Steenbergen, 2018.) The result should give β -values close to the target over the whole range of application. Close in this respect may refer to the average β or to the minimum value (see also section 2.6.3). For example, for the seismic design situation the values used in deriving the partial factors in the second generation Eurocode 8 are 0,42 for the resistance and -0,91 for the seismic action effect (Franchin and Noto, 2023).

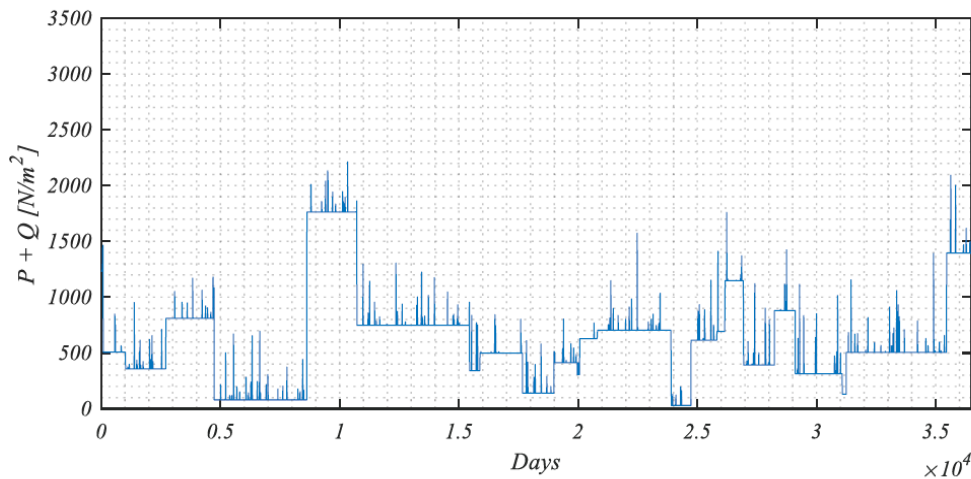
Figure 2.2. Resulting reliability index for steel members, designed according to the Eurocodes,, for the limit state of bending under self-weight, permanent and imposed loads for a 1 (red) and 50 (blue) year reference period as a function of χ_Q which represents the ratio of the variable and total load; the dashed and dotted lines refer to a zero and maximum contribution of permanent loads respectively; the full lines show the average value in the area of interest (illustrative example only)



Source: Meinen and Steenberg, 2018

When the combination of loads is considered, it would be too conservative to assume that the maxima of two or more-time dependent loads in the reference period will occur at the same point in time. Where this is taken into account the so-called FBC models (Ferry Borges and Castanheta, 1971) are recommended (Figure 2.3). Note that time dependency is also important for the safety formats for degrading systems and finding load factors for fatigue loads and/or environmental actions.

Figure 2.3. Ferry Borges and Castanheta load model (FBC) for imposed loads (sustained and intermittent components) (illustrative example)



Source: Faber, Koehler & Sorensen, 2018

2.6.2 Minimizing the deviation from the target

More advanced methods aim at minimizing the averaged difference between the target and the reliability levels obtained by a certain set of partial factors γ for a certain field of application (see EN 1990:2023 Annex C):

$$\min \sum w_i (\beta_i(\gamma) - \beta_t)^2 \quad (2.18 \text{ a})$$

$$\text{or } \min \sum w_i (Pf_i(\gamma) - Pf_t)^2 \quad (2.18 \text{ b})$$

$$\text{or } \min \sum w_i [(R_i(\gamma) - R_i(\beta_t)) / R_i(\beta_t)]^2 \quad (2.18 \text{ c})$$

The summation is over the selected set of structures.

The first option mentioned (2.18 a) is the one most commonly used. See (Baravalle, 2017) for further discussion. The method will further be referred to as the optimization method. The weight factors w_i may reflect the number of structures of type i or even economic considerations. In addition, there may be or should be constraints like a minimum beta or a maximum deviation. Other constraints may relate to having material independent load factors or minimum partial factors for permanent loads.

One should keep in mind that a minimization problem does not need to have a unique solution. There may be a set of different solutions for the vector γ leading to the same structural dimensions (Madsen et al, 1986). For instance, in the simple case of example 2.1 one may increase the load factor and reduce the resistance factor accordingly, leading to the same structure. So, in order to have a technically meaningful answer one has to make a proper choice or introduce an additional constraint. One might for instance fix the partial factor for the self-weight and study the results for several values. Alternatively one may fix the values on the resistance side; actually the example calculations in Annex B2 of this report have been performed along that line, using the gamma values according to EN 1992-EN 1999).

Example 2.4 Simple code calibration example

This example is only intended to show the procedure with minimum mathematics. It is not intended as a useful example as far as the numerical values are concerned.

In this example the following steps will be taken:

1. Choose a portfolio of structures and design situations with corresponding limit state functions and weights w_i ; set the target reliability
2. Define the set of statistical properties
3. Define a verification code format, estimate a set of partial factors and calculate for each design situation (exactly) the minimal required structural dimensions.
4. Calculate for design situation i the corresponding reliability index β_i and for the whole set the weighted average $\beta = \sum(w_i \beta_i) / \sum w_i$

If the (weighted) average value of β_i over the set of design situations (step 4) is about equal to the target, we have a reasonable proposition for the partial factors; other wise we have to repeat steps 3 and 4.

One might also go further and try to find a minimum for the relative deviation of the target, which may be defined as: $D = \sum w_i \{ (\beta_i - \beta_t) / \beta_t \}^2$. But that is outside the scope of this simple example.

Step 1: Definition of portfolio and choice of target

Let us consider a portfolio of two materials and two loads. The limit state function is given by:

$$g = R - F_1 - F_2$$

The ratio of the variable loads to the total load is governed by the parameter χ , defined as:

$$\chi = F_{2k} / \{F_{1k} + F_{2k}\}$$

where χ ranges from 0,2 to 0,8 with intervals of 0,2. So actually we will take the values $\chi = 0,3$, $\chi = 0,5$ and $\chi = 0,7$. The two materials, the two loads and the three ratio values χ will have the same weight w . In total we have $2 \times 3 = 6$ design situations. The target reliability β_t is set as 3,8 for the reference period under consideration.

Step2: Statistical properties

The statistical properties are presented as follows:

	distribution type	mean value	coefficient of variation V	characteristic value
material 1	Normal	m_1	0,10	$m_1 (1 - 1,64V_1)$
material 2	Normal	m_2	0,20	$m_2 (1 - 1,64V_2)$
load 1	Normal	1	0,10	1.0
load 2	Normal	$\chi / (1 - \chi)$	0,20	$\chi / (1 - \chi)$

The characteristic values of the loads are taken equal to their mean value; the mean value of the first load has been set equal to 1,0. The mean for the resistance follows for each design from the design procedure in steps 3 and 4.

Step 3: Design format and partial factors

As design format we choose:

$$R_k / \gamma_R > \gamma_{F1} F_{1k} + \gamma_{F2} F_{2k}$$

Let the estimate for a set of partial factors be: $\gamma_{R1}=1,2$, $\gamma_{R2}=1,5$, $\gamma_{F1} = 1,2$ and $\gamma_{F2}=1,4$.

Step 4: Resulting reliability level

The first design situation where $\chi = 0,3$ leads to:

$$R_{1d} = 1,2 \times 1 + 1,4 \times 0,43 = 1,8; \quad R_{1k} = 1,8 \times 1,2 = 2,16 \quad \text{and} \quad m_1 = 2,16 / (1 - 1,65 \times 0,1) = 2,6$$

Using FORM this leads to $\beta = 3,83$. If this analysis is carried out for all 6 design situations we find beta values ranging from 2,8 to 4,5 with 3,5 as an average. As this is below the target a second trial is performed:

Steps 3 and 4: second trial:

The second choice for the partial factors are: $\gamma_{R1}=1,3$, $\gamma_{R2}=1,8$, $\gamma_{F1} = 1,2$, $\gamma_{F2} = 1,4$. This leads to beta-values ranging from 3,3 to 4,3 with an average of 3,8. This average is okay but the range may, from an economical or political point of view, be considered as quite wide. This can be improved by further optimizing, but, as stated, this is not in the scope of this example. In some applications there is also a requirement on a minimum reliability level.

In the above example the presence of two variable loads in one limit state verification is not considered. If we want to do so, we have to model the variable loads in more detail (e.g. using FBC models as indicated in Figure 2.3) and use time dependent reliability calculation procedures as described in section 2.4. As an approximation one could simply add the maxima of variable loads in the reference period, but that would lead to an overestimation of the failure probability as the maxima of two or more variable loads need not to occur on the same point in time.

2.6.3 Closing remark

Whatever method is chosen to define reliability targets and establishing partial factors, a general recommendation will be that abrupt large changes in the successful design procedures from the past should be avoided, unless there are very convincing reasons. The behaviour of the structures designed according to the rules of the past present important empirical evidence from a major experiment that should be taken into account. On the other hand:

Periods of 20 or even 50 years may be too short to check safety against e.g. design values of extreme winds having return periods of over thousand years.

Some structures may have been uneconomically overdesigned (should be avoided of course).

Results from research (hidden/unhidden safety; more refined models) should be incorporated.

The conclusion again has to be that we are looking for the balanced combination of practical experience, theory and research developments.

For more information see also (Gayton et al., 2004) and (Silva et al, 2014).

3 The Eurocode semi-probabilistic verification method

3.1 Historical background

3.1.1 Chronological overview

CEN's decision to move the EN Eurocodes into a second generation in 2015 following and with the important support of the European Commission and EFTA through mandate M/515, was, preceded by a long history of European standardisation of design standards for structural engineering. Based on the Rome Treaties of 1957, the Coal and Steel Commission of the European Community began drawing up seven Eurocodes with rules for the serviceability and structural safety of building structures from 1975 onwards. For this purpose, the European associations of the corresponding construction methods were asked for support.

After the decision of the European and EFTA member states in 1989 the development of the so-called ENV Eurocode versions for structural design began, the first of which were published in 1992. Due to difficulties in harmonizing all the aspects of the calculation methods, the ENV Eurocodes included "boxed values" which allowed member states to adapt the safety level for use on their territory and for which the partial safety factor concept was very much suited.

As the basic document of the ENV series establishing design principles and requirements as well as the approaches for safety, serviceability and durability of structures, the Eurocode 1, ENV 1991-1 "Basis of design and action on structures – Part 1: Basis of design", was published in 1994 (CEN, 1994).

After 1998 the conversion of the ENVs to ENs under the full normative leadership of CEN began. In the course of this, it was decided to separate the standards for basis of design and actions on structures, i.e. to dedicate an EN 1990 to basis of design and to leave the regulations for the load approaches in EN 1991.

After the conversion of ENV 1991-part 1 into EN 1990 (CEN, 2005/2010) in 2002, in 2007 the publication of EN Eurocodes was concluded. The implementation programme entered the coexistence period, during which the EN Eurocodes were used in parallel with National Standards that had the same scope.

By the end of this coexistence in 2010 the Eurocodes are introduced in the EU countries including the corresponding national documents, which gave the details of how to apply EN- Eurocodes in Member States.

Over the past decade the evolution of the Eurocode suite has been progressed to develop the second generation of EN Eurocodes. This work has included the extension and updating of the principles and fundamentals of the basis of structural and geotechnical design in EN 1990.

In the period until the 1990, for the purpose of practical design, there was a mix of national and material oriented design standards, partly based on the classical allowable stress approach, partly on load (or load & resistance) factor limit state concepts. However, in many research centres as well in pre-codification committees and some code committees both probabilistic and semi-probabilistic methods (as presented in chapter 2 of this document) were well developed. The Eurocodes were committed to the semi-probabilistic partial factor design concept from the very beginning (Sørensen, 2004). At all times of development, the Eurocodes reflected the current international state of the art of safety concepts in civil engineering. Some of the most important elements are briefly described hereafter.

3.1.2 Reliability methods applied in the Eurocodes.

Design of structures can be performed either based on a reliability analysis or on the design value format (Sørensen, 2004). Within the Eurocodes suite of the ENV 1991 (CEN, 1994) and EN 1990 (CEN, 2005/2010), the design values of basic variables are determined from their characteristic values using partial (γ) and combination (ψ) factors (partial factor design).

It has been agreed that numerical values of these factors can be determined using several methods (Gulvanessian et al, 2012):

- a) calibration to past experience
- b) application of probabilistic reliability theory and calibration

For evaluation at a linearized limit state, method (b) can also be expressed as:

the use of the probabilistic FORM (see hereafter) level II method and after calibration derivation of design values for action and resistance.

According to (Gulvanessian et al, 2012) explaining EN 1990 (CEN, 2005/2010): "... as a rule, both procedures (calibration to past experience and application of probabilistic reliability theory) for the determination of γ and ψ factors (level 1) are combined in order to increase the credibility of obtained results." This is visualised by Figure 3.1.

Figure 3.1. Overview of reliability methods

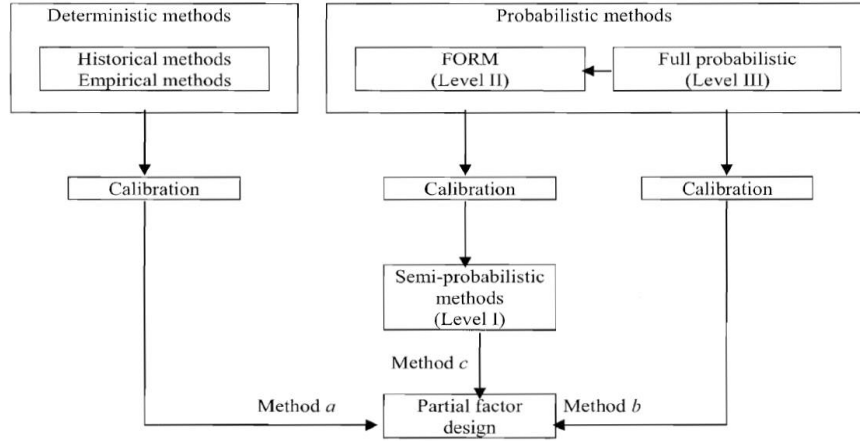


Figure C1 - Overview of reliability methods

Source: EN 1990 (CEN, 2005/2010), Annex C.

The design with partial factors should be the usual method for practice in structural engineering. As far as we talk about probabilistic methods in EN 1990, we can say that EN 1990, especially its Annex C is somehow oriented to level II methods, but not only: it also allows safety evaluation according to full or quasi full probabilistic (level III) methods without further ado.

As mentioned EN 1990 (CEN 2005/2010), especially Annex C is based on concepts of "First Order Reliability Methods" (FORM, an advanced level II method [DIN, 1981]. Consider for the sake of further discussion the basic case where the limit state function is given by $Z = R - E$ with R and E independent normally distributed variables representing resistance and action. In that case we have:

$$\mu_Z = \mu_R - \mu_E ; \quad \sigma_Z^2 = \sigma_R^2 + \sigma_E^2; \quad P_f = \Phi(-\beta) \quad (3.1)$$

where β is the safety or reliability index, named as such by Cornell in 1969 (Cornell, 1969) that can be considered as invariant for mechanically equivalent problem formulations (Kjerengtroen et al 1996). For the more general non-linear limit state function $Z = g(X_1, X_2, \dots, X_n)$ with non-normal variables a suitable transformation from the X -variables to the standard zero mean unit standard deviation Gaussian u -variables was proposed by Hasofer and Lind in 1974 (Hasofer et al, 1974). For independent variables X_i the transformation is given by:

$$\Phi(u_i) = F_{X_i}(X_i) \text{ or simply for normal variables: } u_i = \frac{X_i - \mu_i}{\sigma_i} \quad (3.2)$$

Alternatively, one could choose the tail approximation according to (Rackwitz and Fiessler, 1978).

Given this transformation the reliability index β could be conceived as the minimum distance from the origin of the normalized coordinate system to the failure surface $g(u) = 0$ (the so-called design point):

$$\beta = \sqrt{u_1^2 + u_2^2 + \dots + u_n^2} = \sqrt{(\alpha_1 \beta)^2 + (\alpha_2 \beta)^2 + \dots + (\alpha_n \beta)^2} \quad (3.3)$$

The sensitivity or weighting factors “ α_i ” are equal to the partial derivatives $\partial g / \partial u_i$ at the design point and could be interpreted as direction cosines of the stretch from the origin to the design point. It can easily be proven that the sum of the squares should be equal to 1.

3.1.3 Values for key parameters

The choice of a value for the *target reliability* β_t was subject to some evolution process. In the various national studies at that time (around 1980) there was not much uniformity in the presentation of the results with respect to the reference period. In some studies, the one-year period was chosen, in other studies the 50-year period. The transition between the two results were (neglecting correlation) simply presented as

$$P_{50} = 1 - (1 - P_1)^{50} \approx 50 P_1 \quad (3.4)$$

or in terms of reliability index and n representing the number of years:

$$\beta_n = \Phi^{-1} [\Phi^n(\beta_1)] \quad (3.5)$$

The scatter between the various studies was quite large (see Table 3.1); the scatter between various materials, types of elements and loading situation was even larger, but showed similarities between the various studies.

In the German national regulation (DIN, 1981) basically, an (annual) failure probability of $p_f = 1 \times 10^{-6}$ being equivalent with $\beta = 4,7$ for ULS and 3,0 for SLS was chosen. In the light of predominantly permanent loads this failure probability was considered as relatively high: as (real) permanent loads are not time dependent this value holds for a reference period of 1 year as well as for any time. Therefore, it was proposed to define $\beta = 4,7$ as anchor value for a limit state function considering variable loads only, when referring to one year, for normal failure consequences (CC=2). Other documents that were on the table at that time and pointing more or less in the same direction are presented in Table 3.1. The choice for a life time reliability measure is also already present in the CEB-bulletins 116 (Part 2) and 124/125 (Vol 1, appendix 1). It is there stated that the reference period for the reliability should be chosen “identical the anticipated time of use of the structure”. As a central value (medium number of endangered persons and medium economic consequences) 10^{-5} ($\beta=4,3$) was tentatively proposed. In addition, a tolerable annual individual risk was set at 10^{-5} . As the probability that a user of a structure is killed due to a collapse (depending on the situation) is less than 1,0 the collapse probability of the structure may be larger. The reduction from 4,3 to 3,8 was probably the result of a number of national studies. It has never been an important point of discussion during the drafting of the first EN 1990.

Note:

It should be realized that the relevant parts of the CEB-bulletins 116 and 124 were not mere products of the concrete industry but the joint product of all professional organisations of the Liaison Committee (CEB, fib, ECCS, RILEM, IASS, CIB) and the JCSS. The title was in both cases “Common unified rules for different types of construction and material”

Anyhow, already in ENV 1991-1 (CEN, 1994), the target reliability index for the ULS was chosen equal to $\beta = 3,8$ (CEN, 1994) (CEN, 2005/2010) (Gulvanessian et al, 2012) (DIN, 1981) related to a period of 50 years (for SLS $\beta = 1,5$). In current EN 1990 (CEN, 2005/2010) different target reliabilities can be found depending on various reference periods and consequence classes (CCs). Reliability differentiation was earlier a subject of discussion in for instance CEB 124 and NKB, mentioned in Table 3.1.

The Design Value Method also asks for choices for the *sensitivity factors* α_x . For the Eurocodes, in line with e.g. (DIN, 1981), the sensitivity factors α_E (action side) and α_R (resistance side) were predefined as to $\alpha_E = -0,7$ and $\alpha_R = 0,8$, thus allowing to treat action and resistance side independently in the design, and where the action

side was given some higher importance by increasing the α_E value's amount from 0,6 to 0,7 (deviating from the unity condition, see above). This gave a certain flexibility for cases where the numeric values of α_E and α_R are not completely satisfied. One can allow for an undercut or exceed the target reliability β_t by 5% if the range of the standard deviation ratio is in between $0,16 < \sigma_E/\sigma_R < 7,6$ (CEN, 2005/2010) (König et al, 1982).

In case of more than one load or resistance variable it was on the basis of theoretical considerations decided to use 0,4 α_x for the non-dominant parameters. On the loading side this formed the basis for the derivation of the Ψ and ξ factors (Ferry-Borges et al, 1971) (Turkstra et al, 1980) (Vrouwenvelder, 2002) (Vrouwenvelder et al, 1996).

Characteristic values for loads and resistances were in principle defined as 5% fractile for strength parameters, 50 % fractile for low variable permanent loads and 2% fractile for annual maxima of variable loads. Given these choices, the partial factors could be defined in the frame work of the Design Value Method. This leads however to different values than the well-known $\gamma_G = 1,35$ and $\gamma_Q = 1,5$ which were also chosen on historical grounds (see Figure C1). For more information see Chapter 3.4 and Annex B2. The resulting ψ -values of EN 1990 (CEN, 2005/2010) are conservative (Vrouwenvelder, 2002) (Vrouwenvelder et al, 1996).

Table 3.1. Overview main results of historical documents.

Country	Study	Ref [year]	β_t 1 yr	β_t 50 yr	α_R	γ_G	γ_{Gco} m	γ_Q	Ψ_0	k_{F1}	k_{F3}
USA	Galambos	1982		3,0							
Germany	DIN	1981	4,7		0,8	1,3 ^a		1,3 ^{a,b}	x		
Germany	Koenig/Hosser	1981	4,7	3,8	0,8	1,3		1,3	x	0,9	1,2
Scandinavia	NKB	1978	4,2			1,0 ^c		1,3 ^c			
Belgium	SECO	1984		3,3		1,35		1,5	x		
Netherlands	NEN	1987		3,6 ^d		1,3	1,2	1,5		0,9 ^e	1,1 ^e
UK	CIRIA	1977	>4								
International	CEB-112	1976			0,8						
International	CEB-116	1976		4,3		1,2		1,4		x	x
International	CEB-124/125	1978		4,3		1,2		1,4			
International	CEB-127/128	1980	?						x	x	x
Euro Com	Eurocode 1	1984				1,35		1,5	x		
International	ISO	1998		3,1 ^f	0,8				x		
^a here is also a $\gamma_{sys} = 1,1$; ^b char value defined as $Q_k = Q_{120\text{ yr}}$ ^c in combination with high material factor ^d $\beta_t = 2,6$ for wind ^e variable actions only ^f economic optimum ^x several values, most in the order of 0,4-0,7											
Background studies mentioned in Table 3.1: USA Galambos T., B. Ellingwood, J. G. MacGregor, C. Allin Cornell, Probability Based Load Criteria: Assessment of Current Design Practice, Journal of the Structural Division, ASCE, Vol. 108, No. ST5, May, 1982 Germany: Koenig, Hosser, The simplified level II method and its application on the derivation of safety elements for level 1, Technische Hochschule Darmstadt (1981) Koenig Hosser, Sicherheitsanforderungen für die Bemessung der baulichen Anlagen nach den Empfehlungen des NABau, Bauingenieur 57 (1982) NaBau, Grundlagen zur Festlegung von Sicherheitsanforderungen für bauliche Anlagen, DIN Deutsches Institut für Normung, Berlin, Köln, Beuth Verlag, 1981 DIN – Deutsches Institut für Normung e.V. (1981). General Principles on the Specification of Safety Requirements for Structures. 1 st edition in English, 1981. Belgium: R. D'Havé and P. Spehl, Principes généraux de détermination de la sécurité et de l'aptitude au service des structures, Annales des travaux publics de Belgique n°1 of 1984. The Netherlands: Vrouwenvelder, A.C.W.M., Siemes, A.J.M., Probabilistic calibration procedure for the derivation of partial safety factors for the Netherlands building codes. HERON, 32 (4), 9-31, 1987											

Scandinavia:
 NKB – The Nordic Committee on Building Regulations, Recommendation for Loading- and Safety Regulations for Structural Design. NKB-Report No. 36. (1978)
 NKB – Nordic Committee and Work Reports (1999:01 E). Basis of Design of Structures. Proposals for Modification of Partial Safety Factors in Eurocodes. (1999)
 UK:
 Ciria, Rationalisation of safety and serviceability factors in Structural codes, Ciria report 63, UK, 1977
 EC:
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 CEB::
 Bulletins 112, 116, 124, 125, 127, 128
 Mathieu H. General overview of structural reliability problems, Updated comprehensive abridgement, of the CEB manual " Structural safety CEREMA, 1995

3.1.4 Conclusion

EN 1990, *Basis of structural design*, as head document in the Eurocodes suite establishes for all the structural Eurocodes the principles and requirements for safety, serviceability and durability of structures. It further describes the basis of design and verification and provides guidelines for related aspects of structural reliability. It gives all the operative material independent rules (e.g. partial factors for actions, load combination expressions for ultimate and serviceability limit states), and therefore EN 1992 to EN 1999, which do not provide material independent guidance, cannot be used without EN 1990.

EN 1990, *Basis of structural design*, as a worldwide recognized standard, has undergone a progressing development in the past. Care was always taken to ensure that this and its predecessor documents were kept up to date with the state of the art. Thus, it allows not only an implementation of the safety level based on simple empirical values or on level II methods, but also fully probabilistically oriented evaluations can easily be considered if allowed by the National Annex.

The development of this document also shows its flexibility in terms of application to the most diverse issues, starting with those of structural engineering, through those of earthquakes, up to geotechnics.

Nevertheless, EN 1990 must be further developed as a part of the ongoing need for standards to be updated, partly because of presently still existing shortcomings, partly because of the new challenges that will come. (See also chapter 6.)

3.2 The verification formats according to EN 1990:2023

The basic code format for the verification of the ULS in the Eurocode is given by (inequality (8.1) in EN 1990:2023):

$$E_d \leq R_d \quad (3.6a)$$

where E_d is the design value of an action effect in a certain member, cross section, stress concentration point or connection in the structure and R_d the design value of the corresponding resistance. The verification is carried out for each specific design situation and for all limit states, load combinations and load arrangements.

For the ultimate limit state of excessive deformation there is a similar formulation (inequality (8.2) in EN 1990:2023):

$$E_d \leq C_{d,ULS} \quad (3.6b)$$

where E_d is the relevant displacement or deformation and $C_{d,ULS}$ a corresponding limit value; numerical values may be given in material related codes; no recommendations are given for the partial factors in EN 1990:2023. In EN 1998 (seismic analysis) the deformation criterion is used as a mean to prevent brittle failures.

For the resistance R_d the design value is calculated from:

$$R_d = R\{\eta X_k/\gamma_m; a_d; \Sigma F_{Ed}\} / \gamma_{dR} \quad (3.7a)$$

In this expression a distinction is made between a factor covering the uncertainties in the resistance model and a factor covering the uncertainties in the material properties. A variant formulation is given in EN 1990:2023, equation (8.21):

$$R_d = R\{\eta_1 X_{k1}; \eta_i X_{ki} \gamma_{mi}/\gamma_{mi}; a_d; \Sigma F_{Ed}\} / \gamma_{Rd1} \quad (3.7b)$$

In most design situations the expression is simplified to:

$$R_d = R\{\eta X_k/\gamma_M; a_d; \Sigma F_{Ed}\} \quad (3.8)$$

It may be surprising to find loads or load effects in this expression for the resistance. This, however is necessary to cover all possible limit state expressions. For instance, if the failure domain is governed by the limit state expression $M/M_p + (N/N_p)^2 > 1,0$ (or more advanced models as often present in EN 1992 to EN 1999), it is otherwise impossible to use equation (3.6) as verification format.

The design value of the effect of actions E_d for a specific combination of actions is calculated from EN1990:2023 eq (8.3):

$$E_d = \gamma_{sd} E \{\Sigma(\gamma_f \psi F_k); a_d; X_{Rd}\} \quad (3.9)$$

In this expression a distinction is made between a factor γ_{sd} covering the uncertainties in the load effect calculation and a factor γ_f covering the uncertainties in the load. Note that the uncertainty in the load model as such is also included in γ_f . In most design situations this may be simplified to EN1990:2023 eq (8.4):

$$E_d = E \{\Sigma(\gamma_F \psi F_k); a_d; X_{Rd}\} \quad \text{and} \quad \gamma_F = \gamma_{fsd} \gamma_f \quad (3.10a)$$

This simplification is correct if the relation between load and load effect is linear or if the model uncertainty in the load effect calculation is absent or relatively small. The simplification may be unsafe in the non-linear case where the load model uncertainty is relatively large and the application of partial factors on the effects of actions is more adverse than the application of partial factors on actions. This type of load effect functions $E(F)$ is often referred to as "under linear", "under proportional" or "convex". The code refers to certain limit states in cable and membrane structures as well as some geotechnical structures where this might be the case. The best option is of course to use the basic equation (8.3) in EN1990:2023, presented above as (3.9). Another option is to use EN 1990:2023 eq (8.5):

$$E_d = \gamma_E E \{\Sigma(\psi F_k); a_d; X_{Rd}\} \quad \text{and} \quad \gamma_E = \gamma_{fsd} \gamma_f \quad (3.11b)$$

Some remarks (see also section 3.8):

- use of the most adverse case of (8.4) and (8.5) leads to over conservative and uneconomic results.
- The load factors derived for the linear case are not always appropriate for cases of strong non-linear behaviour. We may leave the field of application and end up with too large deviations from the desired reliability level. In case of doubt advanced full probabilistic verifications are recommended.

- The situation becomes even more complex in the case of load combinations of permanent, variable and prestressing loads

It should further be noted that load effects may also depend on the structural properties. In case of a linear elastic analysis this may be the stiffness but also strength parameters can be involved. It is not always obvious whether, to be on the unfavourable side, in a certain design situation upper or lower design values for a_d and X_{Rd} should be taken. A famous example is the case of the shear force in a girder. By using the lower design values for the bending moment capacity, the maximum value of the shear force is limited and the possible brittle mechanism of a shear failure may be missed. In most cases this is not taken into account explicitly but where important additional requirements are formulated to guarantee a ductile behaviour, like in capacity design in seismic engineering. However, these requirements are seldom calibrated to a safety target (see also section 3.7).

The summation symbol in the above equations stands for a load combination, not necessarily a mere addition. The ψ can be either “1,0”, ξ , ψ_0 , ψ_1 or ψ_2 . As an example, consider the permanent and transient design situation in EN 1990:2023 equation (8.12). In that case we have:

$$\Sigma(\gamma_F \psi F_k) = \Sigma \gamma_{G,i} G_{k,i} + \gamma_{Q,1} Q_{k,1} + \Sigma \gamma_{Q,j} \psi_{0,j} Q_{k,j} + (\gamma_P P_k) \quad (3.12)$$

This formula actually means that for the verification of the ULS reliability we have to consider the combination of all factored permanent and variable loads, but that we can include the combination factor $\psi_{0,j}$ for all but the leading variable action. All actions should be considered as leading one by one. The more refined load combination rules (8.13) and (8.14) serve the same purpose. In a similar way EN 1990:2023 shows combination rules for the limit states for the accidental and seismic design situation, for fatigue and for serviceability.

Not all actions or action components have an unfavourable effect. This is relevant both for (3.7) and (3.9). For instance, within certain bounds, the presence of a normal compression force in a concrete or masonry cross section may increase the bending resistance. Other examples are overturning or uplift. If that is the case it may be unsafe to consider the upper design values in the load combinations.

For variable actions one simply can ignore the load if it acts favourable and take γ_F equal to zero. However, one should keep in mind that loads sometimes follow patterns. For wind for instance, physics may dictate that a whole part of a façade is under suction, regardless of the influence on an action effect. The instructions for those type of load arrangements is taken care of in EN 1991.

In case of favourable permanent loads one should replace γ_G by $\gamma_{G,fav}$ or $\rho \gamma_G$. The first one is used if the net total contribution of a single permanent load source is relevant and gives a favourable load effect. The second one is applied for cases where it is relevant to distinguish between parts of a single source permanent action that act favourably and other parts that do not. It is then important to consider the degree of correlation between load effects resulting from different parts of the same structure. This is particularly important for the limit states of static equilibrium with no resistance term being active. In those cases, the terms stabilizing and un-stabilizing are used. In structural limit state verification, the effect of partially favourable and unfavourable contributions of permanent actions usually is small. As it is quite cumbersome to take these effects fully into account, EN 1990:2023 has introduced the so-called single source rule. Reference is made to section 3.5 for discussion on the accuracy of this approximation.

3.3 Target Reliability

3.3.1 Target for consequence class CC2 and reference period 50 year

According to annex C (informative), the basic target for design of new structures is a reliability index $\beta_t = 3,8$ for a reference period of 50 years and consequence class CC2. As an alternative, EN 1990:2023 Annex C offers annual values with a conversion based on independent failures in subsequent years. In this chapter we will not consider this annual option once more. Reference is made to chapters 2 and 4.

Actually, the beta value of 3,8 (or one-year values in the range 4,2 to 4,7) was derived on the basis of compromises as well as calibration studies (see section 3.1) at the end of the last century. The calibration was mainly intended to derive partial factors leading on average to the same structural dimensions as before, but

with less scatter in reliability. To some extent one should be careful to use the same target in combination with other (improved) probabilistic models for the random variables (see discussion on hidden safety).

Note: See also Annex B

3.3.2 Differentiation due to consequences of failure

The CC2 reliability level of 3,8 may be adjusted in the case of larger or smaller consequences of failure. In chapter 4.3 of EN 1990:2023 five consequence classes (CC0 to CC4) are defined on the basis of human safety and socio-economic considerations. According to Annex C the corresponding reliability indices changes to 4,3 and 3,3 respectively for CC3 and CC1. This indicates a difference of a factor 3 to 10 in consequences. Note that EN 1990:2023 translate the change in beta into a change in some partial load factors by a factor k_{Fi} . This can easily be incorporated in a calibration procedure if needed. In the current 2002 version of the Eurocode actually the numerical values for k_F have been chosen quite intuitively although some background was available (see chapter 3.1).

The consequences in case of structural failure may depend heavily on the type of structure. For that reason, the Annexes A1 to A4 give specific examples for the detailed categorisation for respectively buildings (A1), bridges (A2), towers and masts (A3) and silos and tanks (A4). In principle the consequences for the same subcategory are intended to be in the same order regardless the type of structure.

Note finally that from theoretical considerations the target should also depend on the recourses needed to improve the level. EN 1990:2023 does not explicitly accommodate for this aspect, but can be implicitly accounted for, for instance in accidental and seismic design.

3.3.3 Brittle versus ductile behaviour

Sometimes the presence of brittle behaviour is considered as a reason to choose higher safety levels. Of course, brittle behaviour can lead to less redistribution in case of overloading, and due to the lack of warning possibly more victims. Actually, it is primarily a matter of higher consequences in case of failure and may be settled that way (see also 2.5). EN 1990:2023 does not give further guidance. To some extent the task to provide adequate rules for brittle failure is left to the material related codes and the National Annexes (for instance via Table EN 1990, A1.1). Some specific guidance is given in EN 1998 (capacity design rules for seismic loading). Note by the way that sudden failure without pre-warning also sometimes can occur in structures with ductile materials. Further information can be found in section 3.7.

The question of safety under brittle failure is in a pronounced way present in the design and assessment of glass structures as glass is an ideal brittle material. Thoughtful design and construction methods can help to reduce the warning and redistribution problems, otherwise the use of adjusted target reliability levels should be considered (see also chapter 3.6.2).

3.3.4 Differentiation on structure type

The central target value of 3,8 in EN 1990:2002 for CC2 has mainly been derived on the basis of studies for buildings only. It has never been shown that this value can be justified for other types of structures like bridges, silo's, towers, masts, geotechnical structures, etc. As long as there is no proof that the value is highly uneconomic or unsafe there seems however no reason to change it. In EN 1990:2023 no explicit distinction of this type for that reason has been made. Implicitly there is a higher (annual) reliability level for many types of bridges as the design life time is 100 years. This is on top of an often-higher reliability when bridges are classified as CC3-structures.

3.3.5 Other types of limit states and design situations

Note that the background of the standard ULS target is limited to permanent and transient design situations only.

For the *accidental design* situations EN 1990 and EN 1991-1-7 do not give any specific reliability target. The background document to EN 1991-1-7:2002 refers to ISO2394 where for the loading an annual exceedance probability of the load of 10^{-4} is recommended. This is, by and large, in line with the findings in (Hingorani et al, 2019) and leads to lower values for the design load than would follow from $\beta = 3,8$ for 50 years. On the resistance side, partial factors equal to 1,0 are usually chosen. Also, this indicates that (for economic reasons)

a lower reliability level is considered as appropriate. In a qualitative way this is a rational way of spending resources, but more scientific research is needed to make quantitative.

In the case of unforeseen actions usually a damage scenario is specified and a certain reliability level for the damaged structure for some specific (usually short) period of time is implicitly or explicitly required. The actual reliability is then a combination of the likelihoods of the damage and later failure. No specific background for the Eurocode rules on this point are known.

The new generation of EN 1998 *for seismic design* sets a reliability target in terms of annual probability of exceedance of collapse (which is close to the formal limit state NC in seismic analysis) equal to 2×10^{-4} corresponding to $\beta = 3,5$. This is close to the ISO 2394:2015 recommendation $\beta = 3,3$ in Table G.4 for the central category of consequences, relatively large costs of safety measures and a high variability. Note also that this target is meant for the system level where Eurocode design is often limited to the member level.

In *fire design* a procedure aiming at the level of a reliability-index of 3,8 for 50 years is now present in EN 1991-1-2. The reliability for the traditional classification system based on a minimum time to collapse if exposed to the ISO standard fire curve is not clear.

For the *limit state of EQU* a target reliability differentiation similar to *STR* is possible and a similar set of life time or annual reliability targets. However, the effect on the design in general will be less.

In the case of *fatigue* the target is not different from the target for other failure mechanisms if similar consequences are expected; relevant in this respect is the distinction between damage tolerant and safe life (see also 3.9.1).

For the verification of *serviceability limit states* a distinction should be made between reversible and non-reversible limit states. The value of target reliability index $\beta = 0$ is considered in Eurocodes for checking the reversible serviceability limit state and $\beta = 1,5$ for irreversible serviceability limit state considering the reference period of 50 years. For selection of the serviceability reliability level, see also ISO 2394 and JCSS PMC.

3.3.6 Influence of time: design life, temporary structures, transient design situations, execution

See 2.5.3 for a general technical introduction

The standard design service life in Eurocode is 50 years. Other options for choice for the design service life, ranging from 10 to 100 years, are given. Consequences for the design are relatively clear for the treatment of deterioration mechanisms, but whether partial factors (or characteristic values) should be changed is not clear. The following observations can be made:

It makes sense to aim for a higher target annual reliability level for a longer design life time if the effects of damage in case of collapse are dominated by economic consequences; investments in structural reliability are more cost efficient if you can profit for a longer time;

For human safety dominated failure mechanisms an annual target independent of the design life-time seems to be more appropriate;

For temporary structures that are re-used several times (apart from other safety considerations) the total time of use should be considered;

Also, transient design situations should be considered for the total time of their duration.

The present EN 1990:2023 does not have operational rules. The above considerations, however, should give sufficient guidance for additional rules in the National Annexes. As an example: for CC2 the Dutch code keeps the reliability index of 3,8 as a target for all (accumulated) design life time larger than 15 years. For shorter periods an annual limit is $\Phi(-3,8) / 15 = 2 \times 10^{-5}$.

3.3.7 Material deterioration (fatigue, corrosion, etc.)

Although $\beta = 3,8$ was derived for non-deteriorating structures, it is reasonable to include deterioration simply as one of the mechanisms contributing to failure. One may include here inspection and repair. If annual probability is used the Table C2 standard present annual reliability index of 4,7 in EN 1990:2023 Annex C definitely is too high; a value of say 4,2 seems to be more appropriate due to correlation and time dependency (see section 3.9 and Annex B).

3.4 Background of partial factors for permanent and variable loads

3.4.1 General introduction

The objective of this chapter is to provide further background for the recommended partial load factors in Annex A of EN 1990:2023. According to the generally accepted basic principles of the Eurocode, the partial factors for actions in ULS-verifications are material and structure independent, at least within a wide category of structures like buildings, bridges and so on. As a result of that principle, one has to accept that the factors cannot serve all design situations in an equally adequate way. Instead, they may for instance be calibrated as a weighted average over a set of design equations that represent the field of their application. This is the consequence of the chosen level of simplification and generalization implied by the partial factor format of the Eurocode. The aim is to avoid unfavourable deviations in design situations that either occur very frequently or can have a major impact. On the other hand, one should be careful not to create very large deviations, even for a small niche of applications. For that reason, results for individual cases count and should be checked; sensitivity studies are essential.

It should also be kept in mind that material related Eurocodes EN 1992-EN 1999 can choose material resistance partial factors for a specific material (or even a subcategory of structures) in order to reach the chosen target reliability (e.g. 3,8 for 50 year) given overall and fixed values for the partial load factors in EN 1990. At that stage it is also possible to give tailor made attention to nonlinear effects.

Notes:

- A differentiation of the limit state equations for all the resistance models and all materials is not possible within the scope for this report. Such a refinement, however, is also believed to be less relevant for the goal of this sub-chapter that is a global reflection on the recommended partial action factors. See also the general note above.
- The methods and elaborations presented in this sub-chapter as well as in Annex B should be considered as example applications of EN 1990:2023 Annex C clause C.4.4; possible considerations outside this reliability based approach will be mentioned where appropriate (see also Figure 3.1).
- In this subchapter the same steps are followed as in the simple example 2.4, but probabilistic models, characteristic values and partial factors are here in accordance with the Eurocode approach.

3.4.2 Portfolio, Limit state function and Weights

The limit state considered in this sub-chapter 3.4 is for buildings according to the column DC1 in EN 1990:2023 Table A1.8. We will primarily consider:

- 6 different materials;
- 1-2 limit state models per material;
- 1 self-weight and 1 permanent load;
- 3 different variable loads, but no combinations of variable loads;
- STR, permanent design situation;
- a reference period (and design service time) of 50 years;
- a target reliability of 3,8;

In addition there are some calculations to study sensitivities.

3.4.2.1 The limit state function

The load effects are linear and unfavourable; no details of the resistance model are included. For most materials a simply supported beam is considered, starting from the following limit state function:

$$g(\mathbf{X}) = Z f - (1/8) [g + q] L^2 \quad (3.13a)$$

where:

$g(\mathbf{X})$ limit state function;

$\mathbf{X}$ vector of random variables;

f material strength [kN/m²];

g self-weight (including permanent load) [kN/m];

q imposed load [kN/m];

Z effective section modulus [m³];

L span [m].

After dividing by $(1/8)(g_m+q_m)L^2$, introducing $Q=q/q_m$, $G=g/g_m$ and $R = f/f_m$ and adding the model uncertainties for the resistance (θ_R), the load calculation (θ_e) and the variable loads (θ_Q) we arrive at:

$$g(\mathbf{X}) = \theta_R p R - \theta_e [(1-a_Q) G + a_Q \theta_Q Q]$$

where $a_Q = q_m / (q_m + g_m)$ and $p = Z f_m / (1/8)(g_m+q_m)L^2$ is a deterministic standardized resistance. Index m indicates the mean value. Next we split up the weight term into a self-weight part and a permanent load:

$$G = a_G G_S + (1 - a_G) G_P \text{ with } a_G = G_{Sm} / (G_{Sm} + G_{Pm})$$

So finally a generic standardised linear limit state function is obtained as:

$$g(\mathbf{X}, p_{ij}) = p_{ij} \theta_{R,i} R_i - \theta_e [(1-a_Q) \{a_G G_{S,U} + (1-a_G) G_{P,U}\} + a_Q \theta_{Q,j} Q_j] \quad (3.12b)$$

R , G_S , G_P and Q represent the standardised (mean 1,0) random variables representing resistance, self-weight, permanent and variable load;

The θ - variables represent the model uncertainties for the resistance, load effect and load model;

p is the resistance factor following from the Eurocode design requirement (see 3.15);

a_Q represents the ratio between variable and permanent loads ($a_Q = 1$ for variable load only);

a_G represents the ratio between permanent load and self-weight $a_G = 1$ for self-weight only);

i and j refer to material and load cases.

Notes:

Geometrical uncertainties are not explicitly taken into account but may be part of the model uncertainty.

In other publications often the parameter $\chi = Q_k/(G_k+Q_k)$ is used. For snow loads the values of χ and a_Q are similar. For wind and office load relation between χ and a_Q (for a 50 year reference period) may be estimated as $\chi = 1,4 a_Q / (1+0,4 a_Q)$.

The load model uncertainty for the self-weight model is neglected.

3.4.2.2 The design equation

Given the span, the characteristic actions, the characteristic resistance and the partial factors we may find the appropriate dimensions of the structure (actually the Z -parameter) from:

$$Z f_k / \gamma_R = (1/8) [\gamma_G g_k + \gamma_Q q_k] L^2 \quad (3.13)$$

where $\gamma_R = \gamma_{Rd} \gamma_m$ (covering the resistance model and the material) and γ_G and γ_Q are equal to the corresponding products $\gamma_{Sd} \gamma_f$ (covering the model effect calculation and the combination of action model and action itself). In the standardised representation this is modified into:

$$p R_k / \gamma_R = \gamma_G (1-a_Q) G_k + \gamma_Q a_Q Q_k \quad (3.14)$$

After separating the self-weight part and permanent load:

$$p R_k / \gamma_R = (1-a_Q) [\gamma_G \{a_G G_{Sk} + (1-a_G) G_{Pk}\} + \gamma_Q a_Q Q_k] \quad (3.15)$$

Using this equation one finds the parameter p in equation (3.12b). The factors γ_R on the resistance side will be chosen in accordance with present material related Eurocodes, see Table 3.2. The partial factors on the loading side (to be investigated in this sub-chapter) are $\gamma_G=1,35$ and $\gamma_Q=1,5$ in accordance with EN1990:2023, Annex A.. The procedure is performed for a range of values for a_G and a_Q , each value receiving equal weight in this sub-chapter. Table 3.3 gives information over the applied ranges and the weights w (see section 2.6.2).. These values are best guesses intended to give more weight to what is believed the most frequent or important applications.

It should be noted that in the design equation no model uncertainties are explicitly considered; implicitly this means that the characteristic values are equal to 1,0 (irrespective of the actual mean and coefficient of variation) and are actually omitted in the design equation above and the input tables.

Table 3.2. Partial factors for the resistance according to related Eurocodes.

i	i	Material and Limit state	$\gamma_R \quad \gamma_M$
1	1a	Structural steel yielding strength (bending)	1,00
2	2a	Concrete compressive strength	1,50
3	2b	Re-bar yield strength (bending)	1,15
4	3a	Glulam timber (bending)	1,25
5	4a	Solid timber (bending)	1,30
6	5	Masonry compression strength	1,50
7	6a	Aluminium bending	1,10
8	7a	Ground pile foundation	1,50
9	7b	Shallow foundation	1,40

Source: Eurocodes 1992-1999

Table 3.3. Weights and ranges of variations of a_G and a_Q .

i	Material and Limit state	$w_{R,i}$ (weight)	a_Q ranges	a_G ranges
1	Structural steel yielding strength (bending)	17 %	[0,3- 0,8]	[0,6; 1,0]
2	Concrete compressive strength	12 %	[0,1- 0,7]	
3	Re-bar yield strength (bending)	40 %	[0,1- 0,7]	
4	Glulam timber (bending)	3,5 %	[0,2- 0,8]	

5	Solid timber (bending)	1,7 %	[0,2- 0,8]	
6	Masonry compression strength	12 %	[0,1- 0,7]	
7	Aluminium bending	1,7 %	[0,3-0,8]	
8	Ground pile foundation	6%	[0,1-0,5]	
9	Shallow foundation	6%	[0,1-0,5]	
	Variable Load	w_{R,i} (weight)		
1	Imposed load	50%		
2	Snow	10%		
3	Wind	40%		

3.4.3 Probabilistic models for partial factor calibration

Table 3.7 (at the end of this section) shows probabilistic models and corresponding characteristic values for calibration purposes used in this sub-chapter. This set may be considered as a simplified version of Annex A. The models are based on minimum code requirements.

The mean values for resistance and loads in Table 3.7 do not refer to the actual physical mean values (like a mean strength of 120 MPa or a load of 45 kN). The point is that the mean values of the actions in 3.4.2 have been normalized to 1,0. This is meaningful as the absolute values of the loads and resistances are of no importance for the determination of the reliability in this example. Only the ratios are relevant. The mean value of the resistance is also taken as 1,0. For an assumed or given set of partial factors the value of p follows from equation 3.15. Next the reliability index given the required value of p may be found using (3.12b).

As long as the models provided in the codes are unbiased also the mean values of the model uncertainties are equal to 1,0. Consequently mean values unequal to 1,0 show that the model has a bias. The coefficients of variation in Table 3.7 follow from Annex A.

The columns with characteristic values show the values that are actually used in the design or assessment calculations.

Further detailed information is given below of each category.

3.4.3.1 Materials

According to EN 1990:2023 clause 6.2 characteristic values for materials are (unless specified otherwise) defined as the 5% fractiles of the underlying probability distribution function for materials. So, the typical situation in Table 3.7 would be:

Table 3.4. Typical statistical information for materials in this sub-chapter

Random variable	X	Distribution	Mean	V	X _k (fractile)
Material A	R ₄	Lognormal	1,00	0,10	(0,05)

In this case the characteristic value in design equation 3.15 follows (as a good approximation, the exact expression can be found in EN1990:2023, Annex C.4.4.2) from:

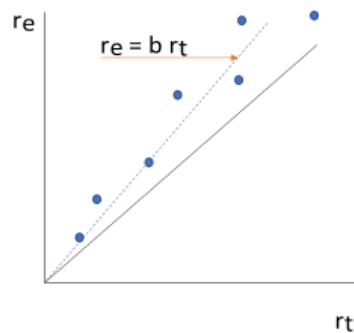
$$X_k = \mu \exp(-1,65V) = 0,83 \quad (3.16)$$

A deviating fractile in the column of X_k requires some formal background, that is, the number should be specified in codes or material product standards. In Table 3.7 this is (making reference to Annex A) the case for steel and aluminium.

3.4.3.2 Model uncertainties resistance side.

Model uncertainties for resistance models are often based on comparison between a theoretical model and experiments where all other variables in the model (yield stress, geometry etc.) have been measured. According to EN 1990:2023 Annex D the experiments lead to a scatter V and a bias b . A positive bias indicates that the model is on the safe side (e.g. lower bound model or models implicitly taking care of the scatter) (see Figure 3.2).

Figure 3.2. Definition of Bias according to EN1990:2023 Annex D



The options that could have been followed for the bias in the various material related Eurocodes after the tests are:

- (1) No change to the model and no change in the partial factor, i.e. $\gamma_{Rd} = 1,0$
- (2) The model was left unchanged and the bias was taken into the partial factor γ_{Rd}
- (3) The model was changed by a factor b making it unbiased.

The consequences for mean and partial factor are shown in Table 3.5:

Table 3.5. Options for treating the bias of the model uncertainty

Option	$\mu(\theta)$	V	γ_{Rd} (bias part only)
(1) No changes	b	V	1
(2) bias in γ_{Rd}	b	V	$1/b$
(3) bias in model	1	V	1

Note that, as mentioned before, the model uncertainty in the design equations 3.15 (just like in the Eurocodes) is not considered explicitly and so implicitly the characteristic value is equal to 1,0, as also stated in Table 3.7).

Options (2) and (3) in the Table 3.5 lead to the same results, it is only a different manner of presentation. Option 2 is in line with a r_d as defined in EN 1990:2023 Annex D eq (D28). Option (3) uses the test results as a correction tool to obtain an unbiased model. The values $\mu(\theta)$ and γ_{Rd} (bias part only) are changed accordingly.

Option (1) is used when the model and the partial factor equal 1,0 already have been fixed and are used in the design stage of the calibration. The tests and the model uncertainty parameters are used only to find the corresponding reliability index for a given design. The design will be different from options 2 and 3, but in the end this will be compensated by different values for the partial load factors in order to reach the same target beta.

So finally the same target reliability level is being reached in all three cases: the option type has influence on the intermediate results, but not on the result of the calibration. One should only make sure that in the calibration procedure the same definitions are followed as in the material related Eurocodes. In particular one

has to be careful if a Eurocode makes adjustments in the model in order to get “nice values” like 1.0 for the partial factors.

For the scatter part of the model uncertainty (so $b=1$) only two options are used (see Table 3.6):

- (1) No change to the model and no change in the partial factor, i.e. $\gamma_{Rd} = 1,0$.
- (2) The scatter was accounted for via EN 1990:2023, equations (D.21) for r_k and (D.28) for r_d . where the expression for r_k refers to the 5% fractile of the product $\theta_R R$ and a similar statement holds for r_d .

Table 3.6. Options for treating the scatter part of the model uncertainty (lognormal distribution)

Option	Mean	V	γ_{Rd}
(1) no change	1	V	1
(2) bias in model (r_k and r_d)	$\exp(\alpha_\theta \beta V_\theta)$	V	r_k / r_d

where α_θ may follow from Annex D. Also here the characteristic values implicitly are taken as 1,0. Note that V of course follows from the tests and is not changed.

In Table 3.7 the mean values of the model uncertainty on the resistance are equal to 1 or b. For the calculations in this report it is assumed that γ_{Rd} is included in the γ_R or γ_M values presented in Table 3.2; for that reason the γ_{Rd} values are not present in Table 3.7.

Note finally that we should carefully consider how the bias and scatter in the model uncertainty is obtained. According to Annex D all random variables present in the model should be measured carefully. In practice, however, sometimes nominal values are used. That way the bias may be incorrect and a part of the scatter in resistance parameters may enter the coefficient of the variation of the model. In order to remove the double counting of uncertainties one should base γ_m on the difference between the scatter in the laboratory and the real world only. There may also be measurement errors disturbing the procedure. These can be accounted for if the standard deviation of the error is known.

3.4.3.3 Model uncertainty in load effect calculations

According to Annex A the model uncertainty for the load effect calculation depends on the type of structure and type of load effect. For statically determinate structures the uncertainty is smaller (say $V = 0,05$) than for statically indeterminate structures (say $V = 0,10 - 0,20$). In this subchapter the value of 0,10 has been chosen, knowing that higher values are possible. But, in particular for (ductile) indeterminate structures there usually is a redistribution of stresses possible that is not taken into account. Although the Eurocode in principle is limited to member level design (and not the system level) it can be argued that a value $V=0,10$ is high enough. Note further that no characteristic value is specified: the uncertainty is believed to be covered by the partial load factors and the characteristic value is not present in equation 3.15.

3.4.3.4 Self-weight and permanent loads

For self-weight in general, the 50% fractiles are taken as representative values as in agreement with EN 1990:2023. The scatter in the table includes the variation in volume, specific weight and the model uncertainty in the self-weight model. Characteristic values for permanent loads are mean values (or 50% fractiles) for $V=0,1$ and 95% fractiles for $V=0,2$.

3.4.3.5 Climatic actions

A time variant load is usually the product of a time variant part and a time invariant part. According to clause 6.1.2.3 of EN 1990:2023 the characteristic value for the time variant part is chosen as the 98% fractile of the annual maximum. The characteristic value for the time invariant part is tuned in such a way that for the product of the two characteristic values the exceedance probability for a one-year period is 2%. As a simple upper bound value $\theta_k = \mu_\theta (\exp V_\theta)$ can be used, however, calculating the exact value is to be preferred.

In this model no attention is paid to climate change.

3.4.3.6 Imposed load (office building)

For imposed loads actually no specific statement is present in EN 1990:2023 for the fractile corresponding to the characteristic value. In this study we take a slightly modified JCSS model as starting point (see Annex A) which leads to mean of 2,22 kN/m² and a V= 0,26 for the 50-year maximum. According to EN 1991 the representative value is 3,0 kN/m². The quotient 3,0 / 2,22 =1,35 explains the characteristic value in the standardized load model in Table 3.7.

Table 3.7. Stochastic models for partial factor calibration (based on Annex A).

Random variable	X	Distr, type	Mean (μ)	V	Char value (Fractile)	
Resistance model uncertainty						
Steel bending	θ_R	Logn.	1,15	0,05	1,00	Averaged over several applications
Concrete compression	θ_R	Logn.	0,97	0,14	1,00	All variability not in concrete lab strength
Concrete bending	θ_R	Logn.	1,03	0,07	1,00	
Glulam bending	θ_R	Logn.	1,00	0,10	1,00	
Solid timber bending	θ_R	Logn.	1,00	0,10	1,00	
Masonry (flexure)	θ_R	Logn.	1,16	0,18	1,00	
Aluminium bending	θ_R	Logn.	1,27	0,15	1,00	
Pile foundation	θ_R	Logn.	1,00	0,20	1,00	
Shallow foundation/slope instab	θ_R	Logn.	1,00	0,15	1,00	
Material strenght						
Steel yielding strength	R ₁	Logn.	1,00	0,05	0,83	
Concrete compr strength	R ₂	Logn.	1,00	0,10	(0,05)	Not including lab – in situ variation.
Rebar yielding strength	R ₃	Logn.	1,00	0,045	(0,05)	
Glulam bending strength	R ₄	Logn.	1,00	0,15	(0,05)	
Solid timber bending strength	R ₅	Logn.	1,00	0,20	(0,05)	
Masonry compr. Strength	R ₆	Logn.	1,00	0,16	(0,05)	
Aluminium 0,2% limit	R ₇	Logn.	1,00	0,05	0,85	
Cone penetration test value	R ₉	Logn.	1,00	0,12	(0,05)	
Undrainred shear strength	R ₁₀	Logn.	1,00	0,20	(0,05)	
Model Unc Load effect calc						
statically indeterminate frames	θ_{cl}	Logn.	1,00	0,10	none	
Self-weight						
Self-weight (steel)	G _{s,1}	Normal	1,00	0,025	1,00	Load model uncertainty included
Self-weight (reinforced concrete)	G _{s,2}	Normal	1,00	0,05	0,98	
Self-weight (glulam)	G _{s,4}	Normal	1,00	0,10	0,95	
Self-weight (solid timber)	G _{s,5}	Normal	1,00	0,10	0,95	
Self-weight (masonry)	G _{s,6}	Normal	1,00	0,07	1,00	
Self-weight (auminium)	G _{s,7}	Normal	1,00	0,04	1,00	For Aluminium: see Annex A
Self-weight (ground)	G _{s,8}	Normal	1,00	0,05	1,00	
Permanent load						
Permanent load (small V)	G _P	Normal	1,00	0,10	1,00	
Permanent load (large V)	G _P *	Normal	1,00	0,20	1,33	
Variable loads						
Wind time-invariant part	θ_{Q1}	Logn.	0,97	0,26	1,08	Extreme for period of 50 a
Wind velocity pressure	Q ₁	Gumbel	1,00	0,14		
Snow time-invariant part	θ_{Q2}	Logn.	0,81	0,26	0,82	Extreme for period of 50 a
Snow load on ground	Q ₁	Gumbel	1,00	0,20		
Imposed load model	θ_{Q2}	Logn.	1,00	0,10	1,35	Extreme for period of 50 a
Imposed load (office)	Q ₃	Gumbel	1,00	0,26		

Note: Mean values of the strength and load variables are all equal to 1.0 as a consequence of the standardization of the limit state function. Only annual action maxima are different from 1.00 to keep them consistent with the distribution of the 50 year maximum. No values are presented for the characteristic values of the model uncertainties as they are not explicitly present in the design equations. Mean values of the model uncertainties different from 1.00 account for bias in the models.

3.4.4 Resulting reliabilities for design according to the Eurocodes

The resulting reliability indices corresponding to EN 1990:2023 format (8.10) are presented in Table 3.8. For the design all partial factors are in line with the Eurocodes: the values for the actions are 1,35 for permanent and 1,5 for variable loads, the values for the resistance side are according to Table 3.2.

The results in Table 3.8 are presented for all considered combinations of the materials and variable actions. Each value is averaged over the considered range of the a_Q parameter, which is the ratio between the variable and total load (see Table 3.3). The overall average result can be characterized by $\beta = 3,74$. In order to check the sensitivity to the choice of the weight values the analysis has been repeated for equal weights. In that case the result was $\beta = 3,65$.

Table 3.8. Resulting 50 year reliability indices for each combination of material and action (averaged over all a_Q and a_G values)

	Wind	Snow	Imposed
Steel bending	3,52	3,17	3,94
Concrete compression	3,84	3,62	4,15
Concrete bending	3,42	3,15	3,78
Glulam bending	3,34	3,08	3,71
Solid timber bending	3,41	3,17	3,74
Masonry	4,10	3,92	4,37
Aluminium bending	3,61	3,31	4,00
Pile foundation	3,55	3,42	3,79
Shallow foundation	3,73	3,59	3,98

Figure 3.3 shows graphically the scatter in beta-values for the various loads (first 3 graphs, including all materials) and materials (last 9 graphs, including all three loads). Figure 3.4 shows the dependence of the reliability index on the a_Q parameter for snow. Each line gives the result for one material. The results for wind snow and live load show quite similar trends.

To some extent the relatively small scatter for foundations may be surprising, given the large uncertainty in resistance. But actually that fact makes the result relatively less sensitive to the scatter in the climatic loads as for instance compared to steel. Note also that the partial factors for foundations are large and the range of a_Q is rather narrow.

Figure 3.3. Results of reliability indices for 50 years, for each loading type and each material; calculations based on design format prEN 1990:2023 (8.12) using partial factors according to the Eurocodes.

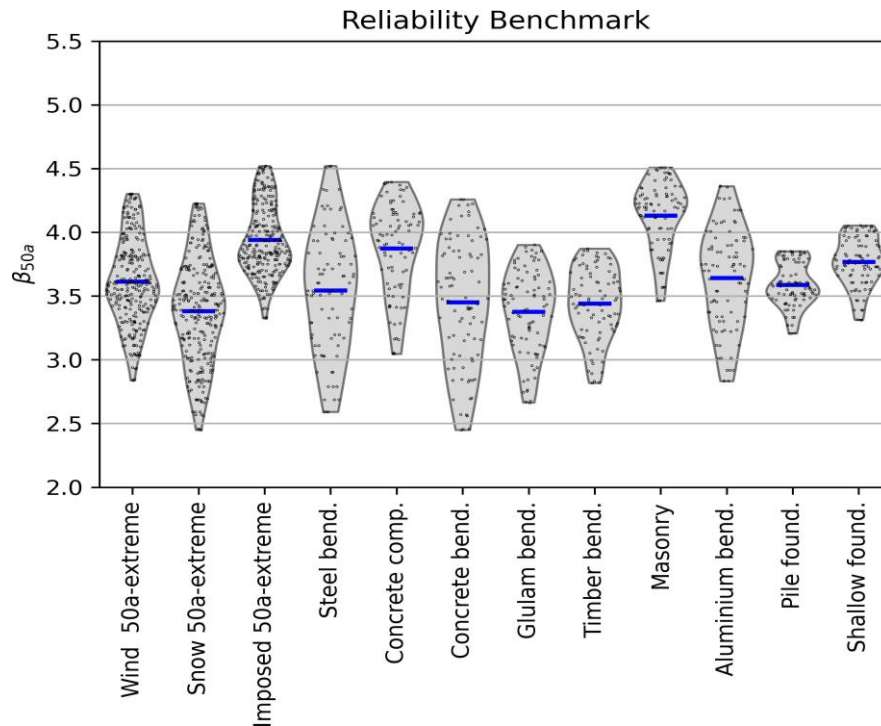
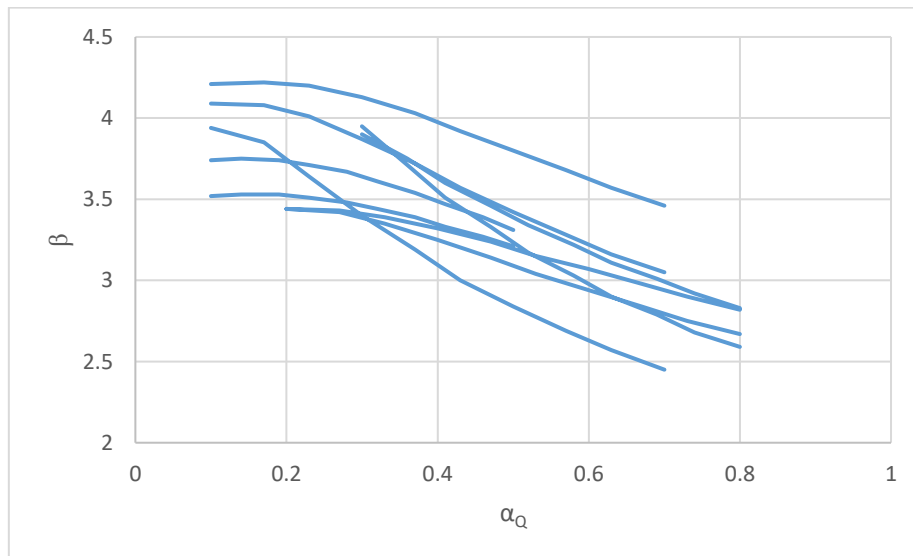


Figure 3.4 Reliability index β as function of α_Q for snow load, each line representing a specific material (steel concrete. etc).



3.4.5 Reflection

Given the result of 3.4.4 the main conclusion is that on average a design according to the rules of the Eurocode (including the values 1,35 and 1,50 for the partial factors for permanent and variable actions) meets the target reliability level $\beta = 3,8$ for a design life time of 50 years with sufficient accuracy. Of course, we should keep in mind that the above analysis is limited in its set up and contains a number of quite arbitrary choices. Nevertheless, it may be considered as a reasonable indication.

At the same time, we may also conclude from Table 3.8 and Figure 3.4 that the scatter in calculated reliability levels is large. Numbers run roughly from 2,5 to 4,5, the larger values corresponding to weight dominated design situations and the smaller reliability levels are obtained for design situations dominated by climatic

actions. The background reason for this scatter has partly to do with the historical development of the present partial factors for permanent and variable loads. The familiar values of $\gamma_G = 1,35$ and $\gamma_Q = 1,50$ were not derived on the basis of reliability theory only but on a mix of theoretical considerations and traditional values. Also, for reasons outside the domain of the theory of structural reliability the values were considered as a good and acceptable compromise.

For additional information and reflection on this issue, reference is made to Annex B where also results from some sensitivity studies are shown.

3.5 Further issues related to action aspects

3.5.1 Load combination factors

The load combination factors ψ_0 and ξ are intended to reduce the non-dominant or non-leading actions in a load combination (see section 3.2). Having all actions in the load combination with their unreduced design value would be over conservative if the actions are statistically independent. The combination values can be included in a standard calibration procedure, but this can make the procedure more extensive and is often omitted.

An approximation is to assume that the maxima in time of all variable loads occur at the same point in time. This leads to upper bound values for ψ_0 values (although still less than 1,0). The error in case of a 1-year reference period is less than for the 50-year reference period.

It is worth to point out that the ψ_0 values for (8.12) are theoretically different from the ψ_0 values for (8.13). The reason is that in (8.12) the leading variable action has no reduction factor and so only all other factors may profit from the fact that maxima do not occur at the same point in time. In (8.13a), however, also the leading variable load is multiplied with ψ_0 . The reduction results from the combination with the permanent load and in that case the maxima always occur at the same point in time by definition. Note further that in (8.13b) the ξ factor actually is also a kind of ψ_0 -values but with a different name.

If we neglect the time variability, the design value for the accompanying action $Q_{dA} = \gamma_{Qj} \psi_{0j} Q_{kj}$ follows from (see section 3.1.3):

$$F(Q_{dA}) = \Phi(0,4 \times 0,7 \beta) = \Phi(0,28\beta) \quad (3.17a)$$

However, if we want to include the time factor we have to apply a more complex formula using the FBC model (Ferry Borges and Castanheta M. 1971):

$$F(Q_{dA}) = \{\Phi(-0,4 \Phi^{-1}(\Phi(-0,7\beta)/n))\}^n \quad (3.17b)$$

Here n is the number of intervals in the reference period for the slowest fluctuating load in the combination. This may lead to the unpleasant but correct result that the ψ_0 - for snow is different if combined with wind or combined with live load. Taking the highest value is a practical solution.

See also example B3 in Annex B.

3.5.2 Free and fixed actions, single source rule

According to clause (6.1.1) of EN 1990:2023 actions can be classified as free or fixed. The distribution of a fixed action is unambiguously defined for the whole structure or structural member. For free actions various spatial distributions over the structure are possible and should be accounted for. In some cases the analysis should be based on the most unfavourable combination of loaded and unloaded areas. This holds for instance for imposed loads in buildings (including partition walls) and traffic loads on bridges. In other cases (like snow and wind) the situation is mixed and a set of prescribed fixed load arrangements have to be considered. Construction loads (cranes, auxiliary construction loads) can be considered as fixed or free depending on their use.

Special attention in this respect is needed for permanent loads. In order to avoid cumbersome calculations for the free component of permanent loads in practice, EN 1990:2023 introduces in 6.1.1 the notion of a ‘single source’ simplification. We will discuss here the rule for the structural part of the permanent load..

The structural self-weight is a random field with in general relatively well known means and coefficients of variations. The difficulty is in the correlation pattern, as there is relatively little data. Parts coming from one source (one material, one manufacture) are believed to be highly correlated. According to the JCSS model code the correlation within one member is almost complete and for the correlation between members in one building a value of 0,7 is recommended. This is probably meant as an average for different situations, but that is not mentioned explicitly.

The simplifying one source (full correlation) assumption is on the conservative side as long as all parts contributing to a certain load effect are all favourable or all unfavourable. If parts of the permanent load are unfavourable and other parts are favourable, the one source assumption is no longer conservative. We may arrive at the famous problem of subtracting two numbers leading to an almost zero load effect. Using the one source assumption we may arrive at a design load effect of zero. See also section 3.5.3.

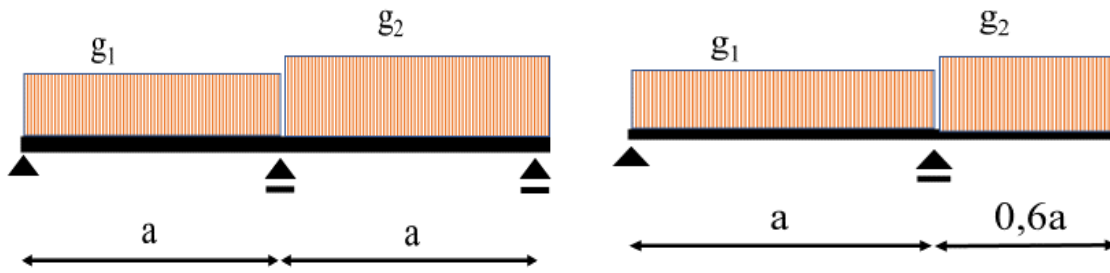
We should however also keep in mind that the partial factor γ_G takes care of the uncertainty in the load itself as well as in the load effect calculation uncertainty θ . The statistical correlation for the weight has to do with method of fabrication. Clearly in cases where the “single source” simplification is applicable (same material, same manufacture, same time of fabrication) the dependency will be close to 1,0 in most cases. For “two sources” it will be closer to 0,0. The uncertainty in θ has nothing to do with the source of the material. In the draft update for the model uncertainty chapter of the JCSS model code the following general statement can be found:

In literature, until now, little attention has been given to correlation patterns in model uncertainty. As a general indicator one may say that uncertainties related to influences or parameters neglected in the models may leave room to independence between various parts of the structure and between structures when the neglect leads to unbiased predictions over the structure. If we neglect for instance temperature effects in a statically indeterminate structure, this will lead to tensile stresses in some parts and compression in other ones. In case of a large number of members, the correlation between two arbitrary members is judged to be small. However, if there are only two members, they may have a strong (negative) correlation. Simplifications on the other hand tend to correlation over the entire population and will partly be observed as a bias in case of testing.

An important group of random variables neglected in the structural analysis are the effects due to deformation constraints in statically indeterminate structures as may follow from temperature variations, humidity variations, settlements and stresses due to imperfect mounting. It is well known that for ductile structures this is not very important, but for brittle elements this can be quite different. Also local errors in the stress calculation model, normally not relevant, can become very important in case of brittle behaviour. Their determination can be more complicated as for ductile behaviour as structural behaviour of brittle structures can show discontinuities.

In general the model uncertainties may be considered as dominant with respect to the weight (see e.g. Annex A). So even in a case when there is clearly only one source in the origin of the structural weight, there still may be a non-full-correlation, assuming that the model uncertainties are not fully correlated between different elements. Let us consider two specific cases:

Figure 3.5 Considered cases for the single source discussion.



Case 1

Given two floors on a continuous beam as in Figure 3.5 left. The floors (neglecting the beams) have self-weights g_1 and g_2 with equal mean values. The bending moment in the centre of the left beam is equal to (including the model uncertainty factors θ):

$$M = (5/48) \theta_1 g_1 a^2 - (1/48) \theta_2 g_2 a^2$$

Note that this may be slightly different from the maximum moment, but that is considered as less important here. If considered as *one source* we have for the design load effect M (using (EN 1990:2023 equation 8.12):

$$M_d = (4/48) \gamma_G \mu(g) a^2 = (4/48) 1,35 \gamma_G \mu(g) a^2 = 5,4/48 \mu(g) a^2$$

If we consider the loads as *two (independent)* sources we have to distinguish between the favourable and the unfavourable contributions:

$$M_d = (5/48) \gamma_G \mu(g) a^2 - (1/48) \gamma_{G_{fav}} \mu(g) a^2 = (5 \times 1,35 - 1 \times 1,0) / 48 \mu(g) a^2 = 5,75/48 \mu(g) a^2$$

Given a mean for M of $4/48 \mu(g) a^2$ (assuming the mean value being equal to the characteristic values) the difference is considered as too small to bother, so we may still consider it as one source even if it is not.

Case 2

For the case of Figure 3.5 right, we find for the design load effect M_d in the left beam (using (8.12) and the *single source* rule):

$$M_d = (1/8) \gamma_G \mu(g) a^2 - (1/4) \gamma_G \mu(g) (0,6a)^2 = 0,047 \mu(g) a^2$$

If we consider the terms as coming from *two (independent)* sources we have:

$$M_d = (1/8) \gamma_G \mu(g) a^2 - (1/4) \gamma_{G_{fav}} \mu(g) (0,6a)^2 = 0,079 \mu(g) a^2$$

Note that the mean value of M is equal to $(1/8) \mu(g) a^2 - (1/4) \mu(g) (0,6a)^2 = 0,035 \mu(g) a^2$. This means that the difference between the two design values is large. This is a typical case where the sensitivity is large. In order to get more insight, let us redo the calculation for the second design load value of the bending moment, but using reliability theory. If we define the limit state function and statistical data as:

$$g(X) = 0,067 - \{ 0,125 \theta_1 g_1 - 0,09 \theta_2 g_2 \}$$

$$\mu(g_1) = \mu(g_2) = 1,0, V(g_1) = 0,05, V(\theta_1) = 0,10, \rho(g_1, g_2) = 0,7, \rho(\theta_1, \theta_2) = 0,5$$

we find that $\beta_5 = 3,8 \times 0,7 = 2,66$, in line with the required reliability. So, it follows that $M_d = 0,067 \mu(g) a^2$ which still is high compared to the “one source” approach.

The difference reduces if we add imposed load: taking $q_k = 0,3 g_k$ the error of the one source approximation becomes in the order of 20 % if compared to the probabilistically derived value. This is still quite substantial and we might even think of worse examples. EN 1990:2023 Clause 8.3.3.1(3) states that the single source rule may be applied provided that the design is not sensitive to spatial variation of those permanent actions. The basic question is whether the designer will recognize this criterion in practice or not. The safe way out is of course by making both assumptions and accept the most conservative result.

Finally, a remark is made about the self-weight from building elements other than structural. These loads can either be time independent or change a number of times during the working life of the structure. They can be fixed or quite random in space. As a consequence, no large correlation can be assumed and the single source

rule is not recommended. The variability is usually not large, so the partial factor for self-weight seems to be sufficient. If cases where the variability is large the 95% fractile is used. Detailed rules are left to the National Annex.

3.5.3 The limit state of static equilibrium

Closely related to the single source problem is the limit state of static equilibrium. Actually, it is the extreme example where the single source rule is not applied in order to ensure the safety of structures with a high sensitivity to an out of balance between action effects arising from permanent actions from in most cases a single source. Also, the factors in EN 1990:2023 Table A1.8 need calibration. The specific point is that in the verification procedure no resistance term is present. For instance, the limit state function could be:

$$g(X) = \frac{1}{4} \theta_1 g_1 a^2 - \frac{1}{4} z a^2 \theta_2 g_2 \quad (3.18)$$

The first term represents the de-stabilizing part and the second term stabilizing part. The factor z is chosen such that reliability index is equal to the CC2 target reliability value of 3,8. Let us take:

$$V(g_i) = 0,05, V(\theta_i) = 0,05, \rho(g_i, g_j) = 0,7 \text{ and } \rho(\theta_i, \theta_j) = 0,5 \quad (3.19)$$

The difference with the statistics used in the single source example is that the coefficient of variation for the model uncertainty has been reduced from of the $V(\theta_i) = 0,10$ to 0,05. The background is that most calculation modes in static equilibrium verifications will be less complex than in structural load effect calculations. Using these numbers, it follows that (for $z=0,79$ and $\beta=3,8$):

$$\theta_{1d} g_{1d} / \theta_{2d} g_{2d} = 0,86 / 1,09 = 0,78 \quad (3.20)$$

So according to the theory we should take a partial factor 0,86 for the stabilizing part and 1,09 for the de-stabilizing part of the weight in order to achieve the required reliability level. In EN 1990:2023 Table A1.8 the values for the partial factors are 1,15 and 1,35 leading to a ratio $\rho = 0,85$ instead of 0,78. Looking from the theory this ratio is slightly on the unsafe side.. It is also quite against the spirit of the partial factor approach to increase a load that acts favourable.

Note 1: For the case of static equilibrium EN 1990:2023 does not specify k_F -factors for $\gamma_{G, stb}$, making the margin with $\gamma_{G, destb}$ very small for CC1.

Note 2: See also example B4 in Annex B which is discussed in more detail.

3.5.4 Prestressing

In the context of this note prestressing is conceived as an active measure to influence the stress distribution in the structure by prestressed cables (tendons), bars or other elements. The purpose of prestressing could be to avoid certain limit states (less deflection, cracking) or mechanisms to occur. In particular one might think of:

- Internal prestressing in girders (bonded and unbonded tendons).
- Cables supporting towers and masts
- Cable stayed bridges (or buildings)
- Extradosed bridges (or buildings)
- Positioning of supports (e.g. by jacking);
- Tension grid (membrane) structures

Note that already the order of the constructive adding/mounting building elements have influence on the internal stress distribution, but that is not discussed in this note.

A general aspect of prestressing is that the prestressing action is not important for the Ultimate Limit state capacity as long as the behaviour is geometrically linear and full ductility is present. If not, or if other criteria are important (deflection, cracking) limit state functions have to be formulated. In most cases the force in the prestressing element will appear as a design value (see EN 1990:2023 eq (8.9):

$$P_d = \gamma_p P_k \quad (3.21)$$

P_k is the characteristic value of the prestressing force (usually a single value) and γ_p is the partial factor according to EN 1990:2023 Annex A or as given in the relevant Eurocodes. General recommendations, assuming no (cor)relation with self-weight, are: $\gamma_{p\text{inf}} = 0,9$ and $\gamma_{p\text{sup}} = 1,1$ for the standard favourable and unfavourable effects. For internal prestressing in concrete beams the value 1.0 is taken; for local effects (crunching) and stability limit states the values are 1,2 and 1,3. For membrane systems we have: $\gamma_{p\text{sup}} = 1,25$.

For possible (cor)relations with self-weight, see section on cable stayed bridge.

Some codes allow factors closer to 1,0 if there is sufficient control on the action force. It should be mentioned that a reliability-based background for partial factors on prestressing is lacking. Most factors are based on experience and direct expert judgement only.

As in many cases the prestressing action is actually the result of a deformation type of action, it could be more reasonable to let the partial factor act on the deformation. However, that is usually not done.

3.5.4.1 Internal prestressing in concrete beams

In case of concrete elements with internal *bonded* prestressing tendons the reaction force of the prestressing action is found in the compressive stress in the concrete. The main purpose is to avoid early cracking or to reduce deformations. The upper value of the prestressing force is well controlled (so low uncertainty) and

$$\gamma_p = 1,0 \quad (3.22)$$

seems therefore to be appropriate. The lower bound value is more uncertain as it depends on stiffness ratios of steel and concrete, relaxation and creep. This can influence serviceability limit states related requirements but does not have a significant effect on ultimate limit states. EN 1990:2023 allows National Annexes to introduce correction factors r_{sub} and r_{inf} where relevant. In the case of *unbonded* prestressing systems the concrete element is ready first (post-tensioned) and the prestressing force can even be calibrated better assuming the friction forces are small. Long term loss effects may be compensated in the course of time if wanted and physically possible.

As far as the ULS is concerned one might even neglect the pretension and consider the tendons as ordinary reinforcement to understand the sensitivity to prestressing force, assuming that sufficient deformation capacity (also for concrete crushing) is available. The contribution is different in case of bonded or unbonded tendons because the unbonded tendons are able to develop only a limited increase of elongation at ULS. This may also influence the stresses at the anchorage zone

3.5.4.2 External prestressing

In the case of external prestressing, we may distinguish between cable stayed structures and guyed towers and masts.

- Cable stay structures,

The main goal of the prestressing is to control the actual deflections. The reacting forces come partly from the self-weight of the bridge deck and partly from the stiffness of the deck structure. At the other side we may have pylons and or anchors. For extremely flexible systems of deck structure and pylon there is only the weight and actually there is no real prestressing any more. For those cases the load combination:

$$\gamma_G G_k + \gamma_P P_k = 1,35 G_k + 1,00 P_k \quad (3.23)$$

is over conservative.

The point is that if the self-weight in a practical case is higher than the predicted value during design, the prestressing force will also be higher as it will be tuned to the combined effect for instance by controlling a deflection at a key location. However, taking γ_P exactly equal to γ_G might be too optimistic due to uncertain short- and long-term changes in stiffness properties; also model uncertainties and possible measurement errors may disturb the accuracy of the calculation. Note that partial load factors also keep track of the uncertainties in the calculation model. The factors should actually be calibrated to a reliability analysis, taking into account all these uncertainties. In that case one could make use of the fact that the uncertainty in the actual weight is very small and only the uncertainty in the load effects is relevant. Further, the actual uncertainty in the prestressing control system (e.g. based on measurements of deformations or displacements) could be taken into account. A *possible* outcome could be

$$\text{self-weight unfavourable} \quad 1,35 G_k + 1,30 P_k \text{ and } 1,35 G_k + 1,40 P_k \quad (3.24a)$$

$$\text{self-weight favourable} \quad 1,00 G_k + 0,95 P_k \text{ and } 1,00 G_k + 1,05 P_k \quad (3.24b)$$

However, this is not used in the present Eurocode. In EN 1993-1-11, for the case where the prestressing force and the self-weight are carefully tuned, the above set 3.24a and 3.24b is simplified to:

$$1,35 G_k + 1,35 P_k \quad (3.24c)$$

$$1,00 G_k + 1,00 P_k \quad (3.24d)$$

In particular for locations where the load effects of weight and prestressing exactly or almost exactly cancel each other, the differences might be noticeable. Again, if sufficient deformation capacity is available, the ULS reliability will hardly be affected.

In deriving the partial factors we should bear in mind that also the strength of the prestressing elements themselves is limited. Depending on the properties of the prestressing elements a value of $\gamma_P = 1,40$ (or even already 1,35) may already be unrealistically high and exceeding the actual resistance.

- Guyed towers and masts

Towers and masts need to resist wind loads and offshore structures even waves (coming from all directions) and tying them by cables is a very efficient way to do so. The prestressing actions in the cables will be counteracted by the prestressing actions in other cables and a compression force in the mast. At the lower end the cables are anchored to a foundation. The purpose of prestressing is to avoid compression in cables and reduction in stiffness of the whole system. Guy ropes in guyed masts are normally relative long and non-linear effect of the ropes are more important. In those cases the permanent load is usually less relevant..

3.5.4.3 Lightweight structures with load reversal

To ensure adequate structural behaviour for all relevant design situations, prestressing is required in structures where load reversal is possible for different combinations of actions or load arrangements. This is particularly the case with lightweight structures. Steel or composite bow-string bridges are considered as an example. Hangers made of prestressing tendons, prestressing bars or even steel profiles are often used for load transfer from the deck to the arch. Vertical or inclined hangers can be provided.

Figure 3.6 Bow-string bridge with inclined, prestressed hangers.



Source: CESMA Ingenieros SL – Madrid; photo: MEGADRONE S.A.C.

The overall structural behaviour depends on numerous parameters, such as the relative stiffness of the deck and the arch, the arrangement and separation of the hangers, as well as their stiffness, among others. Non-symmetrical load arrangements cause the system to bend and can introduce compression forces into the hangers. In order to avoid possible damage to these elements, in particular their anchorages, and changes to the structural system, during the construction process the pre-elongation in the hangers must be adjusted. This adjusting is a very delicate operation since internally the system is statically indeterminate. Generally, it should be realized in several runs with a particular tensioning sequence. However, deviations from the nominal prestressing forces are possible. The limit state is that the internal force in the hangers is negative. Sometimes, a minimum tensile force is assumed as a limit, taking into account possible losses in prestressing forces.

In practice, it is not realistic to check all possible combinations of favourable and unfavourable deviations from the specified prestressing forces in the different hangers by applying the rules for combining actions for persistent and transient design situations (see EN 1990:2023 8.3.4.2). There are also no specific provisions or simplified rules in the various Eurocodes, such as EN 1993-1-11, for this type of prestressing application. However, when verifying Ultimate Limit States, including those related to instabilities of the arch, it should be taken into account that the prestressing forces can be higher or lower than the specified values. The corresponding partial factors γ_p should also cover measurement uncertainties. Considering the tolerances in conventional stressing procedures and the uncertainties associated with converting the applied prestressing forces into their effects, an upper and lower value of, respectively $\gamma_p = 1,1$ and $\gamma_p = 0,9$ seem to be appropriate.

3.5.4.4 Prestressing in membrane structures

Tensile surface structures, being either membrane structures or cable nets (see section 3.6.4), are prestressed systems that can carry loads perpendicular to the surface. The spatial shape is mostly doubly curved. A tensile surface structure can carry loads by activating only tensile stresses. The form can be anticlastic (in general mechanically prestressed like saddle shaped structures) or synclastic (in general pneumatically prestressed by air inflation) or even monoclastic or flat.

Usually, membrane structures are mixed structures. The supporting structure for the membrane can be made of steel, timber or other material. The membrane itself is carrying the external loads to the supporting structure.

For membrane structures the relationship between action and effect is not proportional. Membrane structures require a geometric non-linear analysis.

Figure 3.7 The membrane structure Palais Thermal Bad Wildbad.



© Form TL.

Prestress is applied mechanically by membrane compensation or pneumatically by inner pressure. To ensure the biaxial loadbearing behaviour and to avoid slack areas as well as large deflections, membrane structures are prestressed biaxially. The prestress level is in accordance with the structural analysis. The minimum design prestress should be maintained throughout the design service life.

The recommended lower and upper partial safety factor values for prestress are $\gamma_{\text{Pinf}} = 0,9$ and $\gamma_{\text{Psup}} = 1,25$ respectively (these values are based on expert judgement).

Note: Tolerances in the supporting structure might affect the prestress level, therefore special consideration should be given to the sensitivity of membrane structures to these tolerances. The acceptable tolerances of the supporting structure to the interface to the membrane should be agreed in the design phase.

3.5.4.5 Jacking systems at supports

Prestressing without explicit tension elements like cables etc. may be realized by imposing deformations by jacking at the supports at convenient times during the construction process (Handbuch Brücken, 2014). The counteracting force is delivered by the foundation on one side and the structure on the other. Using this method, one may for instance achieve a compression layer against the cracking that usually appears in continuous composite bridge girders at support (EN 1994-2). One way to deal with this type of prestressing is by treating the jack force as an independent permanent action. As the jack force is relatively well known the partial factor may be lowered. If the jacking process is tuned to some measurable deflection or deformation of the structure the relevant uncertainties should be considered. And like the situation in cable stayed bridges there may be a dependency with the gravity loads.

3.5.5 Accidental actions

In the accidental design situations attention is given to exceptional conditions. The load combination to be considered is (EN1990:2023, clause 8.3.4.3)::

$$E_d = \Sigma G_{kj} + A_d + \Sigma \{ \psi_{1,1} \text{ or } \psi_{2,1} \} Q_{kj} + \Sigma \psi_{2,j} Q_{kj} + (P_k) \quad (3.25)$$

where A_d is the design value for the accidental action which is specified directly. Operational information on A_d is presented in EN 1990:2023 (related to flooding, notional loading on key elements and local damage scenario's like the notional removal of a column) and in EN 1991 (related to fire, impact and explosion). For specific structures more types of accidental actions may need to be taken into account. Typical for accidental actions is that some degree of structural damage is explicitly accepted. EN 1990:2023 Annex E specifies also design strategies related to unforeseen or unforeseeable actions in the form of tying and segmentation.

For water related accidental actions a return period of 1000 year is recommended (see clause 6.13.2); for other accidental actions no recommended fractiles or reliability targets are specified. Guidance may be found in the general principles for setting target reliabilities as discussed in ISO 2394, Annex G. For accidental actions usually

the relative cost of safety measure should be classified as high, leading to lower reliability targets compared with permanent design situations. According to ISO 10252, target values for the annual failure probability in case of accidental design situations are usually set between 10^{-6} and 10^{-4} . For the failure probability given some prescribed damage (e.g. removed column), depending on the circumstances, values between 0,001 and 0,1 are considered as appropriate for a short reference period following the event.

3.5.6 Fire

The fire resistance assessment of Eurocodes is considered as a structural design at Ultimate Limit State in fire situation. In consequence, as it is the case for room temperature design, the fire resistance assessment of Eurocodes comprises several parts related to the reliability issue, such as mechanical and thermal actions, material properties and calculation models. The difference compared to normal room temperature design is that the design actions for fire resistance assessment are composed of both thermal and mechanical actions. Moreover, the former part, i.e. thermal action, occupies a very important place in fire resistance assessment. As a result, thermal effects also play an important role in the response of the structure.

The following paragraphs will provide a summary about the reliability background of the fire parts of current Eurocodes.

3.5.6.1 Mechanical actions

The mechanical actions for fire resistance assessment according to Eurocodes need to be determined with help of a specific combination method taking account of the fact that fire is an accidental situation. Consequently, the design values for various combination coefficients are smaller than those used for room temperature ULS design. According to EN 1990:2023, Clause 8.3.4.3, we have for the accidental fire design situation to take into account, similar as for the first generation of Eurocodes:

$$\Sigma G_{k,i} + [\psi_{1,1} \text{ or } \psi_{1,2}] Q_{k,1} + \Sigma \psi_{2,j} Q_{k,j} \quad (3.26)$$

where the choice between $\psi_{1,1}$ and $\psi_{2,1}$ is up to the National Annex. However, as there is no background document, an estimate for the resulting reliability index on annual or life time is not known.

3.5.6.2 Thermal actions

The thermal actions for fire resistance assessment of Eurocodes are classified into two families: nominal fire temperature curves and physically-based thermal actions. The first family of thermal actions, in particular, ISO-standard fire curve, has been applied for several decades and their reliability depends in fact on the fire resistance requirement imposed by the regulatory authorities. From a scientific point of view, it is actually a classification system with unclear reliability target, but very largely based on real fire feedback. Note also that for instance one positive test is considered as sufficient, neglecting the rules for design by testing as given in prEN 1990:2023 Annex D.

As far as the second family of thermal actions is concerned, the reliability background of corresponding design parameters adopted in current EN 1991-1-2 are based on the outcomes of the research works conducted in Europe (CE, 2002). In the scope of these research works, one of the key parameters for physically-based thermal actions, the fire load, is dealt with in detail according to the semi-probabilistic approach of Annex A of ENV1991-1 (CEN, 1994) in order to get a specific partial factor for this action. The starting point of this approach is to take the failure probability value of $7,23 \times 10^{-5}$ per working life of the building corresponding to the safety index $\beta=3,8$ and adjust it by taking account of the occurrence probability of fully developed compartment fire so that the partial factor for the fire load can be derived. The adopted occurrence probability of a fully developed compartment fire is from German data (Bub, 1979). As several parameters can influence either positively or negatively the occurrence probability of a fully developed compartment fire, they are introduced separately for a more accurate design of fire load. This design approach has been incorporated in EN 1991-1-2. In addition, the characteristic values of design fire load recommended in current EN 1991-1-2 are based on a 80% fractile value which is about 22% higher than the mean value.

In the current draft of EN 1991-1-2, new design values of the partial factor for the fire load are incorporated to take account of two other reliability classes, namely $\beta=4,3$ and $\beta=3,3$. The same methodology to reach a

safety index of 3,8 has been applied in the determination of additional design values of safety factor for the fire load.

3.5.6.3 Material properties

For fire design, two types of material properties are needed, the first one is relative to thermal properties and the second one concerns mechanical properties, both of them at elevated temperatures.

For fire resistance assessment according to Eurocodes, the design values for partial factors (thermal and mechanical) of all the materials are brought to 1,0 whatever their design values for room temperature design are. The characteristic values of all the thermal properties are taken as the mean values. The characteristic values for mechanical resistance are the following:

- Steel: nominal value [CEN, 2020] (0,12% fractile)
- Concrete: characteristic value (5% fractile)
- Timber: 20% fractile (5% fractile for room temperature design)
- Masonry: characteristic value (5% fractile in principle)
- Aluminium: nominal value (not exactly but similar to steel).

It can be found that the basic design values are quite different from one material to another. However, the reliability background of using only one partial factor value of 1,0 for all materials remains still a matter of ongoing research., similar as in the first generation of Eurocodes.

3.5.6.4 Calculation models

As far as the calculation models (in particular for finding the temperatures in the compartment and the structure) of Eurocodes for fire resistance assessment are concerned, the majority of them has no reliability background. However, certain design rules recently developed for the second generation of Eurocodes are based on following validation criteria according to testing standard of passive fire protection materials [CEN, 2002]:

The results predicted with design rules shall not be on the unsafe side by more than 15% of the reference result;

Individual calculation results from design rules on the unsafe side compared to reference results shall not exceed 20%;

The mean value of all differences between calculation results from design rules shall be on the safe side.

One can conclude simply that a probability of 85% is taken to validate the design rules for fire resistance assessment.

It is necessary to point out that all the design models using tabulated data are derived from currently used basic combination coefficients for room temperature ULS design, namely 1,35 for permanent loads and 1,5 for imposed loads.

3.5.6.5 Conclusions

The review of the design procedures of current Eurocodes with respect to fire resistance assessment shows that on the one hand, the reliability of the design rules is not dealt with in a global way and on the other hand, in a lot of cases, the reliability background is not available. Note also that in many countries the fire safety level is governed by requirements specified in minutes fire resistance using ISO 834. These requirements are not part of the Eurocode.

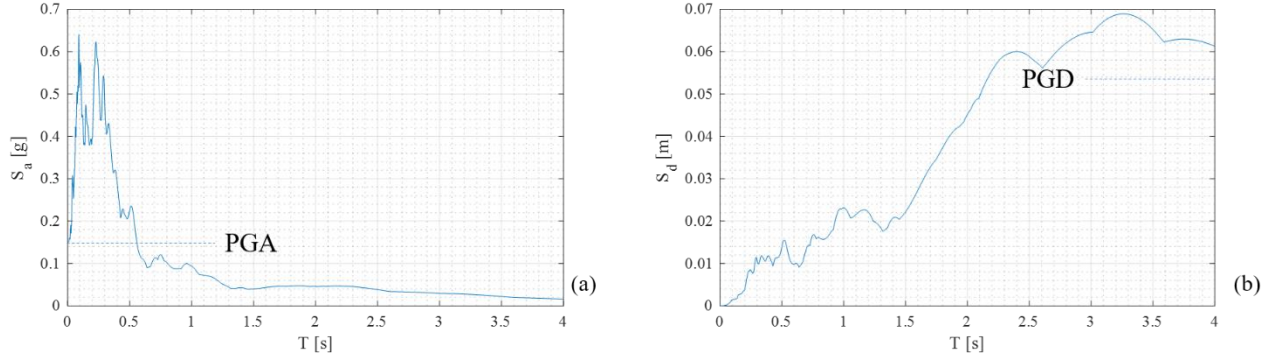
3.5.7 Seismic design situation

3.5.7.1 Characterization of the hazard

Earthquakes result from strain energy accumulation in the earth crust due to tectonic movements, followed by sudden release at the attainment of the crust material strength. The energy released at the source (fault) results in a local kinetic energy density that decreases with distance R from the source and therefore in ground motion of decreasing amplitude (geometric attenuation).

From a structural point of view earthquakes are rapidly varying support motions, rather than applied loads. The motion of the structure (its response) is influenced by its dynamic properties, i.e. mass and stiffness, as well as its strength and ductility. The relation between the maximum response of a simple structure and its natural period of vibration T and damping ratio ξ is described by the (absolute) acceleration spectrum and the (relative) displacement spectrum, shown for an individual earthquake event (i.e. the motion recorded at one site during one event of magnitude M on a source rupturing at distance R from the site).

Figure 3.8 Acceleration (a) and displacement (b) response spectra for an earthquake event (a single M, R pair).



Given the high degree of unpredictability in both loading and response use is made of a probabilistic approach (PSHA or Probabilistic Seismic Hazard Analysis). In most PSHA's the occurrence of seismic events is modelled as a stationary Poisson process. The interval time between two successive events is random with an exponential distribution. The mean arrival time is equal to the so-called return period T_R . Equivalently one can use the exceedance rate $\lambda_{S_a(T)}(s)$ of a given ground motion level s , which is the reverse of the return period.

The basic equations are given by:

$$\lambda_{S_a(T)}(s) = \lambda_0 [1 - F_{S_a(T)}(s)] \quad (3.27)$$

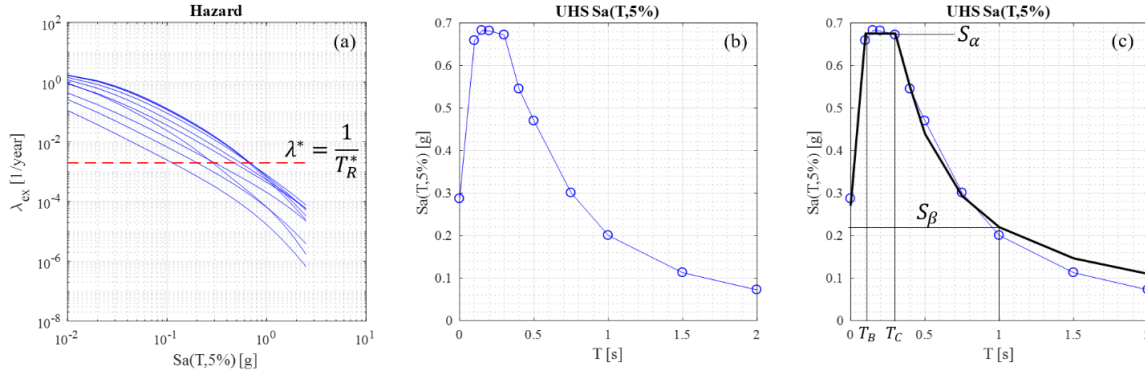
$$F_{S_a(T)}(s) = P[S_a(T) < s] = \iint P[S_a(T) < s \mid M=m, R=r] f_{M,R}(m, r) dm dr \quad (3.28)$$

where:

- $\lambda_{S_a(T)}(s)$ frequency of exceeding any given ground motion value;
- λ_0 rate of relevant seismic events generated by all considered seismic sources;
- $S_a(T)$ spectral acceleration at any given vibration period;
- M magnitude of the earthquake;
- R source to site distance.

By repeating the analysis for different values of the natural vibration period T a set of λ -curves (seismic hazard curves) is obtained. By setting a value of the return period $T_R = 1/\lambda_{S_a}^*$ values of S_a at different natural vibration periods, characterized by the same probability of exceedance are obtained and can be plotted versus T . The spectrum obtained by this procedure is called the Uniform Hazard Spectrum (UHS), and is shown in Figure 3.9.

Figure 3.9 Seismic hazard curves for spectral acceleration at different vibration periods (a), uniform hazard spectrum for a fixed return period T_R (b) and analytical approximation of the uniform hazard spectrum in the code (c).



The UHS is at the basis of the definition of the seismic action in all modern codes for seismic design and assessment, including of course EN 1998. Usually, the UHS resulting from PSHA is approximated with an analytical shape that can be described with a reduced set of parameters, as illustrated in Figure 3.10.

The scatter in ground motion is described within the attenuation law (or ground motion prediction equation, GMPE), which is a key component of the PSHA. These laws, obtained by regression on empirical data (spectra like those in Figure 3.8 for events of known M and R) usually in the form $\ln S_a(T) = \mu_{\ln S_a}(M, R) + \varepsilon \sigma_{\ln S_a}$, essentially describe $S_a(T)$ as a lognormal random variable and its (logarithmic) standard deviation $\sigma_{\ln S_a}$ (the so-called record-to-record variability) that accounts for the scatter and is stochastically independent of M and R ; ε is a standardised normal variable indicating the number of standard deviations that the ground motion at the building site is above or below the median predicted value. In practice:

$$P[S_a(T) < s | M=m, R=r] = \Phi\left(\frac{\ln s - \mu_{\ln S_a}(m, r)}{\sigma_{\ln S_a}}\right) \quad (3.29)$$

EN 1998-1:2004 contained two different spectral shapes, denoted as Type I and II, associated with different magnitude ranges and to be used in regions of lower and higher seismicity, respectively. These shapes were than "anchored" to a T_R -dependent value of the peak ground acceleration a_g . In the current draft of Eurocode 8, prEN 1998-1-1:2020, the spectrum is defined in terms of two T_R -dependent ordinates, S_α and S_β , respectively at maximum acceleration and at $T_\beta=1s$, a choice that allows more refined description of the geographical variation of the seismic action in Europe.

Note: PSHA studies have been carried out Europe wide and are being continuously updated in interaction with SC8 in order to support an harmonized cross-border definition of S_α and S_β within EN 1998.

3.5.7.2 Limit states, return period and load combinations

A total of four limit states are considered in EN 1998:

- operational limit and damage limitation (both SLS);
- significant damage and near collapse (both ULS).

In EN 1998-1:2004 the return period for action related to the SD (significant damage state) for CC2 is set equal to 475 years (actually 10% exceedance probability in 50 years). In the current draft this value is confirmed. If calculations are based on the limit state NC (near collapse) a return period of 1600 years is recommended. It should be noted that EN 1998 also gives attention to the collapse of non-structural elements. They are very sensitive to the seismic load and can cause relatively large damage.

The load combination to take into account for the seismic situation is given by EN 1990:2023 equation (8.16):

$$E_d = \sum G_{k,i} + A_{Ed} + \sum \psi_{2,j} Q_{k,j} + (P_k) \quad (3.30)$$

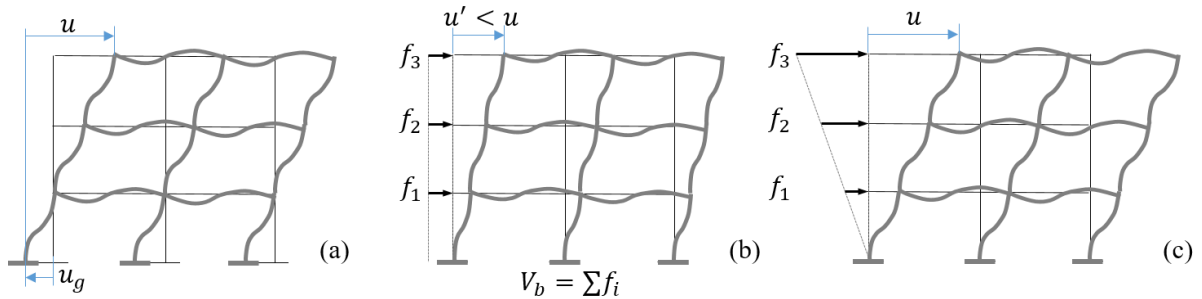
In seismic engineering an extra factor ψ_E is present to indicate the amount of mass related to imposed load live that contributes to the inertia in the dynamic calculation. No reliability-based background is given.

When use is made of response-history analysis (RHA, EN 1998-1-1:2020) the main selection criterion for the sets of recorded ground motions is a good match between the response spectrum obtained as an average of the individual motion spectra and the target spectrum. The latter is the UHS for the site but, notably, the latest EN 1998 allows for a more realistic conditional spectrum as a target (see Annex A). The code requires a minimum of 7 records and there is no compensation for statistical uncertainty resulting from this low number. The resulting average deformation value serves as the demand for the structural deformation capacity. Since usually for structural properties mean values are taken and strain hardening is considered in the model, capacity design requirements checks are checked differently from linear analysis.

While NC is the limit state of real interest for safety, the SD limit state is the one that marks the technical/economic limit between reparability of damage and need for demolition after the earthquake. It should be noted that, according to the current design philosophy, unless other seismic protection strategies like, e.g., seismic isolation, are adopted, damage is expected as a consequence of relying on ductility in the design. For this reason, SD is taken as the reference limit state for the design of new structures. The detailing for ductility implies satisfaction of NC verifications. For the very same reason, NC is the reference limit state in EN 1998-3:2020, i.e. existing structures are explicitly checked for near collapse since the global ductility cannot be "implied".

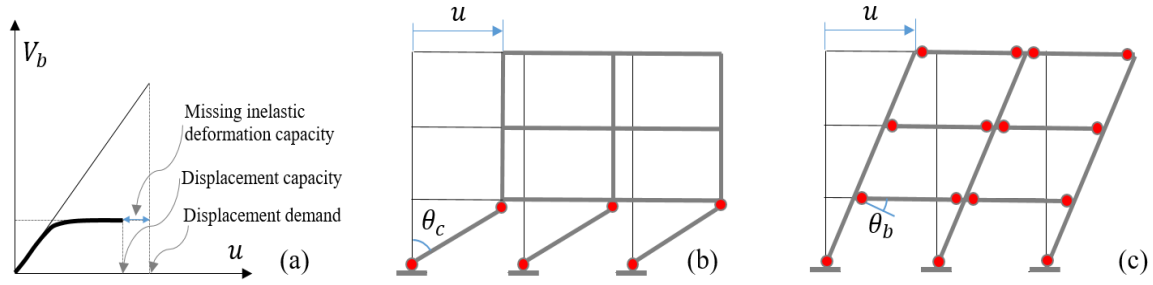
3.5.7.3 Structural analysis

Figure 3.10 Deformation resulting from support motion (a), deformation resulting from (uniformly distributed) equivalent lateral forces (b), deformation resulting from more "realistic" lateral force distribution (also in terms of intensity) (c).



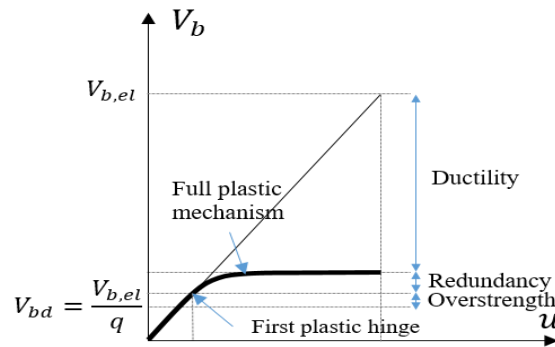
Effects of earthquakes (imposed displacements, as in Figure 3.10a have been traditionally translated into equivalent horizontal (lateral) forces, initially uniform Figure 3.10b and then more realistically distributed Figure 3.10c. In the past, however, design procedures of this type did not provide the structure with the necessary displacement capacity (Figure 3.11a) due to the development of unfavourable mechanisms, as for instance the "soft story" Figure 3.11b. It was only in the 1960s (Blume et al, 1961) that the concept of "capacity design" was introduced, i.e. the idea of proportioning member strengths so as to prevent brittle mechanisms, both at the local level, by requiring brittle shear resistance to be larger than the plastic shear, and at the global level, favouring beams' yielding over that of columns Figure 3.11c.

Figure 3.11. Base shear vs top-displacement curve of a structure with inadequate ductility (a), soft-storey plastic mechanism (b) and beam-sway plastic mechanism (c).



Like other codes worldwide, the first generation EN 1998(2005) also contained the concept of a force-reduction factor, called "behaviour factor" and denoted by q . This factor allows design based on reduced equivalent forces and linear structural analysis (static or dynamic, depending on the relevance of higher modes), relying on ductility and over-strength provided by capacity design and redundancy, as shown in Figure 3.12.

Figure 3.12 Reduction of design base shear as a function of ductility, redundancy and overstrength.



The second generation EN 1998 maintains this previous well-established procedure referred to as "**force-based**", but it also introduces a procedure referred to as "**displacement-based**". In this procedure the checks of verification are on the comparison of the displacement demands versus their corresponding capacities. This procedure, present in the EN 1998 first generation primarily for assessment of existing structures, is now codified also for design.

In the displacement-based approach the verification of ductile mechanisms is carried out in terms of deformations (i.e., according to the format of inequality 8.2 in EN 1990:2023). For example, the chord rotation θ (Figure 3.13) is used for frames. For this reason, models for the inelastic rotation capacity of members are provided for reinforced concrete and steel members in Clause 7 of Part 1-1 of EN 1998-1-1:2020. These models are the updated versions of those already used in EN 1998-3:2005 (see Annex A for further information on these models, and their model uncertainty).

Figure 3.13. Definition of chord rotation for a cantilever column (a), a fixed-end column (b) and beams and columns in a frame (c).

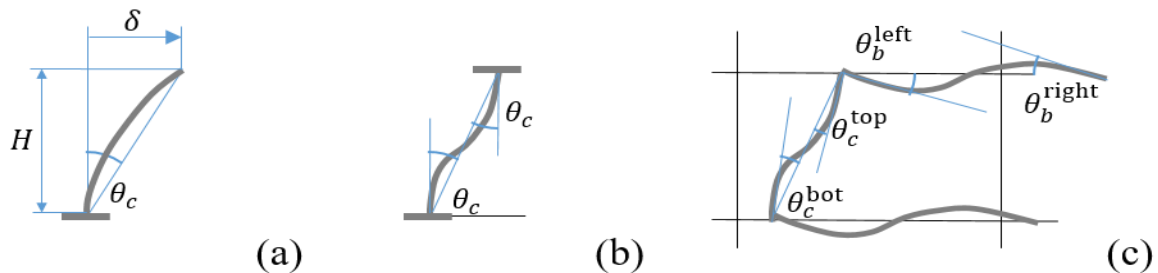
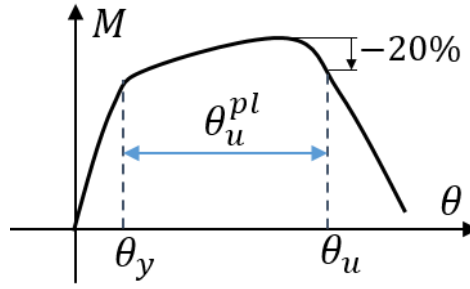


Figure 3.14. Moment-rotation and definition of ultimate rotation θ_u .



In terms of rotations, the design values for the capacities related to the limit states SD and NC are expressed as (Figure 3.14):

$$\theta_{SD} = \frac{1}{\gamma_{Rd,SD}} (\theta_y + \alpha_{SD} \theta_u^{pl}) \quad (3.31)$$

$$\theta_{NC} = \frac{1}{\gamma_{Rd,NC}} (\theta_y + \theta_u^{pl}) \quad (3.32)$$

where:

α_{SD} is the part of the plastic rotation θ_u^{pl} associated with a state of damage consistent with the values of q used in force-based design (α_{SD} is between 0,3 and 0,5);

$\gamma_{Rd,SD}$ is partial factor on the resistance (deformation capacity) at SD;

$\gamma_{Rd,NC}$ is partial factor on resistance at NC;

θ_y and θ_u^{pl} are the median values for the rotation at yield and plastic part of the rotation at ultimate level.

It is noted how the γ_{Rd} factors are to be applied to the member resistance (generalized deformation capacity for ductile failure modes and generalized stress for brittle or force-controlled failure modes), not to the individual input variables like material strengths (f_c , f_y , etc). They are derived to account for the variability of input variables describing the material properties, geometry and construction details, as well as the model uncertainty θ in each resistance model. For existing structures they are functions of the amount and quality of the collected information on geometry, details and materials, described with the so-called knowledge levels (KLG, KLD and KLM, respectively), which can take three possible discrete levels: minimum, average and high (Franchin and Pagnoni, 2018 and Background report CEN/TC250/SC8/N1076).

Verification of brittle mechanisms in the displacement method, such as shear in members or joints, is carried out in terms of forces (generalised stresses). It is important to note that the shear strength models V_{Rd} , however, are modified with respect to those used for other actions, since the shear resistance of members developing plastic hinges, i.e. undergoing cyclic inelastic deformations is obviously different and needs to be reduced as a function of maximum demand ductility (Biskinis, 2004, 2020).

Note: The models used for the concrete shear resistance in EN 1998-1-1:2020 resolves a long-standing issue. In the previous edition the models used for the seismic analysis were not in line with those in EN 1992. In the present version of Eurocode 8, the models (in particular for columns) can be considered as just a modification of the Eurocode 2 approach. The modification is in particular of importance for the assessment of existing structures.

3.5.7.4 Material properties

As far as fractiles for material properties are concerned, materials are divided in "new", "existing" and "added". For all materials, mean properties enter the analysis models for the purpose of establishing action effects (demand) through analysis.

For the member-by-member verifications:

material of **new** members (either part of a new design or of the retrofit of an existing structure) are represented as usual with their characteristic values and enter into the resistance models from the corresponding material-dependent Eurocode

materials in **existing** members are represented in terms of mean values, determined from all available sources of information (EN 1998-3:2020,5), i.e., integrating original design documents, like specifications or better the test reports, with in-situ destructive and non-destructive tests carried out at the time of assessment.

3.5.7.5 Target reliability

In the background of seismic engineering in the first-generation Eurocode there was no clear target reliability. Given the set of rules in the former EN 1998 calculations (Iervolino et al, 2018) indicate that annual reliability indices may range from (see Figure 3.15) about 2.9 to 4.3. The average would be in the order of $\beta = 3,5$ which is close to the ISO2394 recommendation of 3,3 in Table G.4 for the category CC2 and relatively large costs of safety measures. The second generation EN 1998 finally sets a reliability target in terms of annual probability of exceedance of collapse (which is close to NC) equal to 2×10^{-4} ($\beta = 3,5$).

Figure 3.15. Range of annual failure probabilities

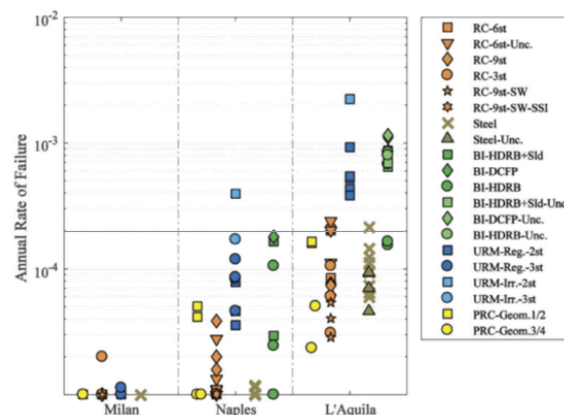


Figure 11. Global collapse failure rates (soil C-type).

Source: Iervolino et al. 2018

3.5.7.6 Seismic protection strategies

The previous paragraphs describe the action characterization, load combinations, material properties and analysis methods, limit states and target reliability in the seismic design or assessment situation, implicitly referring to conventional moment-resisting frames. The same notions apply to reinforced concrete as well as steel, composite or timber construction, and of course to other similar structural types, like shear wall buildings or dual systems (where frames and walls share the lateral inertia forces arising from the seismic action). In all cases, the underlying seismic protection strategy relies on capacity-design to provide the global ductility necessary for the structure to undergo safely inelastic deformation as a result of the seismic energy input.

The traditional ductility-based strategy, however, is definitely not the only one employed, even though it is the most used one, mainly for its similarity to conventional construction practices. The use of mechanical devices as a means to control response to seismic action (control of vibrations) is widespread and it is usually the preferred solution for structures in higher consequence classes, due to the superior performance. EN 1998 provides the basis of design of structures equipped with anti-seismic devices (which are regulated by the product standard EN 15129 in Part 1-1, chapter 6) and additional indications on their use in buildings in EN 1998-1-2, chapter 8 (Base isolated buildings) and chapter 9 (Buildings with energy dissipation systems), and

bridges in EN 1998-2, chapter 8 (Specific rules for bridges equipped with anti-seismic devices). These devices belong to two great categories, often used jointly.

Seismic isolators are devices able to reduce the seismic forces within the structures and are characterized by high stiffness in the vertical plane and low stiffness in the horizontal plane, so that the structural behaviour between structure and ground be uncoupled. The following typologies of isolators are currently used (metallic elastic-plastic isolators were more in use in the past):

Elastomeric isolators, with or without lead core, denoted respectively rubber bearings and lead-rubber bearings; Sliding isolators, with either plane or curved sliding surfaces, treated for controlled friction.

Supplemental damping is provided with devices of viscous (solid, fluid) or hysteretic (steel, shape-memory alloys) behaviour, working in parallel with the isolation system in the relative displacements of the structure with respect to the foundation (buildings) or the piers (bridges, commonly isolated at the deck level). Supplemental damping is also provided with devices working in the relative displacements between floors of buildings, in the form of braces. Buckling-restrained braces (BRB) gained widespread use.

For the numerical simulation of structures equipped with anti-seismic devices it is of fundamental importance to have knowledge of main parameters governing the behaviour of such devices, like damping ratio, friction coefficient, viscosity coefficient, horizontal stiffness, etc. All those parameters should be described including their uncertainty and time-dependency; actually, the mean values and the coefficient of variation of those parameters can vary more than 50% with the ageing process. A probabilistic analysis of the isolation system should then account for the above variability, describing each isolator as uncoupled by the other ones of the same system, both at short and long-life time.

3.6 Issues related to specific materials

3.6.1 Geotechnics

3.6.1.1 Representative values of ground properties

In geotechnical engineering the representative value of a ground property is determined either:

- using a nominal value based on -engineering judgement as a 'cautious estimate of the value affecting the occurrence of the limit state', or by

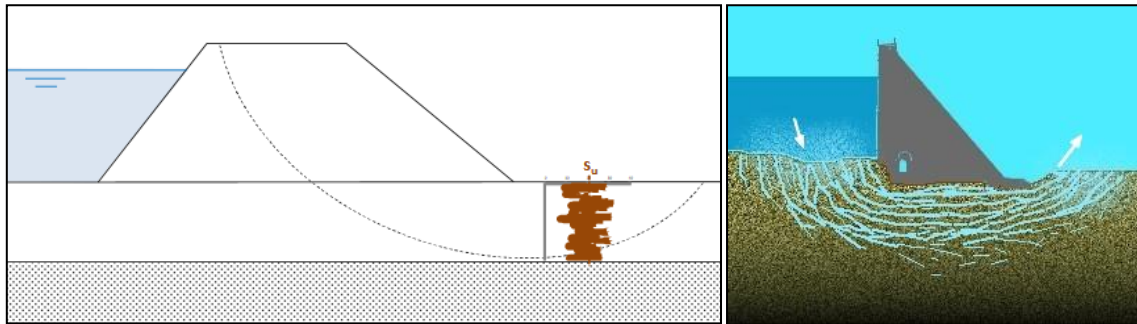
- determining a characteristic value using statistical methods 'such that the calculated probability of a worse value [...] is not greater than 5%'.

Evidently, also judgement-based cautious estimates of nominal values should consider to have the same degree of confidence in order to ensure consistency between the statistical and judgement-based approach in a reliability sense.

Besides the degree of confidence, the other important element in the definition of representative values for ground properties refers to the notion of the '*value affecting the occurrence of the limit state*'. The meaning of the statement deals (apart from the adequate strain level) with the influence of spatial variability on the ground property value to be used in design, as explained below.

Broadly speaking, most geotechnical failure modes involve large failure surfaces or volumes of ground, compared with the size of ground specimens used in laboratory testing or the ground volumes involved in in-situ testing (both in the order 50-500 mm, depending on the type of test). Common examples are failure surfaces in slopes or under shallow foundations, or the volumes of ground involved exerting active and passive earth pressures on retaining walls. In such conditions, the performance of a geotechnical structure, or the occurrence of the limit state, is influenced rather by the average ground property value in the affected ground volume than by the lowest value present (i.e. the 'weakest spot'). However, there are also limit states where spatial averaging is not applicable. Common examples are basal heave of deep excavations or internal erosion under cut-off walls or dams. That is the reason why EN 1997-1:202x considers both cases, with and without spatial averaging, as explained below.

Figure 3.16. Examples of failure modes with and without spatial averaging. The slope failure surface (left) cuts through the entire subsoil layer underneath the embankment, hence, the average shear strength along the failure surface is to be used for the characteristic value. On the other hand, internal erosion under a dam (right) is dominated by the weakest path (right hand side source: Internet).



As a consequence, EN 1997-1: 202x considers the use of two (limit) cases for determining the representative value of a ground property:

- a 'mean value' (spatial average) when the occurrence of the limit state is 'insensitive to the spatial variability' of the ground property in question (i.e. full spatial averaging is applicable);
- a 'point value' when the occurrence of the limit state is sensitive to spatial variability (e.g. the weakest spot governs the behaviour).

When statistical methods are used, in both cases the characteristic value is to be determined such that the probability of a more unfavourable value should not be greater than 5%. When cautious estimates are based on expert judgement (i.e. nominal values), the same degree of confidence should be targeted. The equations in EN 1997-1: 202x are fully in line with Annex D of EN 1990:2023.

When regional or global data sets are used, the equations to be extended by accounting for inter-site variability (i.e. variability of the mean values between sites) ought to be extended by accounting for inter-site variability (i.e. variability of the mean values between sites). Similarly, additional terms can be included to account for partial averaging (i.e. between no averaging and full averaging), where relevant. For background information and derivations reference is made to ISO2394 (2015), (Prästings et al., 2018) and (Deltares 2020). The references (Prästings et al., 2018) and (Bond & Harris, 2008) also provide guidance for deriving characteristic values for shear strength parameters using regression analysis.

The future version of EN 1997 also addresses a form of using prior knowledge in determining the characteristic value, more specifically by using recommended, literature-based values for the coefficient of variation ($V_x = \sigma_x / \mu_x$; see Annex A of EN 1997-1; 2021). When using these V_x , the statistical uncertainty reduces and the Student-t factor for a virtual unlimited test series should be used (i.e. $t_{\infty}^{0.95} = 1,645$).

Recent work involving random field simulation (e.g. Hicks, 2019) has provided insight into the 'effective property distributions', i.e. the probability distributions of equivalent ground properties to be used in limit state analyses when using uniform ground layer properties as opposed to modelling random fields explicitly. Such approaches are superior to the solely statistical-based approximations as described above in so far that they allow for quantifying the effective mean (usually lower than the field mean due to failure surfaces being attracted to weaker zones) and the effective variance (lower than the field variance due to partial averaging) without making strong assumptions.

The considerations above mainly address the situation with direct measurements of the ground property or parameter of interest, for example the undrained shear strength or the weight density as determined in laboratory tests. Often the estimation of ground properties is based on indirect measurements by using transformation models (in geotechnical engineering often called 'correlations', not to be confused with the more narrow definition of statistical correlation). An example would be the estimation of the undrained shear strength from the results of cone penetration tests (CPT). When determining the representative value using indirect measurements and transformation models, it is important to account for the model error (bias and variance) in the transformation model.

Finally, for the purpose of this document the technical committee 'TC304 Risk Management in Engineering Practice' of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) has compiled a

state-of-the-art review document of uncertainty and variability in geotechnical parameters (ISSMGE-TC304, 2020), including approaches to determine characteristic values and transformation uncertainties.

3.6.1.2 Partial factors

The calibration of partial factors as described in section 3.4 involves a generic limit state function that was developed to represent typical structural elements with self-weight, permanent and variable loads, the load effects of which are compared to the corresponding resistance. While this approach should work reasonably well for foundations (e.g. piles or shallow foundations), other geotechnical structures such as embankments or retaining walls do not fit into this format very well. The main reasons is that the ground is involved in (effects of) actions as well as in the resistance for many geotechnical structures at the same time. Hence, the distinction between actions and resistance is not that straightforward, and they are often correlated. Another point is the uncertainty in ground properties in combination with model uncertainty frequently outweighs the uncertainties in the (external) actions, whereas in structural engineering the load uncertainty is typically at least as important as the resistance uncertainty, if not dominant.

These differences between geotechnical structures with other structural elements (and among geotechnical structures themselves) are reflected in the various design combinations (DC 1-4) (Arnold P et al.) as detailed in EN 1990:2023 Table A.1.8, and the different sets of partial factors attributed to them. For many geotechnical design situations, the partial factors on permanent actions (γ_G) are equal to 1 or not applicable at all; and the partial factors on variable actions (γ_Q) are lower than for structural design in some cases. For the remainder, the differences are dealt with in EN 1997 by choosing the resistance and or material factors such that the desired level of reliability is achieved.

Notably, the choice of partial resistance and material factors was based on different grounds in the different countries in the first version of the EN 1997, and it probably will be for the implementation of the new version, too. Some countries rely on reliability-based calibration of partial factors as described in section 3.4 of this document. The majority, however, chooses the partial resistance and material factors such that the resulting designs match the designs obtained with earlier safety concepts as much as possible, such that the achieved level of reliability is at least comparable to pre-Eurocode design concepts. (Arnold et al. (2013)) provides information on how this issue was approached for the first version of EN 1997 in different countries.

3.6.1.3 Factor of safety, a confusing term in geotechnics

In geotechnical engineering often the ratio $FS = R_d/E_d$. (or R_k/E_k) is often referred to as the 'factor of safety'. Clearly this is not a safety factor like the partial factors in EN 1990:2023. Actually, it is an indicator of the degree of overdesign. To avoid confusion FS nowadays is also referred to as the "factor of stability". Note further that the inverse ($1/FS$) corresponds to the well-known unity check $UC = E_d/R_d$), traditionally often used by structural engineers.

3.6.2 Glass

3.6.2.1 Introduction

Glass is increasingly used as structural building material as transparency and brightness become more and more important in modern architecture. For that reason, TC250 has decided that in addition to the existing suite of Eurocodes there is a need for a Eurocode on the design of Glass structures. As first steps a Technical Report on the state of the art in Europe [Feldmann et al, 2014] and the pre standard (EN/TS19100) have been realised. At the moment the EN itself is under development. The background for the reliability and robustness aspects can be found in (Feldmann et al. 2014; Pisano et al 2015a; Ballarini et al. 2016; Pisano et al. 2019a; Sedlacek et al. 1999). This section 3.6.2 is mainly based on that document. Some additional comments are given.

For the material glass there are certain special aspects which have to be considered within the structural design. Firstly, glass is extremely brittle. Cracks in glass propagate at a velocity of up to 1500 m/s (Feldmann et al, 2014) (Nielsen et al. 2009). Consequently, the physical integrity of a monolithic glass pane may be lost in a split second, without any prior plastic deformation or other prior warning of impending failure. This may seriously affect the load-bearing capacity and the residual stiffness of the structural glass member or system of which the glass pane is part. Moreover, depending on the specific glass type and composition, there may be a risk of detached glass fragments or shards detaching from the pane that could cause human injury. Secondly, glass fracture may be caused by actions which are of no consequence in a design with other structural materials. Some examples: i) while an accidental drop of a hammer on a concrete slab is a non-event, the same may have

serious consequences on a glass floor or roof, when no corresponding design provisions are preventing failure of the glass element; ii) partial shading of glass exposed to solar radiation may be sufficient to cause thermal fracture if no corresponding design provisions are preventing this kind of failure; iii) the presence of a nickel sulphide inclusion of the order of magnitude of only 0,1 mm may cause spontaneous fracture in thermally toughened glass even years after installation. Also, here provisions are required, so that the required reliability can be achieved.

Given the above considerations, in addition to the limit states under normal service conditions (SLS and ULS), for structural glass also specific limit state scenarios like Fracture Limit State (FLS) and Post Fracture Limit State (PFLS) are considered. An overview is presented in Table 3.9. Safety verifications are therefore required for LSS-1, LSS-2 and LSS-3 not only for the intact state, but also at the time of fracture (FLS) and also in the post-fracture state (PFLS). FLS and PFLS verifications are required for reasons of robustness, because partial glass breakage must always be expected, even with the best structural design of the intact configuration. FLS and PFLS are actually special ULS verifications, but they are not called such because of the risk of confusion with "normal" ULS verifications.

Table 3.9. Limit State Scenarios (LSS) from prCEN/TS 19100-1

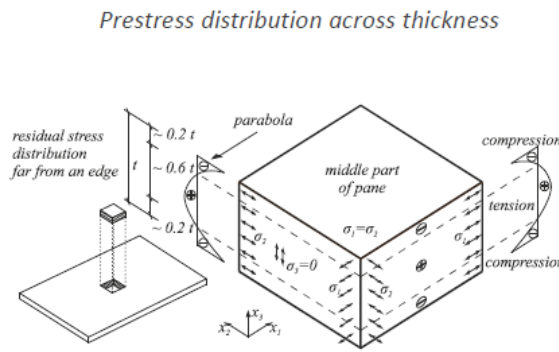
	Limit State Scenario (LSS)			
	LSS-0	LSS-1	LSS-2	LSS-3
Design for the unfractured glass state	SLS	SLS	SLS	SLS
	ULS	ULS	ULS	ULS
Design for the glass fractured state (safe glass fracture)		FLS		FLS
Design for the post-fractured state (residual load capacity)			PFLS	PFLS

3.6.2.2 Strength characteristics of glass (according to prCEN/TS19100-1)

Physically seen, glass has an extremely high strength. It may go up to ten thousand of N/mm². However, for real products such as e.g. flat glass, it is strongly influenced by the size of its surface flaws. The typical sizes of flaws (scratches) of marketed glass are in the range of 0,01 to 0,001 mm. Therefore, technically seen, annealed glass provides strengths of about 50 to 100 N/mm².

A highly reliable measure to increase the effective strength of glass is the tempering process where surface compression stresses, see Figure 3.17, are induced by rapidly cooling down the surfaces, when coming out of the furnace. These compressive stresses are in equilibrium with tensile stresses in the middle of the cross section. As these regions are not in contact with surface cracks, the material here is able to withstand very high tensile stresses. The pre-stress can be induced also by other techniques, e.g. by a chemical toughening procedure.

Figure 3.17. Pre-stress distribution across thickness in the centre of a glass pane

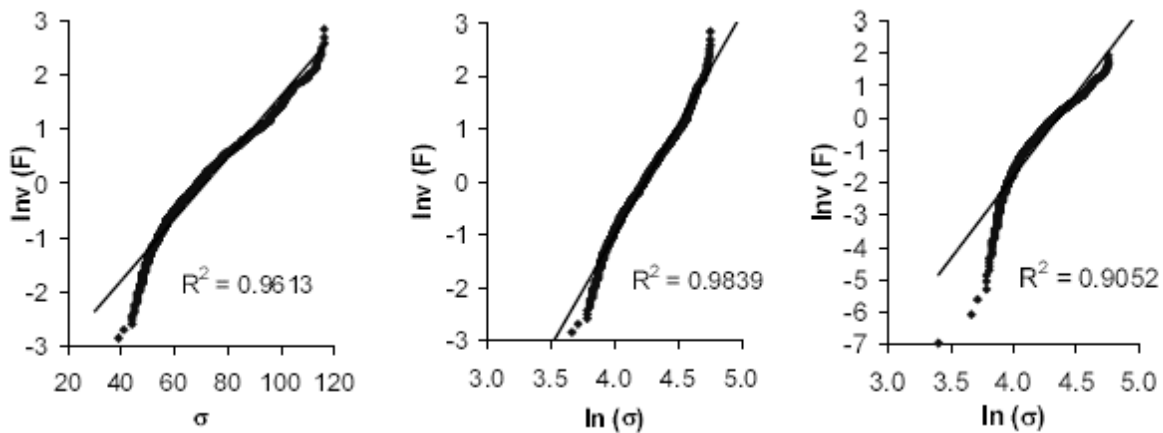


Source: Laufs 2000

Glass breaks when a dominant surface flaw under positive opening stress reaches the critical condition of fast propagation. To model the failure distribution, in the glass world (scientists and industry), the Weibull statistics, based upon the weakest link-in-the-chain concept, has been the most widely used approach. Especially the two parameter Weibull (2PW) distribution, for the probabilistic characterization of the mechanical resistance of glass has been a preference for a long time.

However, for *annealed float glass* there is evidence (see Figure 3.18) that the 2PW is not accurate in the left-hand-side tail (Danzer, 2006) (Yankelevsky, 2014) (Pisano et al, 2015). In addition, it was found that the lognormal approach, as used for other structural materials, is very suitable to describe the failure distribution of glass. (Wellershoff, 2006) (Fink, 2000) (Schula, 2015) (Datsiou et al, 2017).

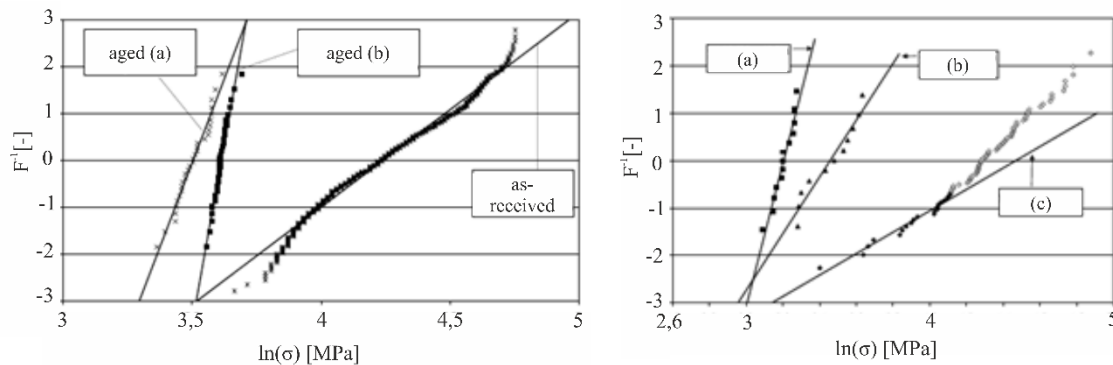
Figure 3.18. Normal distribution, lognormal distribution, Weibull distribution (from left to right)



Source: Wellershoff 2006

The samples for the surveys came from single suppliers and care was taken in the investigation of the specimen used. The testing setup was in accordance to EN 1288-2 and -3 (EN, 2000). The studied glass types were: float, chemically toughened, thermally toughened and heat strengthened glass. The samples of each population form a homogeneous population from the same distributors. The use of the basic strength of $f = 45$ MPa for the characteristic strength and the γ_M factor of 1.8 can be justified from these tests. Statistical uncertainties were included. Moreover, there is other independent research existing (Wellershoff, 2006) in which the glass strength and its corresponding safety were evaluated according to Annex D of EN 1990 (CEN, 2005/2010). Results from these investigations are given in Figure 3.19. It should be noted that uncertainties reflecting the model uncertainties of e.g. a glass beam or column is not explicitly accounted for in γ_M . There are, however, indications (Wellershoff, 2006) that the value of γ_M , nevertheless, is large enough.

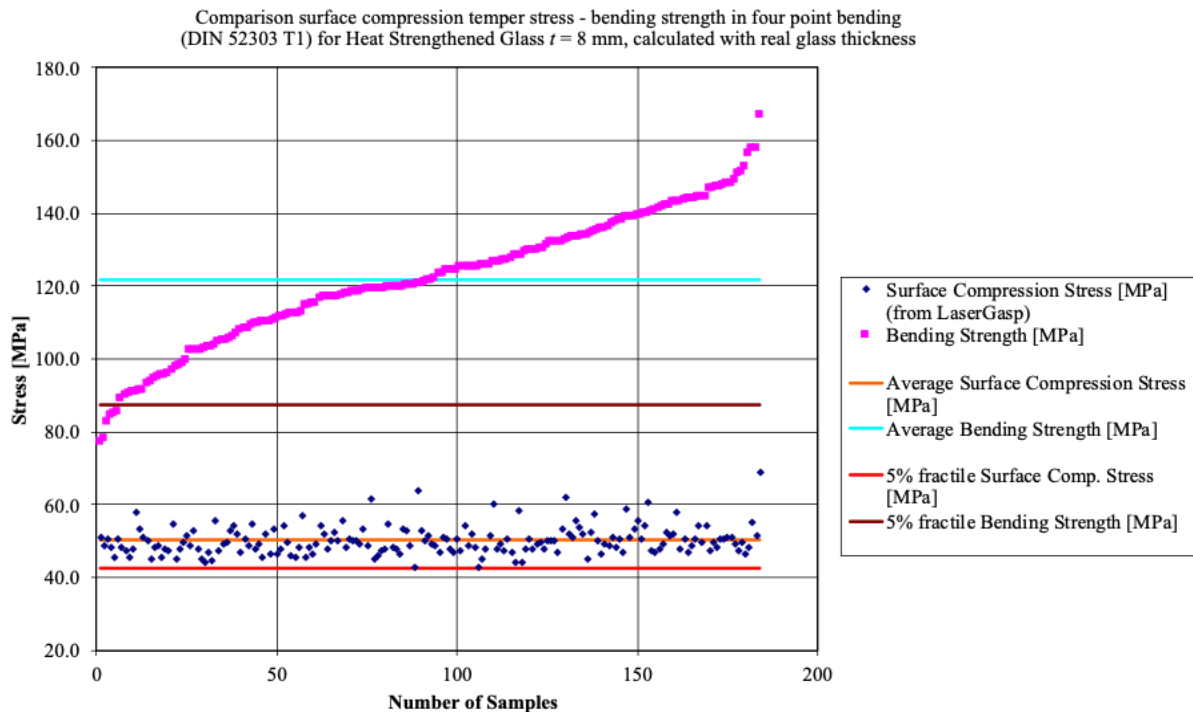
Figure 3.19. (left) strength data of annealed glass surfaces: aged with grit paper (aged (a)), aged with falling abrasives (aged (b)) and as-received; (right) strength data of annealed glass edges: strong axis bending cut edge (a), strong axis bending polished edge (b) and weak axis bending (c)



Source: Wellershoff 2006

The strength of *heat-treated glass* results from the capacity of pristine float glass and the residual surface compression induced by the thermal treatment, which has to be overcome before cracks open. A micro-mechanically motivated approach (Pisano et al, 2017) (Bonati et al, 2018) indicates that the probability of failure is obtained as the statistical convolution of the distributions of pristine annealed glass strengths and heat induced pre-stresses. The beneficial effect of the residual stresses is even more relevant when the state of stress from external actions is not equi-biaxial. Another favourable contribution is represented by crack healing due to synergistic effect of the thermal process and the consequent surface compressions, as discussed in (Pisano et al, 2017) (Zaccaria et al, 2016)

Figure 3.20. Measured results for heat strengthened glass.



Source: Feldmann et al. 2014

In the formula for the design strength of glass the pre-stress is given a separate partial factor γ_p . On the basis of the published literature, in which different distribution functions for the thermal pre-stress were used, it is

shown that a Gaussian distribution is well suited to represent the scatter of the pre-stress (Schula, 2015) (Pisano et al, 2019 and Pisano et al, 2017).

In Figure 3.20, the results obtained by four-point bending and optical measurements, for heat strengthened glass show that the experimentally determined strength values and optically measured values of pre-stress are subject to different scattering. It is easy to see from Figure 3.20, that the values of pre-stress and strength values of heat strengthened glass are statistically independent variables.

If one considers the annealed glass, its strength depends only on the surface or edge condition (micro cracks). However, the thermally induced residual stress state within the glass plate after the plate has been thermally toughened depends on its chemical material composition, its physical properties, the roller conditions of the toughening furnace and its quench section parameters (e.g. speed of the air as it exits the nozzle).

3.6.2.3 Design bending strength formulae and calibration of safety factors

In the current version of prCEN/TS 19100-1 two formulae for calculating the bending strength resistance of glass are given. In both of them, the pre-stress (index p) is taken into account in the design strength $f_{g,d}$ via the difference between the strengths of pre-stressed glass (index b) and annealed glass (index g), taken as the corresponding 5% lower fractiles of the corresponding statistical distribution, in agreement with product standards.

$$f_{g,d} = k_e \cdot k_{sp} \cdot \lambda_A \cdot \lambda_l \cdot k_{mod} \cdot \frac{f_{gk}}{\gamma_M} + k_e \cdot k_{e,p} \cdot \frac{f_{bk} - f_{gk}}{\gamma_P} \quad (3.33)$$

$$f_{g,d} = k_e \cdot k_{sp} \cdot \lambda_A \cdot \lambda_l \cdot k_{mod} \cdot \frac{f_{gk}}{\gamma_M} + k_e \cdot k_{e,p} \cdot \frac{f_{bk} - f_{gk}}{k_i \gamma_P} \quad (3.34)$$

The factor k_i is the interference factor, which specifically accounts for this statistical property. Whereas formula 3.33 follows the classical rules reported by most of the existing standards, according to which heat-treated glass is a material per se, formula 3.34 paves the way for a new approach. The presence of two formulae will permit a gradual transition to industries and engineers of the practice.

The partial safety factors γ_M and γ_P (Table 3.10) are borrowed from the established practice, but are also statistically justified to be in agreement with the target in EN 1990:2023. The factor γ_P results from the statistical analysis of non-destructive pre-stress measurements, but does not consider the aforementioned statistical interference. This is instead recovered through the coefficient k_i . Note that this reliability differentiation is on top of the differentiation factor K_F according to prEN 1990:2023-Annex A.1.7.

Table 3.10. Partial safety factors for material and pre-stress copied from prCEN/TS 19100-1

Design situations		Class of consequences		
		CC1	CC2	CC3
Persistent & Transient (fundamental combination)	Basic material γ_M	1,6	1,8	2,0
	Surface pre-stress γ_P	1,1	1,2	1,3
Accidental	Basic material γ_M	1,0	1,1	1,2
	Surface pre-stress γ_P	1,0	1,0	1,0

The choice of the partial safety factor for CC1 and CC3 is due to the fact, that higher or lower β -values of CC1 and CC3 respectively compared to CC2 render safety factors being 10% higher or lower. This is also confirmed by level III analyses by (Pisano et al, 2017) (Pisano et al, 2019). The γ -factors for the surface prestress are already explained above. For the γ -factors of the accidental load combination usually $\gamma_M = 1,0$ for CC2 is used

for ordinary material. In glass design, these values are increased by 10% on the safe side. This is not based on an explicit or formal reliability background. For the time being this is left to the individual member states. For the sake of completeness it should be mentioned that (Pisano et al. 2019) is based on the traditional Weibull statistics for glass.

Furthermore, the strength of annealed glass ($f_{g,k}$) certainly depends on several factors, such as the waviness of the surface (caused by the float process), the size of the glass plate, the type of applied stress and the duration of the load. The size- and stress-effect are properly considered in the Weibull statistics, whereas the phenomenon of static fatigue, due to the subcritical crack propagation, renders the design strength time-dependent. This is due to subcritical crack-growth and is described as standard in different literature via a micromechanical crack propagation approach:

$$k_{mod} = \left(\frac{1}{n+1} \right)^{\frac{1}{n}} (R)^{\frac{1}{n(n+1)}} (\dot{\sigma})^{-\frac{1}{n+1}} (t_L)^{\frac{1}{n}} \quad \begin{array}{l} n: \text{Material parameter} \\ (n=16 \text{ soda-lime glass}) \end{array} \quad (3.35)$$

where $n=16$ is the value chosen for humid conditions as worst case. Therefore, this k_{mod} accounts for the reduction due to subcritical crack growth, or in other words intercrystallite corrosion, formerly known as “static fatigue”. Note that in glass design this is relevant and classical cyclic fatigue is of minor importance, see below.

If the design/verification is carried out in proximity of the glass edge, the influence of the edge finishing (e.g. cut, strong defects; ground, lighter defects; polished, very little defects) has not to be considered. The level of pre-stress in the glass is affected additionally by the thermal toughening process (horizontal, vertical) itself. These factors are taken into account in the design formulae by introducing the coefficients k_e , k_{sp} , λ_A , λ_i , k_{mod} , k_p and $k_{e,p}$. The values of those factors are believed to be chosen in such a way that they can accommodate the bias and scatter involved.

Ageing was considered via the research on abrasive actions during the service life of glass, as well as the implementation of those actions into destructive testing in (Datsiou, 2017) (Datsiou et al, 2017) (Datsiou et al 2017). In the process of researching work cyclic fatigue of glass has also been studied (Hilken). Therein the influence of fatigue during the service life was found to be substantial. However, when considering the cyclic fatigue effect of fluctuating wind, it turns out, that the consideration of the k_{mod} -factor coming from static fatigue is sufficient. This is due to the fact, besides others, that a threshold below which no failure occurs was found (Hilken). Using common crack propagation laws, the cyclic fatigue can be predicted sufficiently. When using pre-stressed glass, almost no cyclic fatigue effects could be observed, except of the case, where all of the peaks of the cyclic amplitudes significantly reached beyond the surface prestress, which normally does not occur. Note that CEN TS does only refer to structures, whose design assumes quasi-static loading only including wind, which means that other fatigue actions are out of scope of the CEN TS 19100. Possibly, the Eurocode to be developed will consider this later on.

3.6.2.4 The bending strength safety of annealed glass in national codes on design

In the frame of establishing the safety level of the European design code of glass structures, it is also of interest to know the corresponding levels of the current national codes. Many of the current national codes, are using the semi-probabilistic design approach, including the actions as specified in EN 1991. Therefore, the γ_M values on the resistance side, can more or less directly be compared. The literature survey shows, that the differentiation of the glass strength in terms of reduction factors, as given in CEN/TS 19100 Annex A and Annex B cannot be found in the national codes. However, most of the national codes are also using γ_M in the range of 1,8, just as CEN/TS 19100 does. Moreover, the newly introduced reduction factors (k - and λ -factors) of CEN TS 19100 effectively result into additional safety margins, compared to national codework.

3.6.3 Fibre-Polymer Composites

The CEN Technical Specification (TS 19101) ‘Design of Fibre-Polymer Composite Structures’ applies to the design of buildings, bridges and other civil engineering structures, including permanent and temporary structures, composed of Fibre-Polymer Composite materials (also known as Fibre-Reinforced Polymer structures) or combinations of Fibre-Polymer Composite materials and other structural materials (for hybrid-Fibre-Polymer Composite structures). The scope includes, but is not limited to, Fibre-Polymer Composite laminates, profiles, sandwich panels (including web-core panels), bolted and bonded joints, composed of glass, carbon, basalt or aramid fibres, thermoset resins and adhesives, and polymeric foam or balsa wood cores. Fibre-

Polymer Composite cables, concrete rebars, strengthening components and honeycomb cores are not covered by the TS. The main manufacturing processes included are pultrusion, filament winding, hand layup, resin transfer moulding (RTM), resin infusion moulding (RIM), and vacuum-assisted resin transfer moulding (VARTM). In contrast to most other building materials the manufacturing process has a major impact on strength and stiffness of the Fibre-Polymer Composite structure as it defines the fibre lay-up.

The TS applies to Fibre-Polymer Composite structures in which the temperatures inside members, joints and components, under service conditions, are higher than -40°C and lower than $T_g - 20^{\circ}\text{C}$, where T_g is the lowest glass transition temperature of either the Fibre-Polymer Composite material, the core or adhesive materials. T_g is specified as the onset value of the storage modulus decay, obtained from dynamic mechanical analysis (DMA), and should be at least 60°C .

3.6.3.1 Basis of design

The standard equation for the design value of the resistance is given by:

$$R_d = \frac{1}{\gamma_{Rd}} R \left\{ \eta_{c,i} \frac{X_{ki}}{\gamma_{mi}} ; a_d ; \Sigma F_{Ed} \right\} \quad (3.36)$$

where:

γ_{Rd} is a partial factor accounting for the uncertainty in the resistance model, and for geometrical deviations, if these are not modelled explicitly; for values see prCEN/TS 19101:2020.

$R\{\dots\}$ denotes the output of the resistance model;

$\eta_{c,i}$ is the conversion factor accounting for effects of moisture and temperature, effects of ageing of materials, including the uncertainty of those effects;

X_{ki} represents the characteristic values of material or product properties (defined as 5% fractiles);

γ_{mi} is a partial factor for a material or product property accounting for unfavourable deviations of the material or product properties from their characteristic values and the random part of the conversion factor;

a_d denotes the design values of geometrical parameters;

F_{Ed} denotes the design values of actions used in the assessment of the design value of the effect of actions, E_d ;

i is for the i^{th} material or product property.

For the design of creep rupture, fatigue, adhesive connections, and fire, the product $\gamma_{Rd} \cdot \gamma_m$ is replaced by γ_m , which is a single partial material factor taking into account all the uncertainties and deviations mentioned above.

The partial factor for a material or product property, γ_m , depends only on their coefficient of variation and shall be determined from testing.

For the partial factor for the resistance model, γ_{Rd} , values are given for (i) profiles and laminates (for material failure and global and local buckling); (ii) sandwich panels (for material failure, global and local buckling, laminate wrinkling, core indentation and punching failure); (iii) bolted joints (for net-tension, pin-bearing, shear-out, block shear, and pull-out failure).

Where possible, γ_{Rd} values were derived from detailed statistical analyses of a significant number of resistance models, based on the assumptions of EN 1990:2023 and ISO 2394:2014; some of the values were also validated through explicit reliability analyses of design equations based on the Eurocode format. (Sá et al, 2021)

and (Pacheco et al, 2021) present probability distributions for the model uncertainties of the resistance models based on trustworthy experiments collected from the literature.

The conversion factor, η_{ci} , is composed of two conversion factors for temperature and moisture effects and can be expressed as $\eta_c = \eta_{ct} \cdot \eta_{cm}$. Nominal values are given for specific Fibre-Polymer Composites, core and adhesive materials, and for specific environmental conditions, for a 50-year period (which may be extended, if appropriate maintenance and inspection procedures are implemented). These nominal values were derived from experimental results collected from the literature. As an alternative to the nominal conversion factors, the temperature- and moisture-dependent material properties can be determined by testing in accordance with Annex D in EN 1990:2023. The same applies for properties affected by environmental conditions not covered by the nominal factors.

The conversion factor for temperature, η_{ct} , takes into account changes of material properties due to material temperatures under service conditions (see above), excluding effects of long-term exposure. Nominal values are provided for Fibre-Polymer Composite materials; these depend on whether the properties are fibre- or matrix-dominated and are functions of T_g . Further values are listed for sandwich foam and balsa wood core materials, with the former also depending on T_g and the latter on density.

The conversion factor for moisture, η_{cm} , takes into account changes of material properties due to moisture absorption over time, including ageing effects resulting from long-term exposure. Nominal values were derived for unprotected Fibre-Polymer Composite materials and epoxy adhesives, for three exposure classes.

A sub-clause concerning creep effects is further included, complemented by an annex. Creep effects on the strength of Fibre-Polymer Composite structures are considered by limiting the stress levels for the quasi-permanent combinations of actions. Fibre-type-specific stress limits are provided in the clause concerning Ultimate Limit States, see below. Creep effects on the deformations are taken into account by creep coefficients, which reduce the relevant moduli of elasticity of the materials. Values are specified for different moduli for pultruded Fibre-Polymer Composite profiles, Fibre-Polymer Composite laminates/plies and core materials, for a 50-year period; they were derived from experimental results collected from the literature. Annex A provides further values for additional periods of design service lives.

3.6.3.2 Materials

Strength and stiffness of Fibre-Polymer Composite components depend not only on the chosen type of fibre and resin as well as on the specific fibre lay-up. Standard test methods are specified to obtain the Fibre-Polymer Composite, core and adhesive material properties normally required in the design of Fibre-Polymer Composite structures. Instructions for how to extract or manufacture test coupons are also provided, in order to avoid damaging a Fibre-Polymer Composite member and for when the laminates are relatively thick. For a preliminary design, indicative (mean) values of selected material properties are listed in an Annex, which can be applied, provided that appropriate values of coefficients of variation are assumed to derive design values.

3.6.3.3 Connections and joints

In Fibre-Polymer Composite structures bolted, adhesive and hybrid bolted-adhesive connections, and joints (whereby a joint is composed of one or more individual connections) occur. Concerning bolted connections under in-plane forces, the resistances in the case of laminate failure modes to be verified are net-tension, pin-bearing, shear-out, and block-shear failure. Subjected to out-of-plane tensile forces, the failure modes at the bolt locations are pull-out failure through the laminate, bolt failure in tension, and failure of the laminate at a bolt location owing to interaction between in- and out-of-plane forces. At the joint level, shear and tying force failures of web cleat joints are considered.

A Fibre-Polymer Composite structure comprising adhesive joints shall be designed as fail-safe, i.e. joint failure shall not result in failure of the structure or critical parts thereof. Failure of an adhesive joint is considered as an accidental situation according to EN 1990:2023. The failure mode of adhesive connections shall be either cohesive failure in the adhesive or fibre-tear failure in the adherend; adhesive failure, i.e. failure in the adherend-adhesive interface, shall be prevented by appropriate material selection and surface preparation.

The resistance verification of adhesive connections is performed according to Eq. 1, using a single partial material factor for the adhesive connection resistance, $\gamma_{M,ac}$, as mentioned above. This partial factor depends on (i) the type of inspection and maintenance and accessibility of the adhesive connection, and (ii) the application conditions, either with fully controlled, i.e. reproducible process parameters, or only partially controlled parameters. Owing to insufficient experimental results, the range of values specified is adopted from literature

(EUROCOMP, 5.1.10, P(4), i.e. from the products of the partial factors $\gamma_{m,1} \cdot \gamma_{m,2}$). The background to these values could not be traced and they thus require further validation. Three methods are described for the design of adhesive joints and connections: (i) design assisted by testing; (ii) design based on stress analysis; (iii) design based on fracture mechanics. Stress-based and fracture mechanics-based failure criteria are provided for the latter two methods and experimental validation is mandatory in both cases.

Provisions are included for the design of hybrid bolted-adhesive connections. Depending on the relative stiffness of the adhesive layer, the individual resistances of the bolted and adhesive connections can be summed or not.

3.6.4 Membrane structures

3.6.4.1 Analysis

The membrane surface is *form found* by numerical methods to determine an equilibrium state. The form found state shall result in acceptable levels of stress, strain, and geometry under design loads that are determined in accordance with the form found geometry. This means that a geometrically non-linear analysis is mandatory.

Uncertainties regarding the effects of the action due to uncertainties of the structural analysis model and the analysis type should be considered.

The supporting structure shall also be stable without the membrane or membrane parts. The stability of the whole structure depends on the integrity of each component. A local failure can cause a general collapse. Complementary cables and redundant devices should be envisaged to increase the safety at critical points, such as mast tops, arches or frames, in order to prevent failure which can cause great damage or personal injuries.

3.6.4.2 Material properties, characteristic values

Tensioned membrane structures can be fabricated from fabrics like polyvinyl chloride (PVC)-coated polyethylene terephthalate (PET)-fabrics (PES-PVC), polytetrafluoroethylene (PTFE)-coated glass fibre fabrics (glass-PTFE) or foils made from ethylene-tetrafluoroethylene (ETFE), or other materials.

Characteristic (or nominal) values shall be determined in accordance to Table 3.11.

Table 3.11. Relevant standards for different materials and properties

Material	Property	Relevant standards or rules
fabrics	tensile strength	EN ISO 1421 or EN ISO 13934-1 and EN 1990 :2023 Annex D
fabrics	tensile stiffness	EN 17117-1 or any other appropriate rule
foils base material	tensile strength	5% fractile from tests at 23 °C according to EN ISO 527-3 minimal 5 specimen with k_r according to EN 1990:2023 Annex D The coefficient of variation V_x shall not exceed 0,05 Recommended for ETFE: 40 MPa
foils seam or edge	tensile strength	Similar to base material The coefficient of variation V_x shall not exceed 0,08 Recommended for EFTE: 30 MPa
foils	tensile stiffness	biaxial testing

3.6.4.3 Material properties, design values

The design resistance for a specific design situation is calculated according to equation (8.18) of EN 1990:2023.

For *textile* structures, the design value R_d of a resistance is calculated as

$$R_d = \frac{1}{\gamma_M} R_k (k_{age}; k_{biax}; k_{dur,*}; k_{temp,*}; k_{size}; k_x) \quad (3.37a)$$

For *foil* structures, the design value R_d (ULS) or R (SLS) of a resistance is calculated as

$$R_d \text{ (or } R) = \frac{1}{\gamma_M} R_k (k_{age}; k_{biax}; k_{dur,*}; k_{temp,*}; k_{single,*}; k_{QL,*}; k_x) \quad (3.38)$$

where

R_k is the characteristic value of the particular resistance determined with characteristic or nominal values for the material properties and dimensions;

γ_M for simplicity, the partial factors γ_m and γ_{Rd} given in Formula (8.18) of EN 1990:2023 may be combined into a single partial material factor ($\gamma_M = \gamma_m \cdot \gamma_{Rd}$);

k_{age} is the modification factor for environmental (aging, deterioration) effects;

k_{biax} is the modification factor for biaxial effects;

$k_{dur,*}$ is the modification factor for various load durations (permanent, long-term, medium-term, short-term)

$k_{temp,*}$ is the modification factor for various temperature effects ;

k_{size} is the modification factor linked to the panel size;

k_{single} is the modification factor for single layer ETFE structures;

k_{QL} is the modification factor considering the manufacturers quality level;

k_x is a variable for a modification factor for not yet specified effects;

Modification factors k_{biax} , k_{age} , $k_{dur,*}$, $k_{temp,*}$ can be determined with project specific tests. If compliant tests from other projects are available, these values can be used.

The recommended value for the partial factors γ_M are given in Table 3.12:

Table 3.12. The recommended value for the partial factors γ_M .

Application	Resistance parameter	Partial factor
structures from PES-PVC or glass-PTFE fabrics	material	$\gamma_{M0} = 1,4$
	connections	$\gamma_{M1} = 1,5$
ETFE foils	material	$\gamma_{M0} = 1,1$
	connections	
Note:		

Source: prCEN-TS19102-MasterDoc-2021-11-20

The partial factor for the resistance of connections γ_{M1} may be determined according to formula 3.38 taking into account the production quality of the executing production facility with Quality Level 3 (corresponding to EXC3, High execution quality level).

At least 30 series of minimum 5 tests per series from the last 6 month shall be available on connections of the base material under consideration. Weld seams and edge connections should be considered, as well as all welding machines and welders, for which this partial factor should be applied.

$$\gamma_{M1} = (1 - 1.64 \cdot V_x) / (1 - 0.8 \cdot \beta \cdot V_x) \geq \gamma_{M0} \quad (3.38)$$

where:

V_x is the mean value of the coefficient of variation of at least 30 series of minimum 5 tests per series from the production of the last 6 months [$V_x = (\text{standard deviation})/(\text{mean value})$]; V_x shall be validated by the tests carried out in the specific project on the specific detail. The coefficient of variation V_x shall not exceed 0.08 for the mean value and in the specific project.

β is the reliability index, $\beta = 3.8$; other values may be chosen depending on the reliability class of the project

3.7 Checking deformation capacity / brittle behaviour

Ductility and global/local deformation capacity are important structural properties and can have large influence on the behaviour of structures. The following aspects are worth mentioning:

Sufficient ductility or deformation capacity contributes to a redistribution of loads when locally internal forces and stresses have reached the limit value;

As a result, local peak stresses or locally weak strength areas can be neglected or averaged;

In a similar way it makes the structure less sensitive to some types of modelling and calculation inaccuracies;

Temperature effects, settlements, eigen stresses (e.g. resulting from inhomogeneous cooling), etc are of less or no importance;

Ductility can help to survive dynamic actions for instance in case of seismic or wind actions;

Under certain circumstances (e.g. gradual load increase, observable deformation) ductility can give a timely warning for collapse and thus a reduction of the failure consequences.

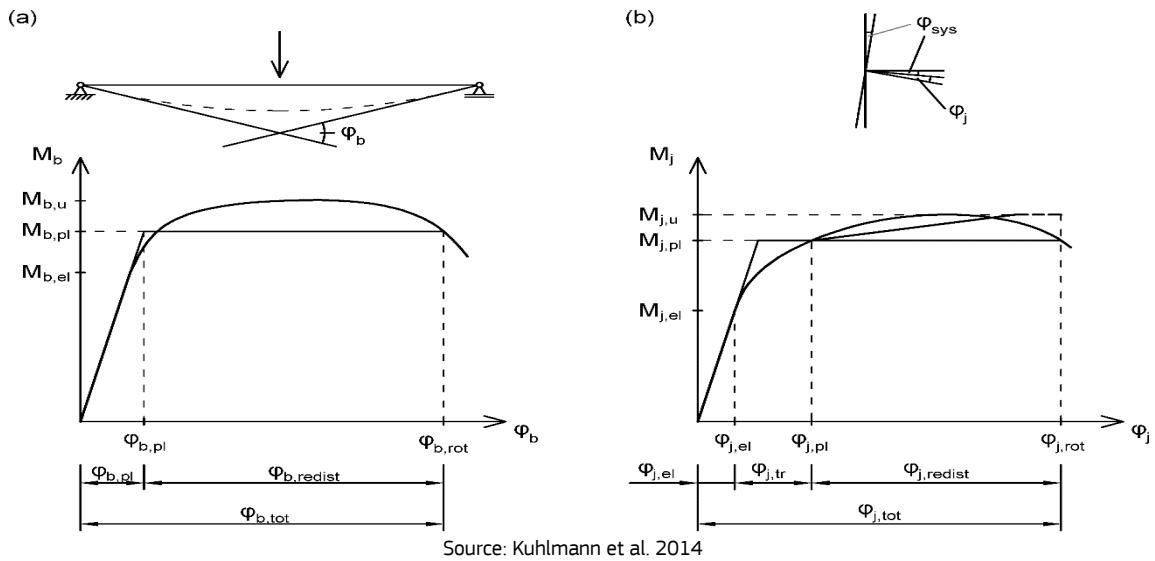
Note that ductility may be defined on the material level, the member level and the system level. A ductile material does not guaranty automatically a ductile structure: detailing rules are equally important. Material related codes contain many rules implicitly dealing with this matter. Examples are the minimum reinforcement requirements and limitations on redistribution in concrete or the f_w/f_y ration in the case of steel structures.

It is important to realise that also materials classified as ductile in the end have a finite deformation capacity and that we have to deal with many types of hardening and softening behaviour. For this reason we need models for e.g. the moment-rotation-relationships of joints or other potential critical zones. To some extent they are available in the literature and also in codes (e.g. 8.2 in EN 1990:2023), however, from a reliability point of view the deformation or strain properties are not treated on the same level as the resistance side. The distinction between characteristic values and design values is not always made and mean values and coefficients of variation are not codified. It is also not always clear how to use ductility information in the nonlinear (FEM) analysis and the background of some rules are only documented in a limited way. Given the increased use of nonlinear numerical methods of analysis an explicit modelling of post yield behaviour (including fracture energy and strains at rupture) are however becoming more and more important. For advanced non-linear calculations for structures with large failure consequences it can, for that reason, be recommended to take results from recent research projects into consideration. This section gives an impression.

Example: Plastic design of steel structures

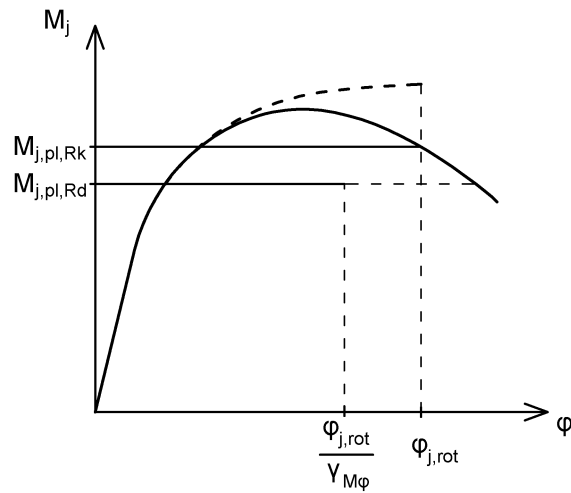
Typical situations where deformation capacity or rotation capacity is checked (at least indirectly) is when deriving the conditions for the application of the plastic hinge theory. There it makes a difference whether the plastic hinge forms in the beam or in the joint. For beams the difference of rotation $\varphi_{b, \text{redist}}$ is considered, whereas for joints the total rotation $\varphi_{j, \text{tot}}$ counts, see Figure 3.21.

Figure 3.21. Comparison of moment-rotation curve of (a) beams and (b) joints according to (Kühnemund, 2003)



Regarding the reliability concept it should not be neglected that there is an interaction between resistance and deformation. If the rotation capacity is taken belonging to the reduced resistance $M_{j,pl,Rd}$ it is automatically increased. So may be the reference value should be taken from $M_{j,pl,Rk}$ and in a second step reduced by an own partial factor (Figure 3.21). A better approach is a combined checking as realised by the so-called beamline theory.

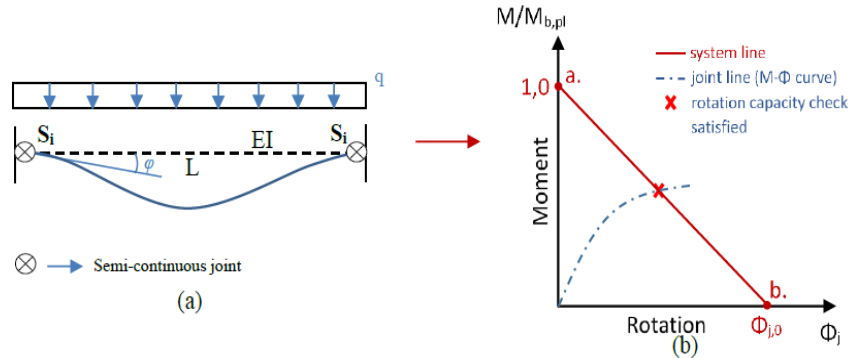
Figure 3.22. Definition of the available rotation capacity of joints according to Kühnemund, 2003



In the recent years, (Schäfer, 2005) introduced the concept of the so-called beamline method. The beamline method is an integrative design approach regarding the design of the structural system and the choice of the joints as part of it, also mentioned in (Demonceau et al, 2021). It provides in a single diagram and compares with each other (i) the requirements of the global structural system, in terms of joint strength and rotation capacity (system line), and (ii) with respect to the local system the available strength and rotation capacity of a joint (joint line), see Figure 3.23b. For the derivation of the system beamlines extensive analyses varying both the system's characteristics as well the joint characteristics should be done. The global system analyses are performed on beam models and the mechanical characteristics of the joints are simulated through the application of spring elements. The simplest, representative for the derivation of a beamline structural system is depicted in Figure 3.23a.

The semi-continuous joints are represented by springs, the stiffness S_i of which is varied. In such way beamlines can be derived for one type of structural system, one type of load case and for different values of the joints' mechanical characteristics. A further variation of the other two parameters, structural system and load case leads to the derivation of system beamlines for certain value ranges of structural systems' type and geometrical characteristics, load cases and mechanical characteristics of joints. On the other hand, the beamline for the joint can be derived dependent only on the $M-\phi$ curve of the joint configuration. The beamline method of (Schäfer, 2005) was developed for sway-frames.

Figure 3.23. (a) Example of beam model for the derivation of beamlines (b) derived beamline.

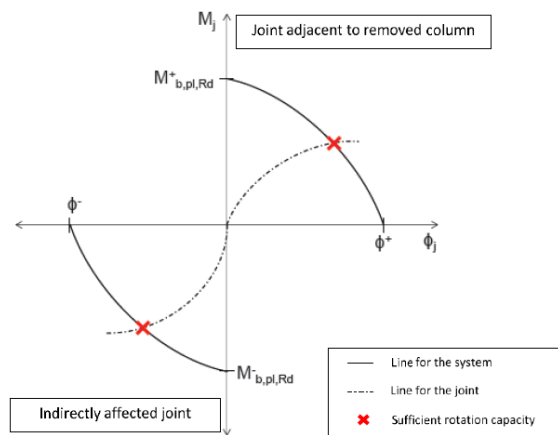


Source : Huber et al. 2004

Later, [Keller, 2019] using the results of ROBUSTIMPACT (Demonceau et al, 2021) evaluated the influence of the system on the required joint rotation capacity and expanded the "beamline method" for non-sway frames and for the load case "loss of a column".

For the load case "loss of a column" the directly affected joint is under positive bending and the indirectly affected joint under negative bending moment. Therefore, two beamlines for the system are needed, one providing the requirements of the system in terms of rotational capacity of joints under positive bending and one providing the requirements in terms of the rotational capacity of joints under negative bending, together with the $M-\phi$ curves of the joint for positive and negative bending, see Figure 3.24.

Figure 3.24. Check of sufficient rotation capacity for composite joints.



Source: Keller, 2019

The topic of deformation capacity and brittle behaviour is very relevant for the resistance against many accidental loads and the design for robustness. Recommendations like prescriptive tying rules in the code usually require the fulfilment of a set of underlying assumptions and performance requirements like a large deformation capacity in joints and prevention of brittle failures (e.g. shear failures). In O'Connor et al (2021), minimum levels of ductility are given for a specific level of stiffness from the surrounding structure of common

buildings. Nonlinear structural analysis offer a useful tool in the assessment of robustness rules. Depending on the sophistication of the adopted models, nonlinear phenomena preceding component failure/structural collapse can be represented to various degrees of accuracy, including the flexural strength, compressive arching action, tensile catenary/membrane action, diaphragm action of slabs/walls, etc.. In this respect, the use of nonlinear analysis in robustness assessment allows the consideration of the structure at relatively large deformations thus exceeding conventional strength-based norms, provided the deformation capacities at which components fail by fracture are not exceeded (Izzuddin et al., 2008). It would be helpful if codes would give more guidance on models and reliability based design values.

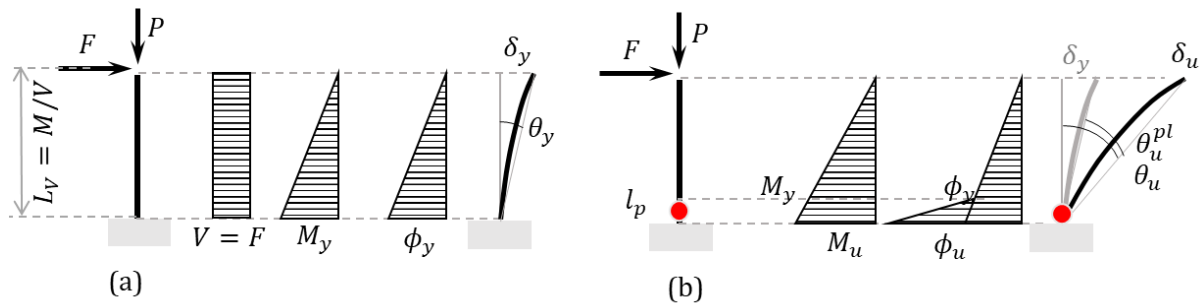
Most explicit attention to the ductility aspects and avoidance of brittle failure until now has been given in the *seismic design situation*. Economic solutions for a high seismicity are possible only by making use of the energy dissipation in the plastic ranges of the load deflection diagrams (Freeman, 1978; Fajfar, 2000). To avoid unintended premature brittle behaviour the notion of *capacity design* was introduced, making sure that brittle elements were sufficiently stronger than the ductile ones. Eurocode 8 (EN 1998-1-1,7:2020) now also includes criteria on inelastic deformation and strength criteria for reinforced concrete, steel, timber and masonry construction. The increasing use of nonlinear FEM in non-seismic design situation will likely require similar developments.

Next to the mechanical and statistical modelling of post yield behaviour on the material, member and structural level there is the question of the reliability level required. The consequences of ductile (member) failure are often less serious than for brittle failure, partly because of the load distribution effects and partly because of the warning effect. As a result it may make sense to use a higher consequence class in case of brittle behaviour. According to EN 1990:2023 this leads to increased levels of the partial load factors. It may however be considered as more convenient to keep the load factors constant and have an increased partial factor on the resistance side. For the moment this is up to the National Annex.

Example from Eurocode 8

For the sake of illustration, the general form of these models is described with reference to frame members, exemplified in Figure 3.25 with a cantilever scheme, where the chosen deformation parameter is the chord rotation θ (see also 3.5.7). The cantilever scheme is also the prevalent one in the experimental campaign that has produced over the years the data supporting the models now included in the code, by now amounting to hundreds of inelastic cyclic tests.

Figure 3.25. Deformed shape, top displacement and chord rotation for a cantilever column: a) at yield, b) at ultimate.



As shown in Figure 3.25a, until yield, the response is elastic, curvature and moment are proportional and linear (this is an approximation for concrete members where stiffness changes in the portion where the moment is larger than the cracking one). After yield, due to hardening, the moment can still increase until a maximum value and an area undergoes plastic deformations: this is the plastic hinge of length l_p . Deformations are concentrated in this area and the integral of curvature of the plastic hinge length can be considered a rotation around the hinge centroid, that causes a plastic increase of the top displacement δ_p . The simplest mechanical models of the chord rotation at yield and ultimate are thus:

$$\theta_y = \frac{\delta_y}{L_v} = \frac{\phi_y L_v}{3} \quad (3.39)$$

$$\theta_u = \frac{\delta_u}{L_v} = \frac{\delta_y + \delta_p}{L_v} = \theta_y + \theta_u^{pl} = \theta_y + \frac{1}{L_v} l_p (\varphi_u - \varphi_y) \left(L_v - \frac{l_p}{2} \right) \quad (3.40)$$

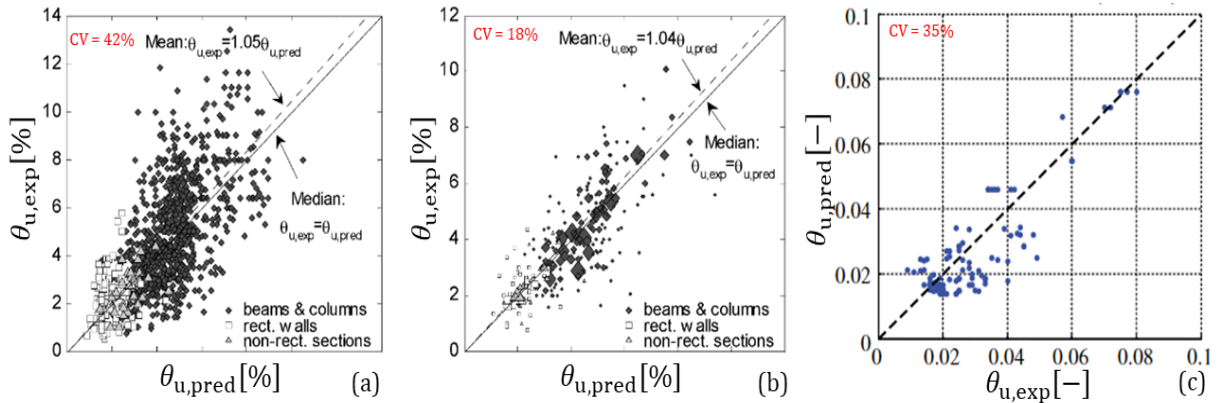
where the yield and ultimate curvature are obtained from a moment-curvature section analysis, (or predictive analytical equations for the most common cross section shapes like rectangular and circular), and $\theta_u^{pl} = \theta_u - \theta_y$ is the plastic part of the chord rotation

Even this so-called mechanical model, however, has an element of empiricism, in the model for the plastic hinge length. It is included in corrected form in the Eurocode 8 to cover those elements whose section shape is not common. For typical elements in frames, with rectangular sections in RC buildings or wide flange/I-beam sections in structural steel, empirical formulas are preferred, due to their better agreement with experimental results.

Specific models for the yield and plastic rotations included in Eurocode 8 are based on research by (Biskinis and Farids 2010a, b) for new and existing RC members (evolution of those in EN 1998-3:2005, due to Panagiotakis and Fardis, 2001), by Grammatikou et al (2018a, b) for retrofitted members, by Lignos and Krawinkler (2010, 2013) for steel members.

All these models are developed to be unbiased, i.e. have a median of the ratio of experimental to predicted rotation equal to one, and report the coefficient of variation of this ratio, which describes the model uncertainty (something between 20% and 40%, indicatively). Examples are shown in Figure 3.26. In particular, Figure 3.26a and b refer to the ultimate rotation of concrete members (Grammatikou et al, 2018a), and two different statistical techniques to fit the same model and leading to markedly different coefficients of variation (42% versus 18%). Figure 3.26c refers to the plastic rotation of steel beams (Lignos & Krawinkler, 2011).

Figure 3.26. Comparison of predicted and experimental values of ultimate chord rotations for: a) and b) concrete, c) steel beams.



Source: Grammatikou et al. 2018b and Lignos and Krawinkler 2011

Final note: the models and rules presented here may differ from the solutions that have been developed in EN1998 for the seismic design situation. However, in both cases the intention is to reach a certain minimum safety level. For more discussion see also chapter 3.8.

3.8 Nonlinear analysis / FEM

3.8.1 Types of nonlinearity

The majority of design calculations and partial factor calibration is performed on the basis of *Linear structural behaviour* that is defined by a linear relation between strains and displacements, a formulation of equilibrium which is independent of deformations and a linear dependency of stresses and strains.

Within EN 1990:2023 the following types of nonlinearity are defined

geometric non-linearity, caused by changes in geometry, in most cases sufficiently accurate approximated by using the second order theory;

material non-linearity, caused by non-linear stress-strain relationship in the material;

contact non-linearity, caused by non-linear behaviour at the contact boundary between structural parts during introduction of actions.

Additionally, we may have *non-linearity of the limit state function* if the local resistance is a non-linear function of the various influencing variables. This, of course, is highly relevant for a proper application of reliability theory (in particular for simplified methods like FORM and PFM) and is dominantly present in the literature on reliability theory. In this chapter, however, we will concentrate on the classical types of structural non-linearities as defined in EN 1990:2023.

Note: Additional to the general statements in EN 1990:2023, chapter, 8 a special Annex to EN1990:2023 for this topic is under preparation; unfortunately this annex was not ready when writing the background in this report.

3.8.2 Use of the non-linear finite element method (NL-FEM) in assessment and design:

Two different types of procedures are to be distinguished:

(a) Conventional member by member checking, calculating load effects by applying linear or nonlinear FEM to be used as input to local resistance models.

(b) Checking the overall failure of the whole structural system (non-linear resistance) using nonlinear FEM.

The linear version of (a) is already a standard for a long time. When using the non-linear option, the load effects are not only depending on elastic properties, but also on other material properties.

In both cases (a) and (b) the consideration of non-linearities (in particular second order effects) are mandatory if they are unfavourable and relevant for the overall failure.

Notes:

Some computer programs generate automatically the actions from self-weight. The user has to take care himself that partial factors for favourable and unfavourable acting self-weight are introduced correctly. Note that these automatic generations of the self-weight can also influence the load path.

As superposition is not valid in nonlinear analysis, the start of the analysis is the formulation of a relevant load combination including a (time dependent) specific load arrangement. As such it is to be recommended to start with a classical elastic analysis to find out which load combinations and arrangements are relevant to be considered in the non-linear case.

FEM/NM on a small local scale (part of a wall or beam) can also be used to derive equivalent averaged material parameters for use in the overall analysis.

In non-linear limit state functions the theoretical values of the partial factors depend on the degree of non-linearity; for instance: for a steel column the coefficient of variation (and thus the partial factor) of the load bearing capacity may depend on the slenderness ratio. If nevertheless the same partial factor is used over the total range of column types it should be checked that deviations from the target reliability are within the accepted limits (see EN 1990:2023, Annex C).

3.8.3 Application of the partial factors on the action side

As discussed in section 3.2, EN 1990:2023 offers in 8.3.2.2 three options to calculate the load effects in a structure. In general (see section 3.2)

$$E_d = \gamma_{sd} E \{ \Sigma (\gamma_f \psi F_k); a_d; X_{Rd} \} \quad \text{EN1990:2023 (8.3)} \\ (3.41a)$$

To simplify matters, γ_{sd} and γ_f can be combined in one factor γ_F

$$E_d = E \{ \Sigma (\gamma_F \psi F_k); a_d; X_{Rd} \} \quad \text{EN1990:2023 (8.4)} \\ (3.41b)$$

When the problem is linear the partial factor also can be applied on the effects of actions:

$$E_d = \gamma_E E \{ \Sigma (\psi F_k); a_d; X_{Rd} \} \quad \text{EN1990:2023 (8.5)} \\ (3.41c)$$

The first one is theoretically to be preferred, but from a practical point of view the other two are used mostly in practice. In most FEM software only the option (3.41b) is operational, which is usually on the safe side.

In equation (3.41c) the design values of the action effect will be based on the highest partial factor (For example, a factor 1,5 for self-weight).. This may lead to a very uneconomic result. In the linear case when superposition is allowed it may be better to use eq. (3.41d).

$$E_d = \Sigma \{ \gamma_{E,i} E_i(\psi F_{k,i}); a_d; X_{Rd} \} \quad (3.41d)$$

3.8.4 Application of the partial factors on the resistance side

For the resistance side we may choose as starting points the EN 1990:2023 formula (8.18) or the simplified formulas (8.19) and (8.20).

When applying factors to material properties, the design value of resistance should be calculated from:

$$R_d = R \left(\frac{X_{rep}}{\gamma_M}; a_d; \Sigma F_{Ed} \right) \quad \text{EN 1990:2023 (8.19)}$$

When applying partial factors to the resistance, its design value should be calculated from:

$$R_d = \frac{R(X_{rep}; a_d; \Sigma F_{Ed})}{\gamma_R} \quad \text{EN 1990:2023 (8.20)}$$

Formula (8.19) is valid if the model uncertainty factor is close to 1.0 and (8.20) if the material factor is close to 1.0. Both expressions are correct if the relation is linear. When choosing an option in the nonlinear case, one should consider both reliability as well as economic aspects. Material related Eurocode have made their own choices (see EN 1990:2023 eq 8.3.5.1). Note that some types of non-linearity may work out differently for (8.19) and (8.20).

Notes:

For application of the formulas for R_d in NL-FEM on the *system* level, the resistance parameter R is usually defined as *an overall load amplifier* that in the incremental analysis runs from zero up to the value where the structure is no longer stable. The design value of the resistance then corresponds to the value of the load amplifier where instability occurs. Of course one may also stop the analysis earlier as soon as a sufficient

degree of reliability has been proven. This way, however, one does not get information on how much the structure is overdesigned. For more information see sections 3.8.4.1 and 3.8.4.2.

The factor γ_R includes the effects of uncertainty in all materials, local resistance models and overall system behaviour (see also the text on the adjustment of model uncertainty under FEM calculation further below).

In both equations 8.19 and 8.20 the material factor γ_M and the model uncertainty resistance factor γ_{Rd} have to be combined into one factor, γ_M or γ_R which of course may lead to some inaccuracies, for instance in the case of nonlinear resistance models. Part of this type of inaccuracy, however, also is present outside the application of non-linear FEM. In cases of multiple failure modes maybe conservative approximations have to be made. (see EN 1990:2023, eq. 8.21).

3.8.4.1 Application of EN1990:2023 (8.19)

The input to the FEM analysis is given by:

design values of the loads F_d ;

design values of the material properties: $X_d = X_{rep} / \gamma_M$;

geometrical properties a_d .

After (in steps) applying the loads F_d and assuming the structure is still stable there are members in the linear state and members in the non-linear (or yielded) state. In this approach, in principle, the local load effects in all members can be checked for all standard local limit state requirements according to the relevant Eurocodes in the same way as for linear FEM. However, if all necessary checks as well as the corresponding post limit state behaviour have been implemented correctly in the non-linear FEM this is not necessary as it is already carried out implicitly (actually we have arrived at case b).

It may occur that different limit states (local yield or rupture conditions) require different γ_M values for the same basic variable X . Most FEM-software offer no option to deal automatically with that situation. One may of course make a conservative choice by taking the highest value. Otherwise, one has to make several runs and suppress where relevant (artificially) the limit state requiring the low γ_M value.

Note: See also the section on avoidance of brittle behaviour.

3.8.4.2 Application of EN1990:2023 (8.20)

The input to the FEM analysis is given by:

design values of the loads F_d ;

representative / characteristic values of the material properties: X_{rep} ;

geometrical properties a_{nom} (or a_d where appropriate).

FEM programs do not always allow to introduce the partial factor γ_R on the level of local (member) resistance. A possible solution is to transfer the γ_R to the loading side and raise the loads up to $\gamma_R F_d$: We may formulate this as:

$$R_d = R(X_d; a_d; \sum \gamma_R F_{Ed}) \quad \text{EN 1990:2023 (8.20a)}$$

Actually, this comes down to a kind of load factor design). In case of material softening or geometrical stability problems it is conservative (but possibly very uneconomic). Next all members can be checked in the conventional way, leaving out of course γ_R . If the FEM contains all required limit state checks (and corresponding post line state behaviour) there is no need for further checking. Note finally that using (8.20a) in an incremental non-linear analysis the total loads $\gamma_R F_{Ed}$ should be applied proportionally in steps.

Remarks for the application of (8.20):

Depending on the type of resistance criteria favourable acting permanent actions should be multiplied by a factor 1.0 instead of γ_R (see also Annexes B.6.1 and B.6.2).

If several materials are present, one may choose one material as dominant (or decisive) and adjust the representative values of others (like in (8.21)). Note that the choice may have influence on the final result.

If in (8.20) mean values are used as representative values for the material properties the above procedures should be adjusted accordingly.

In general non-linear analysis is more complicated than linear analysis because of the fact that the superposition rule is not valid anymore. One has to concentrate on the most critical load combinations. It may be helpful to start from a survey in the elastic domain.

For further discussion and background, see the FEM-shear wall example in Annex B.5

3.8.5 Checks on deformations and deformation capacity

Next to (or as part of) the load bearing capacity assessment of the structure there is a need to consider the related deformations. This may be part of the checking of service-ability limit states with respect to deflection criteria. However, (see section 3.7) it is also relevant for the checking of local and global ductility requirements. In incremental FEM-analysis where proper models for post-elastic behaviour of both members and connections are incorporated (as done for instance in chapter 7 of EN 1998-1-1) this may occur partly implicitly. If calculations are based on the upper or lower bound theorems of plasticity theory no stepwise increase of the loads is performed and a separate check that the deformations and deformation capacities are sufficient may be necessary. Note finally that a force controlled incremental FE analysis can only detect the ultimate capacity of the structure but can't detect the degree of post critical ductility in the global behavior.

3.8.6 Reflection on the choice of upper and lower design values of structural properties

In addition to the direct strength parameters f , also values for other structural parameters have to be chosen, like the modulus of elasticity, hardening or softening models, strain at rupture, imperfections, bond, fracture energy, etc. A usually conservative approximation is to use standard upper and lower design values according to C4.4.2 in EN 1990:2023 Annex C. More refined methods are possible but at the cost of more calculation work. It is for instance pessimistic and not always necessary to take the lowest design values for the (ductile) resistances of all members. For less dominant members or parameters the use of say characteristic values (5% fractiles) could be safe enough. This depends of course on the degree of correlation between members, which however usually is only scarcely known.

One should also be aware that in general the use of lower design values for *local* resistance parameters is not always on the safe side. In some cases, a high value of a local resistance parameter may be unfavourable. A well-known example is that brittle (or strongly softening) mechanisms like shear or connection failure may be overlooked due to ductile yielding in nearby cross sections or members. To detect these brittle mechanisms on member level one should perform simulations using high (design) values for resistance parameters in ductile mechanisms. Another option is to prevent these mechanisms by applying capacity checks as in seismic engineering. Missing these mechanisms may have far stretching consequence for redistribution of loads and mobilizing reserve capacity under dynamic loadings. This issue is in particular necessary to consider when capacity design, seismic design or even full plastic design is done.

3.8.7 Adjustment of model uncertainty and corresponding partial factors under FEM calculation

When using FEM, it might be that the model uncertainties related to the FEM model are not consistent with the model uncertainties for classical design using design equations in the material Eurocodes (which may be biased for several reasons). In those cases, it is possible to check the FEM-calculation model directly with a representative set of experiments and add model uncertainties using EN 1990:2023 Annex D. This may give rise to the need for additional general or operational rules in chapter 8 of EN 1990:2023 of the following type:

- Design for non-linear problems assisted by numerical models
- Design for non-linear problems assisted by use of computational methods shall achieve the level of reliability required for the relevant design situation taking into account the model uncertainty of the computational method.
- Adjustment of material partial factors to account for the model uncertainty related to the numerical model.

Note: Further information about method validation and model uncertainty is given in Annex D following the same principles as for design assisted by experimental testing.

In this respect the status of Annex D in a specific CEN member state may be important.

Note: a revision of Annex D to include design assisted by numerical methods is under preparation in SC10/WG3.

3.8.8 Requirements for the quality of nonlinear numerical methods used

According to EN 1990:2023, clause 7.2.2(4) non-linear numerical models should be validated (or rather justified), to establish whether the numerical model correctly reproduces the necessary physical phenomena. It means that the numerical models used should be compared to numerical, experimental or benchmark results representing the main failure modes of the elements. These covers:

General aspects:

Checks on the correctness of the material modelling

Tests in order to check the characterization of relevant structural failure modes

Note: Comparing the result of a model of a structure with an experiment may also be misleading because the sources of differences cannot always be determined and errors from various sources can cancel each other.

Verification for the specific project:

Mesh sensitivity

Sensitivity to action input values

Adequacy and type of the solution method.

The loading process adopted by non-linear simulation in order to evaluate the structural resistance shall be defined according to engineering judgement consistently with the considered combination of actions at ultimate limit state. The loading process should where relevant account for the loading sequence due to staged construction methods.

Note: If methods go beyond the calculation models described in the Eurocodes, e.g. a more advanced nonlinear numerical model, then the partial factors cannot be used with calibration as described above. It is up to the National Annex to set up further guidance in order to achieve the required reliability level.

3.9 Calibration in case of degradation

3.9.1 Fatigue using SN-curves

3.9.1.1 Limit state formulation

Fatigue is mechanism where cyclic actions or action inducing repeated stress cycles cause effects like crack growth that can lead to failure. The actual failure can be caused by a relative large cycle or by stresses caused by another type of action. In some cases dynamics can play an important role. Given the nature of fatigue, not only the intensity of the action cycles but also the number of stress cycles is of great importance.

A common approach to deal with the fatigue mechanism is the so called S-N-curve-method. This method is based on experimental S-N-curves for constant amplitude loading, relating the (constant) stress range $S = \Delta\sigma$ to the number of stress cycles to failure N . Above the fatigue limit (if present) and in its most simple form the failure criterion is given by

$$N \Delta\sigma^{-m} = K \quad (3.42)$$

The parameters K and m are resistance parameters depending on material specific aspects and local structural geometry. Note that for practical applications in steel and aluminium structures a two branch version for the S-N-model is used having parameters K_1 , K_2 , m_1 and m_2 . More details are discussed in Annex B.6 of this document. For other applications the model can also depend on the (cycle) mean stress level (as for instance in concrete, timber and FRP).

To account for variable amplitude (VA) loading, the Palmgren-Miner damage accumulation rule (hereafter called "Miner sum") is often used which leads to the following limit state function for a given period of time t (design service life or time of inspection):

$$g(t) = D_{cr} - D_n \quad (3.43)$$

where D_{cr} is the Miner sum at failure (with expectation often assumed to be close to 1,0) and D_n is the damage based on equation due to n applied load cycles present in the period t .

$$D_n = \sum_{i=1}^n \frac{1}{N_i} \quad (3.44)$$

with N_i following from equation (3.42) for an applied stress range $\Delta\sigma_i$. To evaluate D_n we thus need the stress ranges $\Delta\sigma_i$, usually presented either as a stress range histogram or a spectrum and a total number of cycles n for the design life time (or other period) under consideration. The histogram or spectrum may be derived on the basis of load effect simulations in combination with rain flow counting or reservoir counting methods. In some cases the load model presents an equivalent constant amplitude value. In general the model for the determination of the load effect $\Delta\sigma$ contains several (partly uncertain) global and local model factors.

Due to the fact that according to Miner's rule (although not really true in reality) the order of the loads is not relevant, we may evaluate (3.44) by an ordering in magnitude. This means that we can divide the spectrum into a number of stress intervals, calculate the damage D_i for each stress interval and multiply with the number of cycles in the interval; D_n follows by adding the contributions of all stress intervals. It is clear that the model uncertainty in the fatigue model should be accounted for where in fact the standard rules from EN 1990:2023 Annex D apply.

3.9.1.2 Target reliability

For structures in reliability class RC 2, the European standard EN 1990:2023 recommends a target reliability index for the design life time. It ranges from 1,5 to 3,8, depending on the degree of inspectability, reparability and damage tolerance.

Note that one should be careful here: the correct way to calculate the failure probability for the life time is by explicitly including the effect of inspection. This will lead to an increase of the reliability index and the target should still be 3,8. The reduction of the target to 1,5 is intended to compensate for the case where inspection is carried out in reality but not included in the calculation. It is in fact a strong simplification. One should never combine the two approaches: that is incorporating the positive effect of inspection on the calculated reliability level and taking the reduced 1,5 value as a reliability requirement. Actually there is even a need for a better specification of the inspection requirements and corresponding repair that would justify this reduction.

3.9.1.3 Probabilistic verification

The failure probability for a total period of time from 0 to t is given by:

$$P_f(t) = P(g(t) < 0) \quad (3.45a)$$

where $g(t)$ is given by (3.58). The stress distribution or spectrum in the expression for $g(t)$ follows from the application and the structural properties. One might also calculate the last year of the design service life or, where relevant, the intended inspection or repair period (see also section 2.4). For the annual evaluation holds:

$$P(\text{failure in year } t) = P(g(t) < 0) - P(g(t-1) < 0) \quad (3.45b)$$

(or, alternatively, the conditional failure rate, see chapter 2). This probability should be less than the annual target, which according to Table C.2 in EN 1990:2023 for reliability class 2 corresponds to $\beta = 4,7$. This value

however, is an upperbound and a lower value may be more appropriate because of the correlations in resistance and model uncertainties in time.

Note: the approach using an annual reliability target is considered to be more suitable for the fracture mechanics approach.

3.9.1.4 Semi-probabilistic approach

According to EN 1990:2023 formula (8.11) the design value of the spectral stress range level $\Delta\sigma_{i,d}$ should be derived from:

$$F_{fat,d} = \gamma_{Ff} F_{fat} \quad (3.46)$$

where F_{fat} is the (nominal) value of relevant fatigue action and γ_{Ff} is a partial factor for fatigue actions and covers the model uncertainties in the actions and the stress calculation. The precise meaning of F_{fat} (characteristic, mean or maximum loads/stresses) may be material specific. The same holds for the choice of the verification is in the elastic or plastic region. The value of γ_{Ff} depends on the models for fatigue actions and on the models for the effects of the fatigue actions. The recommended value of γ_{Ff} is 1.0 when the models for fatigue actions are safe sided such, that the uncertainties of the action effects, which are approximately equal to 10%, are covered. Otherwise, the recommended value is 1.1. Models for fatigue loads can be found in EN 1991 for e.g. wind (prEN 1991-1-4, Annexes F and H), traffic and waves (prEN 1991-1-8). These models offer (or should offer) the basis for the fatigue calculation for all materials. However, the further elaboration from the external loads to the relevant internal load effect that is suitable for a fatigue damage are highly material specific. As a result also the systematic deviations, the sources and degrees of uncertainty and thus the appropriate partial factor γ_{Ff} depend both the type of loading, the type of structure and the material. Uncertainties can be related to the simplified set of vehicles, the accuracy of the influence lines, the local stress model, the dynamic effects and in some cases the presence of the permanent load. Note that also the inspection regime and the consequences of the fatigue damage (e.g. damage tolerant) are relevant for the value of partial factors.

Accordingly, in EN 1992 and EN 1993 factors (referred to as λ factors at the moment) have been introduced for a number of load models in order to avoid damage accumulation. . Presently in the SC3-WG9, SC3-WG13 and AHG Fatigue, attempts have started to work on more realistic traffic assumptions considering also the influence of uncertainties. For the use of FRP in wind turbine blades technical specifications are under development. It is important that similar approaches are followed for other materials as well as for wind and wave actions.

On the resistance side the (characteristic) S-N curves are usually based on the 95% survival probability (including statistical uncertainty in accordance with EN 1990:2023-Annex D). The design values for the resistance (to be defined in the same dimensions as the fatigue load F) may be found according to EN 1990:2023 (8.22):

$$R_{dfat,d} = R_{fat} / \gamma_{Mf} \quad (3.47)$$

where R_{fat} is the representative fatigue resistance and γ_{Mf} the partial factor for fatigue resistance accounting for the consequence of fatigue failure and the application of inspections (and ease of repair) of fatigue-sensitive members.

It should be noted, that there are certain preconditions for the fatigue verification which are hidden in additional serviceability limit state requirements. Examples are that the steel structure should stay in the elastic range for the frequent load combination or that for the headed studs in EN 1994 there is a restriction on the upper loading for the characteristic load combination

Fatigue can be caused by more than one fluctuating action. In that case the distribution (spectrum) should be based on the combined history. When the mean stress level needs to be considered in the damage rule, the mean values may be based on EN 1990:2023 (8.17):

$$\Sigma F_d = \Sigma G_{k,j} + \Sigma \psi_{2,j} Q_{k,j} + (P_k) + F_{fat,d} \quad (3.48)$$

Variable actions of the same type as the fatigue action should not be considered in the part $\Sigma \psi_{2,j} Q_{k,j}$.

This applies if the absolute value of the load effect or if the mean load effect is of importance for the fatigue verification. Depending on the type of sensitivity of the model with respect to the mean value of the stress cycle it may be necessary to consider the mean value of fluctuation load per cycle and not the life time quasi static value.

3.9.1.5 Reflection

Fatigue in welded steel or aluminium structures or reinforcement is a process where initial defects at the micro level develop into visible cracks that may reduce the load bearing capacity of the joint. During a moment or period of heavy loading the weakened structure may come to yielding and/or to instable fracture. In that sense, splitting up the reliability verification into a standard ULS verification and a fatigue verification using the SN-approach is a simplification that might be on the unsafe side. On the other hand, it is believed that the parameters in the SN-model based on small specimen are on the safe side. Since the majority of the life is in the initiation and growth of small cracks, the crack size at final fracture is relatively unimportant for the total life, provided that the material satisfies the toughness rules of EN 1993-1-10. This situation becomes different when evaluating fatigue inspections.

The probabilistic S-N curve as given in Eq. 3.42) is a simplified approximation. More accurate but more complicated descriptions are given in literature, e.g. (D.Leonetti et al. 2017, 2020). Using such descriptions often results into lower partial factors (Maljaars, 2022). The partial factors in EN 1993-1-9 are based on these more accurate and complicated descriptions.

Models that describe the process of crack growths and subsequent fracture are available and should be preferred from a reliability point of view, in case fatigue inspections apply. A problem in practical application is that keeping track of all local geometrical details and spatial variability is an almost impossible exercise.

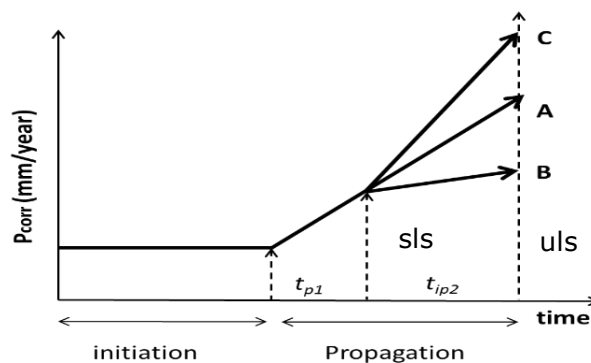
See Annex B for further information on crack growth models and examples, also for other materials.

3.9.2 Durability Verification Concrete

3.9.2.1 Limit state formulation and target reliability

Under normal conditions concrete protects embedded reinforcing steel against corrosion. However, environmental actions such as carbonation or penetration of chloride ions disrupt the protective properties of the passive layer and may lead, over time, to corrosion of reinforcing steel. This stage until corrosion is active is called the initiation period.

Figure 3.27. Schematic presentation of degradation with time.



After the start of corrosion (the so-called propagation period) its effects on a RC structure usually include cracking of the concrete cover, reduction and eventually loss of bond between concrete and corroding

reinforcement and finally reduction of cross-sectional area of reinforcing steel. So, it may lead to the exceedance of SLS (loss of appearance) or in combination with mechanical actions to exceedance of the ULS.

As identifying the onset of corrosion is quite difficult, because very early corrosion does not induce any external sign and the concrete properties are not affected, proxy limit states or condition limit states (CLS) (see also ISO 2394, *fib*-bulletin 34 (2006), *fib*-model-code 2010, Brite EuRam III, 2000) are introduced like for instance the “durability/deterioration limit state” (DLS) which considers the corrosion penetration attack until 50 µm for homogeneous corrosion and 500 µm for pit corrosion as not impairing the structural reliability assumed in the design. Note that if a *condition limit state* verification is used, it does not replace the more complex serviceability of ultimate limit state verification, the targets should be chosen accordingly.

Having this in mind the following typical limit states associated with corrosion of reinforcement may be distinguished:

- De-passivation and onset of corrosion;
- Development of 50 µm-homogeneous attack or 500 µm pit depth which may result into a micro crack (say < 0,1 mm) parallel to the bar in concrete surface (referred to as Durability or Deterioration Limit State);
- Crack associated with appearance, say 0,3 mm in width (Serviceability Limit State);
- Loss of bond;
- Actual loss of (local) load bearing capacity (Ultimate Limit State).

The target reliability required for each of these limit states in general will be stricter when going from depassivation to the ultimate limit state, roughly climbing from for instance $\beta = 1,3$ for de-passivation and 1,5 for the DLS to 3,8 for the ULS (design working life values).

3.9.2.2 Initiation and propagation models

Corrosion due to carbonation may start when the depth of the carbonation front is larger than the cover depth. So the LSF for the start of corrosion may be defined as:

$$g(t) = c - x_c(t) \quad (3.49)$$

where:

c is the concrete cover [mm];

$x_c(t)$ is the carbonation depth [mm] at the time t ;

t is the period of time [year] under consideration.

The value of x_c depends on a diffusion process of CO_2 and depends on many geometrical and material parameters as well as the outside environment (*fib*, JCSS, EN 1992). Note that given (3.47) we may calculate the probability of depassivation for every point in time. Taking $t=t_d$ (where t_d is the design working life) we may find the failure probability $P\{g(t_d)<0\}$ for passing this limit state for the design service life.

The limit state function for the initiation of corrosion due to penetration of chloride ions can be formulated as:

$$g(t) = C_{\text{crit}} - C(t) \quad (3.50)$$

where C_{crit} is the critical chloride content with respect to the binder to achieve depassivation of the reinforcement and $C(c,t)$ is the chloride content at the level of the reinforcement and time t .

The propagation model is based on onset, rate and accumulation of damage. Propagation models include the effects of corrosion on structural behaviour, like cracking, spalling and loss of mechanical strength. Important is the spatial distribution of cross-sectional loss. This is crucial for modelling pitting corrosion. In general, the limit state function at time t can be expressed as (*fib*-model-code 2010, Val et al 2019):

$$g(t) = p_{crit} - V_{corr} (t - t_{dep}) \quad \text{for } t > t_{dep} \quad (3.51)$$

where p_{crit} is the critical corrosion penetration for the limit state under consideration (e.g. spalling or loss of bending strength), V_{corr} is the speed of corrosion, t is time and t_{dep} is the starting time of the depassivation state. Note that $P\{g(t_d) < 0\}$ is the probability of reaching the limit state within the design service life.

All models include quite a number of parameters (concrete quality, cover exposure conditions, etc.),

3.9.2.3 Reliability verification for the ULS

For the ULS of a beam under bending the Limit state function may be given by:

$$g_{ULS}(t) = \theta_{MR} M_R(t) - \theta_{ME} M_E \quad (3.52)$$

where θ_{MR} is the model uncertainty of the resisting bending moment; $M_R(t)$ is the resisting bending moment at time t ; θ_{ME} is the model uncertainty of the load-induced bending moment and M_E is the bending moment induced by the applied loads:

$$M_E = 0,08 (g+q) L^2 \quad (3.53)$$

For simplicity we take conservatively the load effect M as the maximum value in the life time. In case of possible degradation on the desistance side the resisting bending moment $M_R(t)$ includes the reduction in the reinforcement area:

$$M_R(t) = n A_s(t) f_y h_{eff} \quad (3.54)$$

where $A_s(t) = (\pi/4) d(t)^2$ with $d(t)$ the time dependent diameter of the reinforcement bars at time t given by (assuming uniform corrosion, no pitting):

$$d(t) = d_0 - 2\Delta d(t) = d_0 - 2(t-t_{init}) V_{corr} \quad (3.55)$$

where d_0 is the initial diameter and $\Delta d(t)$ is the corrosion loss, V_{corr} is the time average corrosion rate; t_{init} is the time to initiation, which follows from $g(t_{init}) = 0$ but of course keeping $\Delta d(t) > 0$. A further numerical elaboration can be found in Annex B.

In present practice such an analysis is usually not carried out, not even in a semi-probabilistic approach. The above combined analysis of chemical and mechanical hazards is normally split up into two separate verifications: one for the uncorroded structure and one for the corrosion initiation. The verification of the uncorroded structure is based on $\beta = 3,8$ for 50 years or some equivalent annual target, For the corrosion initiation target the target is not specified (deemed to satisfy rules are used) or taken quite loosely as $\beta = 1,3$ or $1,5$ for 50 years.

In cases having an aggressive environment and little possibilities for inspection this might be quite dangerous. Note also that in addition to the direct effect of rebar reduction we may have to deal with loss of steel ductility, loss of bond and reduction in concrete cross section. In those cases, in design as well as assessment, extra attention compared to standard procedures may be justified.

3.10 Robustness

3.10.1 Introduction

Robustness is defined as the ability of a structure to withstand unforeseen adverse events without being damaged to an extent disproportionate to the original (unforeseen) cause. The term unforeseen refers to actions and influences not defined in quantitative terms. The basic requirement for robustness is formulated in clause (4.4) of EN 1990:2023 and some guidance on design strategies and design methods is given in EN 1990:2023 Annex E (informative). Further elaborations can be found in EN 1991-1-7 and the material related Eurocode's. These elaborations however still give rise to much discussions due to different opinions on a general design philosophy. This section in combination with Chapter 5 is intended to give some further guidance to the developments in this field.

Design for robustness should be an integral part of the conceptual design phase of the structure, taking into account different strategies that can be applied. In general, the design of a structure for robustness can involve the – explicit or implicit – identification of:

- Hazards H_i ;
- Local (direct) damage D_j ;
- Systemic damage S_k (follow-up/indirect damage), encompassing progressive collapse;
- Direct and indirect consequences.

In accordance with EN 1990:2023, robustness is primarily intended to be used with respect to unforeseen events, actions and influences (unspecified or really unknown hazard) but may also prove to be an adequate design strategy for actions which the code does define in quantitative terms (usually termed "identified actions").

According to EN 1990:2023 the occurrence of (gross) human errors should be counteracted by adequate Quality Assurance procedures as specified in Annex B. Nevertheless, still human errors can occur and a proper degree of robustness is then helpful in mitigating the consequences. At the moment, however, no numerical operational methods are available.

3.10.2 Design strategies to prevent unacceptable damage

It is appropriate to underline that unacceptable damage can be prevented by adopting design strategies which do not necessarily increase the robustness of the system. This can be clearly be concluded by analysing how (structural) risk can be determined.

In a risk-based context, the expected value of risk can be determined based on the following equation according to Annex B in EN 1991-1-7 (see also chapter 5):

$$\text{Risk} = \sum \sum \sum C_{ijk} P[S_k | D_j, H_i] P[D_j | H_i] P[H_i] \quad (3.56)$$

where:

$P[H_i]$ represents the probability of occurrence of hazard H_i ;

$P[D_j | H_i]$ represents the probability of initial damage D_j conditional on hazard H_i ;

$P[S_k | D_j, H_i]$ represents the probability of follow-up damage S_k conditional on damage D_j and hazard H_i ;

C represents the consequences.

Reducing any of the four factors can be considered as a design strategy to control risk levels.

The design strategies preventing unacceptable damage have been largely investigated in literature (Ellingwood & Leyendecker 1978, Gross & McGuire 1983, Dusenberry & Juneja 2002) and they can be categorised as follows:

Design strategies which prevent the *occurrence of abnormal and dangerous events* (i.e. influencing $P[H_i]$ in equation 3.56). This strategy aims at reducing the exposure of the structure to hazards, by reducing the probability of occurrence and/or the intensity of abnormal and hazardous events. Examples of such strategies include the use of safety barriers against impact or base isolators to limit the intensity of a

seismic action.. Event control strategies commonly do not lead to a change in structural robustness, in particular in cases when the structure's tolerance to damage (or progression thereof) is uncorrelated with the event control strategies used, e.g. non-structural measures such as impact barriers

Design strategies which prevent the *occurrence of an initial damage* (i.e. influencing $P[D_i|H_i]$ in equation 3.56). In general, this strategy aims at identifying and designing key members by improving the member behaviour through sufficient resistance and energy dissipation capacity. It is to be noted, however, the key member design approach does not necessarily enhance the robustness of the system, unless the member changes are correlated with the system robustness behaviour. In addition, problems may arise if unidentified adverse events or actions occur, which are not covered by the design of key members, neither against the identified, variable or accidental, nor the notional actions. Further weak points of the strategy lie in the process used to identify the key members themselves and the reference to a tolerable damage limit. When key members are being "identified as those whose absence would result in damage that is greater than a tolerable limit" [EN 1990:2023 E.4(4)], individual member removal seems to be assumed, with no consideration of scenarios with more than one member being damaged under an extreme event. In addition, the term "tolerable limit" is vague and does not imply guidance for the practicing engineer [CEN/TC 250/WG6.T2].

Design strategies which *prevent unacceptable spreading of damage* (i.e. influencing $P[S_k | D_i \cap H_i]$ in equation 3.56). This strategy is aimed at the global system behaviour so that the spread of the initial damage remains limited. Two alternatives are typically considered:

The alternative load path approach which redistributes the forces originally carried by the failed members. However, if the initial damage produces an overloading in the remaining structure, the alternative load path procedure may promote progressive collapse and reduce the robustness of the system. Thus, robustness design by providing alternative load paths may require a higher load bearing capacity for the remaining structure than that needed for persistent and identified accidental design situations. Since, in principle, any member may fail due to unidentified accidental actions, an uneconomic design is a possible outcome of such a strategy. In this regard, as with the key member design strategy, open questions also relate to the need to consider or not consider scenarios where more than one member is damaged under an extreme event.

Structural segmentation through which the spreading of failure is prevented or limited by isolating the initially damaged portion of the system. This strategy has advantages of being able to define by design the extent and location of the failure regions, but this comes also with some disadvantages. Structural segmentation typically involves designing structural fuses, e.g. weak interfaces, and favours dividing the system in statically determinate sub-systems. When exposed to persistent, transient and identified accidental situations these sub-systems will exhibit increased risks when compared with adequately designed and detailed indeterminate solutions. Structural segmentation aims to reduce risks globally, but in the process may increase risks locally. A key issue is therefore how to fine tune the design details so that adequate robustness and risks levels are attained. Note that indirect design methods are currently missing for segmentation, and that design of statically determinate sub-systems may involve increased requirements for structural safety and serviceability and thus can lead to a higher material consumption compared to other strategies

3.10.3 Design methods for robustness

Design for robustness should be based on the Consequence Class (CC) of the structure, as given for example in Table 3.13. The table is intended as a possible inspiration for the NBS's.

The specificities of each design approach can be as follows:

For CC1 structures no particular robustness provisions are required if the structure is designed according to the partial factor design philosophy specified in the Eurocodes for persistent, transient and seismic design situations and the failure consequences are acceptable. Note that the latter should result from using methods given in Chapter 5.

For CC2 structures two groups can be defined, respectively CC2a and CC2b according to different levels of consequences. Both refer to similar groups of structures but exhibit different levels of magnitude of failure consequences. Therefore, different levels of sophistication of the analysis and design methods should be applied. A possible discrimination between the two classes can relate to:

When using indirect design methods for CC2a, typically only horizontal tying is involved whereas for CC2b vertical tying is required in addition.

When using direct design methods for CC2a typically simpler analysis requirements are involved, for example, linear elastic models may still be appropriate, whereas for CC2b non-linear analyses are often deemed necessary.

Note that although it should be required that the reliability of the designs achieved using either direct or indirect design methods should be comparable, the current robustness provisions are not generally associated with a specific target level of reliability. Alternatively, indirect design methods may be used as baseline (minimum) for robustness requirements.

For CC3 structures, additional explicit identification and enhanced quantitative characterisation of adverse events are typically considered necessary, involving, in particular, for existing structures the use of more complex reliability-based methods. Member states may provide rules for the additional use of qualitative or quantitative risk analysis in special cases.

Table 3.13. Relationship between Consequences Classes and Robustness design approaches (example)

Consequences Classes	Robustness design requirements	
	Analysis method	Design method
CC1	No particular provisions are necessary with respect to robustness.	
CC2a Lower Risk Group	Analyses can be based on simplified models of loads and structural behaviour.	Design for robustness using direct and/or indirect design methods.
CC2b Higher Risk Group	Simplified models still possible. Dynamic and/or non-linear analysis models may become relevant. Analyses may however be based on simplified models of loads and structural behaviour.	
CC3	Additional explicit identification and enhanced characterisation of adverse events. Level II to IV reliability methods including qualitative/quantitative consideration of possible risks where appropriate and (as stated in EN 1990:2023, Table E2) specified by the relevant authority or agreed upon for the specific project by the relevant parties	

It is highlighted that design for robustness consists of the identification and assessment of design scenarios. Herein, each design scenario relating to a set of events or conditions occurring during the execution or lifetime of the structure leading to a state of the system for which the potential for attaining unacceptable consequences should be assessed. Design scenarios depend on the type of design method to be considered, ranging from realistic event sequences to notional damage scenarios.

In general, the rules for accidental design situations are applicable; however, in some cases persistent design situations should be considered, in particular for cases where permanent actions control the design.

In addition, it is noted that the design solution obtained using direct and indirect design methods is not the same. The former may involve a design focused on the system scale whereas the latter focuses, in general, only on the local scale. One does not replace the other. The same applies to hazard dependent and hazard independent scenarios (e.g. identified and unforeseen hazards, respectively).

Therefore, in many cases, a suitable way for economically meeting all structural safety and robustness requirements should be based on a combination of different design strategies, as appropriate. For this purpose, a pragmatic approach can be adopted implying continuous structural systems with a ductile behaviour;

-For hazard independent scenarios, provision of either alternative load paths via direct or indirect design methods, or, predefined collapse mechanisms for the event of local failure of selected key members, simulated by notional scenarios (as noted above, indirect design methods are currently missing for segmentation);

-For hazard dependent scenarios, provision of alternative load paths via direct or indirect design methods

When using the alternative load path strategy, check if the associated failed area can be deemed acceptable. If not, repeat until the latter is verified. Research was performed by (Tanner and Hingorani, 2015) which can be used to define a acceptance criteria based on the failed area.

3.10.4 Reflection

Even before the well-known Ronan Point tower building accident in 1968, structural integrity design requirements for ties were included in CEB bulletin 60 concerning precast panel constructions. A complete historical review of prescriptive design rules for robustness can be found in (Russell et al, 2019). The “tying force” rules included in many codes are inspired in these provisions. There is still ongoing debate on whether the objective of these tying forces was just to aid in keeping the structural shape of precast panel constructions or to prevent local failure and maintaining integrity after a member loss. What is clear that there is no available evidence that rules developed for precast panels construction still apply to modern construction methods. Furthermore, although the original intention of the ties was to prevent brittle failure, this is not explicitly checked and therefore it is not certain that the tie, or secondary mechanism can form without further failures occurring. Advanced structural analysis in combination with experimental evidence may help to come to a better understanding.

Finally note that current design loads to design key members (i.e. pressure load of 34 kPa) have not been derived from case studies which allow for their generalised application. It makes sense to keep this in mind. In any case, it appears reasonable to include the consequences of key member failures in the considerations..

3.11 Observational methods, design assisted by testing and monitoring

Ideally all structural design models, both on the action and on the resistance side, are based on the laws of mechanics and, where relevant, on other laws from the world of natural sciences. These laws have been validated in the past on the basis of numerous experiments, both on a fundamental level (first principles) and on an applied level (buckling curves, shear models). The same holds for the values adopted for the variables in the models. This whole set of experiments and observations is the basis for the models and design values in the codes.

In some cases, the available set of data may be considered as insufficient, for instance for innovative materials) and a new set of experiments is set up. Annex D of EN 1990:2023 (Design by Testing) gives guidance to that process and gives formulas on how to deal with the test results compatible with the Eurocode design philosophy. A main topic in this respect is the treatment of the scatter in the test results, in particular in the situation of a small number of tests (statistical uncertainty). The procedure in Annex D is, in line with the rest of EN 1990:2023, based on the Bayesian interpretation of probability where no distinction is made between the various sources of uncertainty. In practice this means for instance that there is no need to choose, in addition to the target reliability, separate confidence intervals for the estimates of means and standard deviations as has to be done in classical frequentist approaches.

The observational method is a design approach with testing/observations not only before but also during the execution or early stage of use. It is already used for quite a long time in geotechnical engineering (Peck (1969) and later modified as it formally entered EN 1997. However, the basic idea can be and are also applied to other types of structures. An example is the application of vortex limiting devices: they are only applied if practical behaviour show the need.

The essence of the approach is to design a structure based on an estimate of the structural or ground properties less cautious than the design values in partial factor design (e.g. best estimate values), monitor the behaviour of the structure during or after construction and to have prepared contingency measures to be taken if the encountered properties deviate negatively from the expectation. A key aspect of the approach is to assign trigger values to the observed parameters for the implementation of contingency measures. Such decision rules establishing the triggering of prepared contingency measures have to produce a safe margin to structural failure during the execution phase and ultimately have to ensure sufficient reliability of the finalized structure.

When the method is applied the reliability level in the final state would be insufficient if based on the large uncertainties present at the beginning. However, during the process of execution the reliability level can be

updated using Bayesian procedures. (e.g. Spross & Johansson, 2017). The effect of updating and, hence, the effectiveness of the approach will depend largely on the degree to which uncertainties in time-invariant variables (e.g. ground properties or permanent loads) dominate the design problem. The objective is to meet the formal reliability requirements in the final state of the structure, using the information obtained during execution (and early use) to update (reduce) uncertainties in the structural or ground properties. Contingency measures should be applied when necessary. In order to achieve this a risk analysis at the start of the project is recommended.

Next to the reliability of the final product also the reliability in the various phases of construction (for instance related to the safety of human lives) can be important. In principle such requirements can be treated using the similar methods as discussed above.

Note that standard inspection and monitoring activities during the entire life time of the structure are based on the same principle as design by observations. The basic issue, however, in that case is not the almost immediate behavior but the long term behavior, including all kinds of deteriorating influences.

Finally it is important to realize that it may be important to have in advance a clear understanding on the procedure with the client and the relevant authorities.

4 Assessment of Existing Structures

4.1 Motivation

Current and future activities related to the development of cities, industrial areas and infrastructures should be determined by sustainability goals (JRC 2015). New needs are therefore not simply answered by adding new buildings and infrastructures to the existing built environment or by replacing existing with new engineering works. Rather, ways are being explored to modify existing systems to meet new demands, or simply to extend their service life. For this purpose, the reliability of existing structures must be verified, activity that is usually denominated as assessment [CEN/TS 17440 2020].

In general, an assessment of an existing structure may be required in the case of (ISO 2394 2015):

- a change in the purpose of the structure compared to that for which it was originally designed or previously assessed;

- deviations in the properties of the structure from those adopted in the original design or in the previous assessment.

Examples of changes in the purpose of a structure include:

- change of use;

- increase of actions;

- extension of the originally intended design service life;

- increase in reliability requirements (e.g. due to higher consequences in case of failure).

Deviations in the properties of a structure can be due to the following causes, among others:

- modifications of the structural system (e.g. extension, upgrading, repair);

- deterioration induced by time-dependent actions and influences (e.g. corrosion, fatigue);

- damage induced by accidental events, overloads, changes of the boundary conditions (e.g. differential settlements);

- defects resulting from previously undetected errors during design, construction or use.

The general principles of structural reliability laid down in the Eurocode for structural design (EN 1990; 2023) and of course also in chapter 3 of this document also apply to the assessment of existing structures. However, there are also important differences between assessing existing and designing new structures. The use of design-oriented methods to assess existing structures therefore often leads to a high degree of conservatism with serious economic, ecological and socio-political consequences if resources are invested in the unnecessary strengthening or replacement of existing structures with the associated disruptions (JRC 2015). This chapter briefly discusses relevant differences between assessment and design. In the Eurocode system, these differences are addressed in the Technical Specification (CEN/TS 17440 2020). The conversion of this document into an additional Eurocode part “prEN1990-2 Basis of structural and geotechnical design – Part 2: Assessment of existing structures” is under development.

4.2 Scope

The main difference between assessing the performance in existing and design phase structures is that many characteristics whose values are merely anticipated in the latter can be measured in the former, taking also into account the effects of the construction process and subsequent life of the structure, during which it may have undergone alteration, deterioration, misuse and other changes to its as-built state. The accuracy of the assessment results obtained by applying load and strength models can usually be improved by collecting more data about the structure under analysis and about the actions and influences to which it is exposed (see 4.4 and 4.5). This does not mean, however, that the uncertainties can be completely resolved: in-service inspection and testing are also associated with uncertainties (ISO 2394 2015). Therefore, assessment is conducted by stages (CEN/TS 17440 2020), raising the quality of the information available from stage to stage where needed. Note also that at the start of an assessment the uncertainties may be even greater than in design for instance as the consequence of (unexpected) damage or deterioration.

Another relevant difference between the design of new structures and the assessment of existing ones concerns the level of reliability required (see 4.3). The choice of this level depends, among other parameters, on the amount and expense of effort needed to reduce the risks associated with a structure. The cost of achieving a higher level of performance (i.e. increased safety) is usually high in existing structures compared to the cost of improving the same performance in the design phase of a new structure. This aspect should be considered when choosing the level of reliability required for an existing structure (JRC 2015), along with the influence of the remaining service life. Note that in principle a minimum reliability level related to human safety resulting from ethic reasons should not be lowered.

Deterioration due to environmental influences, repeated actions or use-induced wear, is typically a cumulative process that can adversely affect structural reliability. Differences between existing and new structures also evolve from the treatment (not in principle but in practice) of the possible interactions of cumulative deterioration and extreme action effects (see 4.6).

Although the detailed design of any engineering structure requires multiple iterative steps, the ultimate result is a clear definition of the type, layout and dimensions of the load bearing system and its individual elements and details, as well as an appropriate selection of construction materials. This can be fundamentally different in an existing structure in which different approaches may be advisable, depending on the results and conclusions from the assessment with regard to the relevant requirements for the system (see 4.7). Appropriate recommendations should be formulated to the owner, taking into account the conditions for possible interventions and future operation of the construction work. Such recommendations can therefore include constructional or operational intervention measures that belong to different categories.

4.3 Target reliability levels

Significant aspects to consider when determining the target value of the reliability index for the assessment situations relevant to an existing structure include (CEN/TS 17440 2020):

- the possible cause and the mode of reaching a limit state (as far as they are relevant for the consequences of failure);
- the possible direct and indirect consequences of failure in terms of risk to life, injury, potential environmental and economic losses, social and political consequences, loss of cultural heritage value, etc.;
- the relative cost of safety measures to increase reliability (as long as the level is not governed by minimum human safety aspects);
- the reference period.

The acceptable reliability levels for existing structures can be different from those required for new structures. The following types of considerations can justify lower levels for existing structures:

- economic: the relative cost of safety measures to increase the reliability of an existing structure can be very high, while the additional cost of increasing the reliability in the design phase of new structures is generally low;
- societal: the strengthening or replacement of existing structures can lead to the resettlement of residents, the interruption of activities or may influence the values of cultural heritage, circumstances that normally do not play a role in the design of new structures;
- sustainability: sustainability goals are of fundamental importance when deciding to extend the service life of existing buildings and infrastructures, or when adapting such systems to new needs, as this implies a reduction in resource consumption compared to replacement with new structures or structural elements, likewise, the rehabilitation of existing structures usually allows the selection of the most suitable solutions and materials from the point of view of sustainability.

Target reliability levels can be derived based on explicit risk analysis (see chapter 5) or economic optimization, meeting acceptable human safety levels, e.g. with regard to current best practice (EN 1990-2:2021). When selecting target reliability levels, it should be taken into account that the intended remaining service life of an existing structure often may be shorter than the design service life of new structures. This is in particular relevant for cases where consequences of failure are dominated by economical losses. Note that the reference period to which the target reliability is related can be selected regardless the anticipated remaining service life. If annual target reliabilities are used, the corresponding structural performance should be achieved in each year

of the remaining service life. This is particularly relevant in the case of deteriorating structures in which the final year of the remaining service life is decisive (see 4.6).

The target reliability levels for new structures can in most cases be considered conservative, when used for the assessment of existing structures due to the aspects discussed above. Note that the final responsibility for setting target reliability levels and corresponding safety elements is a on the national level and should be mentioned in the relevant National Annex. Chapter 5 given some guidance in making appropriate choices.

4.4 Updating information

4.4.1 General

The acquisition of new data about an existing structure by means of inspections, measurements or tests is intended to supplement the available prior information, which may often be quite vague, with respect to aspects such as geometrical properties, actions and influences, construction material and geotechnical properties, as well as the actual condition of the structure, its behaviour or deformation capacity (CEN/TS 17440 2020). When new information becomes available, all relevant data need to be evaluated, taking into account the existing prior information. This process is known as updating of information, which is one of the main tasks of any assessment. In addition to the rules given in the Eurocode on basis of assessment (CEN/TS 17440 2020; prEN 1990-2, 2021), the updating of information is also covered in the assessment and retrofitting part of the Eurocode on earthquake resistance (prEN 1998-3 2021), in particular in its chapter 5 on information for structural assessment.

Two complementary approaches can be considered to update information about the properties of a structure and its performance under the actions and influences to which it is exposed (ISO 2394 2015):

- the updating of the probability of structural failure by using information from load testing or about the performance of the structure in the past;

- the collection of data on individual basic variables by performing on-site inspections to update previously available uncertain information.

4.4.2 Updating the failure probabilities

Formally, the direct update of the probability of failure can be carried out using fundamental relationships from probability theory (ISO 2394 2015; prEN 1990-2 2021), together with the information that an existing structure or structural component has withstood an applied load effect. In practice, load effects are known in the case of load tests or, to a lesser extent, if the maximum loads applied over the past service life can be estimated.

In this context it should be noted that the observation that a structure or structural component has withstood a load test does not directly reveal its actual resistance, nor does it provide a direct measure of structural reliability. Such an observation only indicates that the minimum resistance of the structure at the time of the test is greater than the effect of the applied load, allowing for the uncertainties associated with the conversion from loads to load effects. Therefore, to update the probability of failure of a structure on the basis of a known load, the original cumulative distribution function for the structural resistance used for this purpose can be cut off at the level of the known deterministic level. In the more general case a more advanced procedure may be needed, for instance , allowing for the relevant model uncertainties including those associated with the conversion from an action to its effects (i.e. depending on measurement uncertainties or other less known effects the cut off may be sharp or gradual). If no prior knowledge on the resistance is available, the probability of failure may (conservatively) be assessed based on the effects of the known action and its statistical distribution.

Similarly, satisfactory performance of a structure over a period T of service indicates that in the absence of significant degradation, its minimum resistance is greater than the maximum (random) load effect applied over that period. To estimate the probability of failure, the distribution function for the structural resistance can be updated taking into account the estimated random action effect over the same period T . Note that if there is deterioration the analysis becomes more cumbersome.

Additional measurements and inspections may reveal more information of the structural properties. This information, of course, can be taken on board in the updating procedure. If there is for instance a well-established degree of correlation between stiffness and strength, information on the stiffness may be used to update the distribution of the resistance.

4.4.3 Updating the basic variables

When updating the probability distribution function of a basic variable, X_i , its parameters (e.g., mean value, standard deviation, coefficient of skewness, lower bound, etc.) may be considered as random variables (JCSS 2000; ISO 2394 2015; prEN 1990-2 2021). Prior distribution functions for the unknown parameters of the investigated variable should reflect all the information available before the in-service data acquisition is carried out. For example, distribution parameters that represent the state of uncertainty associated with the Eurocode design rules, compiled in Section 3.4 and Annex A, can often be used as a starting point for updating. Given such prior distributions and statistical data from new observations, posterior distributions can be derived, e.g. by (repeatedly) applying a Bayesian method (JCSS 2000). In general, the following assumptions are appropriate for the type of distribution in most applications (no difference with the design situation):

for dimensions and material properties, respectively a Gaussian and a log-normal distribution may be adopted;

a Gaussian distribution may be appropriate for permanent action effects;

an extreme value distribution may be convenient if it is intended to represent a maximum value within a chosen reference time (e.g., effects due to variable or accidental actions).

The choice of probability distribution functions should be made with caution, considering possible bias and skewness. The coefficients of asymmetry and kurtosis may provide valuable information for determining the appropriate theoretical model (i.e., probability distribution function). Note also that variables originally uncorrelated may get correlated after updating; in that case updating the marginals is not sufficient. It is also important to note that any method used for inspection, measurement or testing has a limited resolution. Uncertainties associated with such methods include (ISO 2394 2015):

measurement error;

inherent variability of a measured parameter;

model uncertainty when a parameter cannot be measured directly so that a relationship is needed between it and the corresponding measured parameter;

statistical uncertainty due to a limited number of measurements.

These uncertainties should be taken into account when determining the statistical data that describe the measured parameter. This can be achieved, for example, by increasing the coefficient of variation of the corresponding parameter (prEN 1990-2 2021). In this context, it should also be taken into account that the uncertainties associated with non-destructive testing methods are generally higher than with destructive methods. For assessing the earthquake resistance of existing structures, EN 1998-3:2021 accounts for the above uncertainties by using partial factors γ_{Rd} that depend on the amount of information gathered during the assessment, as described in discrete terms by so-called Knowledge levels (KLs), and are derived considering the inherent variability, the statistical uncertainty as well as the model uncertainty for each resistance or deformation capacity formula in the code.

4.5 Verification

4.5.1 Overview

Similar to the design of new structures, when assessing existing structures, it should be verified that with an appropriate reliability level no limit state is exceeded for all relevant assessment situations (prEN 1990-2 2021), verifications can be performed using different methods:

- quantitative assessment based on calculations;
- qualitative assessment based on past performance;
- combination of both types of methods.

If qualitative methods are used to assess a structure at the relevant limit states based on satisfactory past performance over a sufficiently long period of time, a detailed structural investigation should be carried out to show that a number of specific conditions (e.g., no damage nor deterioration, no changes to the structure or the environment, etc.) are met. Note that there is no reliability background for qualitative assessment methods. On the other hand (see 4.4.2), the available information about the past performance of a structure can also be

used in a quantitative assessment, for example, for direct updating of failure probabilities or for resistance updating based on action effect histories.

The most accurate way of assessment would be to explicitly consider updated load and strength variables through the use of reliability methods or risk-based decision procedures. However, such methods and procedures are time-consuming, calling for a specific operational knowledge of probabilistic methods, and are only used in special cases. For example:

- if, using the partial factor format, it cannot be demonstrated that the structure or element achieves the required reliability;
- when uncertainties are outside the usual ranges;
- in cases with severe failure consequences or insufficient robustness;
- for decisions regarding a whole group of similar structures (e.g. calibration of partial factors);
- when evaluating the efficiency of different options for intervention (e.g. monitoring and maintenance strategies).

To verify whether an existing structure meets the relevant reliability requirements for all assessment situations, the partial factor format is normally used, equivalent to the method specified for structural design (prEN 1990 2020). The difference is that the relevant parameters, including the characteristic values of the basic variables and partial factors, can be modified based on updated information (prEN 1990-2 2021). A particular form of additional information is a proof loading (see 4.4.2). Such tests, as well as study of track records, may reveal implicitly a part of the hidden safety (see 2.2.3). When elaborating the results it is important to distinguish between conclusions related to the structure (or structural member) that was actually proof loaded and other (similar) structures or members.

4.5.2 Partial factor format

When using the partial factor format, the assessment values for the action effects, E_a , are compared to the assessment values for the respective strengths, R_a . The so-called assessment values (prEN 1990-2 2021) are equivalent to the design values for new structures (prEN 1990 2020). Structural reliability requirements are met if, similar as in design, inequality 4.1 is fulfilled for all relevant assessment situations:

$$E_a \leq R_a \quad (4.1)$$

where E_a and R_a can be expressed as functions of the assessment values of the basic variables like in design.

Resistance models that cannot be used for the assessment of all existing structures should be adjusted so that the effects of structural condition, including deterioration on the load bearing capacity can be modelled explicitly. Such assessment-specific provisions should be developed for future editions of the material-oriented Eurocodes (CEN/TC 250/WG2/WG2.T2 2021). If resistance models for the design are adjusted or expanded in their scope, in order to enable the assessment of existing structures, partial factors for resistance, γ_{Rd} , should cover the degree of uncertainty associated with these models.

Note that the observation of a certain degree of deterioration should not only result in a modification of structural dimensions or properties. It may also be necessary to adjust other parameters as for instance mean or extreme load values. The mere fact that the structure was affected more than foreseen could be an indication that some actions (or environmental influences) were estimated too low. Similar adaptations may be necessary for structural or geotechnical parameters.

4.5.3 Assessment values of the basic variables

Provided that the basic variables relevant for the reliability verification of an existing structure are specified by suitable probability distribution functions (see 4.4.3), their characteristic values, x_{ki} , the associated partial factors, γ_{xi} , and the assessment values, x_{ai} , can be determined. By way of example, consider a log-normally distributed resistance variable x_i , associated with a non-deteriorated structure, with updated mean value, m_{xi} , standard deviation, σ_{xi} and coefficient of variation, V_{xi} . The updated characteristic value, x_{ki} , is obtained as the

lower 5% fractile of the updated probability distribution function of x_i . If we use, the Design Value Method, we may derive for coefficients of variation, $V_{xi} \leq 0,25$, the updated partial factor, γ_{xi} , from:

$$\gamma_{xi} = \frac{x_{ki}}{\mu_{xi}} \cdot e^{\alpha_R \cdot \beta_t \cdot V_{xi}} \quad (4.2)$$

where β_t is the target reliability index and α_R the FORM sensitivity factor for resistance. For the latter, the value recommended in the Eurocode (prEN 1990 2021) may be used as a first approximation if the respective application limits are met. If not, as the uncertainties and relative importance of variables for existing structures may differ from those in the design phase, α_R could be determined by reliability analysis or could be adopted conservatively (e.g., $\alpha_R = 1$). Finally, the assessment value of the same variable, x_{ai} , to be applied in the reliability verification of the existing structure analysed, is obtained from:

$$x_{ai} = \frac{x_{ki}}{\gamma_{xi}} \quad (4.3)$$

Similarly, for a normally distributed action variable, x_i , with updated parameters μ_{xi} , σ_{xi} and V_{xi} , the updated partial factor, γ_{xi} , can be obtained from Formula (4.4). As in the previous case, the value recommended in the Eurocode (prEN 1990 2021) for the FORM sensitivity factor for effects of actions, α_E , may be used as a first approximation (but the rules for application should be checked):

$$\gamma_{xi} = \frac{\mu_{xi}}{x_{ki}} \cdot (1 - \alpha_E \cdot \beta_t \cdot V_{xi}) \quad (4.4)$$

The updated assessment value of the action variable, x_{ai} , can be derived as follows from the updated characteristic value, x_{ki} :

$$x_{ai} = \gamma_{xi} \cdot x_{ki} \quad (4.5)$$

For permanent actions, the updated characteristic value, x_{ki} , is normally considered to be equivalent to the updated mean value, μ_{xi} .

For the effects of variable actions with a Gumbel distribution (e.g. imposed loads, climatic actions), assuming that the uncertainties associated with the time-invariant part of the action model (see 3.4) are not relevant, the updated partial factor, γ_{xi} , may be determined on the basis of formula (4.6), where Φ is the cumulative distribution function of the standardised normal distribution, σ_{xi} represents the updated standard deviation and the other parameters correspond to those of formula (4.4). As already mentioned (see 4.3), all these parameters should relate to the chosen reference period. In the absence of a specific reliability analysis, the sensitivity factor, α_E , may again be approximated by using recommended values (prEN 1990 2021), if the application limits are met. Finally, the updated assessment value of the action variable, x_{ai} , is obtained by applying formula (4.5). The latter also applies to the general case, if the uncertainties associated with the time-invariant part of the action are relevant, although the updated partial factor cannot be represented with a closed formula.

$$\gamma_{xi} = \frac{1}{x_{ki}} \cdot \left(\mu_{xi} - \sigma_{xi} \cdot \frac{\sqrt{6}}{\pi} \cdot \left(0,577 + \ln \left(- \ln \Phi \left(- \alpha_E \cdot \beta_t \right) \right) \right) \right) \quad (4.6)$$

The above examples of updated partial factors and assessment values take no account of additional effects, e.g. due to model uncertainties. Further information on general procedures for taking such uncertainties into account can be found in the Eurocode (prEN 1990 2021).

For what concerns seismic assessment, EN 1998-3 adopts nonlinear analysis methods as the default choice. These methods employ nonlinear finite element models of the whole structural system (see 3.8). To avoid using different sets of material properties for respectively the model and the verifications (a choice that with the extensive use of the corresponding first generation Eurocode Part, has led to considerable complications in its practical application), the assessment values of material properties are prescribed to be the mean, i.e., the same as in the nonlinear model. Partial factors are then calibrated to consider this fact.

4.6 Deterioration

4.6.1 General

Deterioration due to environmental influences, repeated actions or use-induced wear, is typically a cumulative process that can adversely affect the reliability of existing structures. When designing new structures according to the Eurocodes, the possible combination of cumulative deterioration and extreme action effects is often neglected: durability design is usually considered separately from the design for ultimate and serviceability limit states (see chapter 3). In some material-oriented Eurocodes, the design to prevent deterioration is based on condition or proxy limit state verifications. Such simplifications are certainly not appropriate for the assessment of existing structures that are already affected by deterioration mechanisms:

reliability requirements should be verified for the combined effects of cumulative deterioration and the relevant actions and influences likely to occur during the remaining service life;

condition limit states intended to prevent deterioration from affecting the performance of a new structure may not apply to existing structures that are affected by ongoing deterioration;

indicators that are not based on measurable quantities cannot be used for inspection and maintenance planning.

In those cases adjusted project specific risk based criteria may be needed.

4.6.2 Remaining structural capacity and verification

Resistance models are usually based on a combination of mechanical principles and empirical relationships. Models that explicitly take into account the effects of deterioration on the resistance, adjusted or newly developed should preferably be based on physical principles. The scope of such models should include all relevant material-specific deterioration mechanisms and the associated uncertainties should be quantified.

Models should also be developed to describe the propagation of deterioration as a function of time, with the aim of predicting the condition of an existing structure over the remaining service life, going out from its actual condition at the time of assessment. Depending on the conditions to which the structure is exposed (e.g. environmental influences, repeated actions), these models should describe the onset and the rate of the cumulative processes that affect the parameters influencing the remaining structural resistance. The spatial distribution of the processes should be accounted for if relevant.

In deteriorating structures, the reliability requirements should be based on a reference period of one year (see 4.3) and verifications should be carried out for the final year of the intended remaining service life (see 2.4 for time effects and 2.5.3 for annual versus (remaining) lifetime target reliabilities). The uncertainties associated with the models that describe the propagation of deterioration as a function of time should be taken into account, along with those associated with the resistance models for deteriorated structures. However, the former may be reduced by implementing structural health monitoring techniques to provide information about environmental influences on the structure, degradation processes or structural performance and their variation over time

4.7 Closure

The staged assessment process of an existing structure can be completed if clear conclusions can be drawn from the findings regarding the assessment objectives, or if an additional assessment step is unlikely to provide relevant new knowledge. Depending on the assessment findings, the structure or a structural member may, within the scope of the assessment:

achieve the reliability required, assuming adequate inspection and maintenance during the remaining service life;

achieve the reliability required at the time of the assessment, but not for the complete period of time during which the existing structure is intended to remain operational, taking into account the anticipated development of its condition and the planned level of maintenance;

fail to achieve the reliability required;

need immediate correction of the existing condition by means of urgent risk mitigation measures.

If the required degree of reliability is not achieved with regard to structural safety, robustness, serviceability or durability, intervention is needed. Appropriate interventions should be defined case-specifically, taking account of the following:

the type and importance of the structure;

the type of basic requirement (prEN 1990 2021) that is not met;

possible cause and mode of attaining a limit state;

expected consequences of failure;

options of interventions that are available.

5 Guideline for probabilistic and risk-based decision making

5.1 Introduction

Note on terminology: In this document risk is understood as measure for the combination (product) of likelihood and consequences of an adverse event (see also chapter 2.3). Although this definition is widely accepted in the European construction sector, it is different to the one in other publications where a risk analysis is limited to the probabilistic aspects like in "seismic risk analysis. Sometimes even, the terms risk and probability are considered as synonyms.

The main application of risk assessment and reliability analysis is to serve as an indicative background for the calibration of partial factor design procedures, being the mainstream of the Eurocodes. This makes the information in this chapter meaningful for all NSB's that are responsible for formats and numerical values in the National Annexes. However, the NSB may also define conditions and procedures for direct application of risk and reliability-based design on a project level. The most common condition for such allowances is that the Eurocode gives no solution for certain types of structures or certain types of design situations. However, also in cases where Eurocodes are believed to be less adequate from a safety or economy point of view, the National Annex may allow, recommend or require a more advanced risk or reliability based approach. Examples are:

- as an alternative to the prescriptive robustness rules for structures with high consequences in case of failure;

- within procedures described in EN 1991-1-8 (for design purposes) and EN 1990-2 2021 (for existing structures);

- for unconventional structures (e.g. very large structures, new design concepts, new materials).

Note that risk and reliability-based approaches may be used for special aspects of the structure, while most of the design is using conventional partial factor design procedures based on standardised consequence classes and target reliability classes.

5.2 Reliability based design procedures

This approach is used if the standardised target reliability according to EN 1990:2023 Annex C is accepted as it is, but no satisfactory corresponding set of partial factors is available.

The goal might be to check for adequate reliability only, but one might also aim at minimization of costs and/or resources. It is strongly recommended to use similar probabilistic models as presented in Annex A of this document or (if not possible) to use the same level of schematisation.

In order to calculate the annual or lifetime failure probabilities one needs:

- Limit state functions that represent underlying scenarios;

- Probabilistic models for the random quantities;

- Reliability calculation procedures.

Limit state functions and the underlying failure scenarios should in principle not be different from those used in partial factor design and they correspond where possible to the mechanisms described in the material related Eurocodes. Of course, it is possible that refinements of those models are intended or that no limit state formulations are presented in the Eurocodes. Note that when deriving a mechanism formulation for use in reliability analysis from existing design rule (either in Eurocode or elsewhere) one should:

- take care of application limits;

- investigate the model uncertainties;

- avoid introduction of hidden safety elements or make them transparent and public available to practitioners and code-writers (see also chapter 2.2.3).

Probabilistic properties for reliability-based design should follow if available from Annex A of this background document. If not possible a combination of data and engineering judgement should be used. Engineering judgement should be treated with care and in particular the influence of persons having somehow an interest in the outcome of the calculations should be avoided. Sensitivities to arbitrary choices should be assessed and distributions with fixed upper or lower limits should not be used unless there are convincing reasons. Updating

of information on the basis of tests and observations shall be taken into account where appropriate (e.g. when assessing existing structures).

The calculation of the failure probability can be performed by (a combination of) standard calculation techniques such as FORM/SORM, mode combination rules, response surface techniques, the outcrossing approach and various Monte Carlo Simulation techniques. In principle, the outcome of a reliability analysis is the failure probability for the reference period under consideration and there is no need to check a set of specific design situations. However, for reasons of computational manageability it may be necessary to consider a set of individual design situations (transient, accidental, member, yielding, brittle fracture, etc) and combine them afterwards. For examples reference is made to Annex B.

In the seismic context, EN 1998-1-1 introduces with Annex E (Simplified reliability-based verification format) the possibility of checking reliability in the seismic design/assessment situation with a simplified formulation based on nonlinear analysis, using the total probability theorem to evaluate the annual probability of exceedance of the (global, structural system level) limit state according to:

$$P_F = \int_0^{\infty} P(LS|S_e) \left| \frac{dH(S_e)}{dS_e} \right| dS_e \quad (5.1)$$

where $P(LS|S_e)$ is the fragility function and $H(S_e)$ the hazard function (mean annual probability of exceedance of a given value of the local intensity S_e , see 3.5.7. The method is simplified because under the assumption that the fragility is lognormal and the hazard is linear in the log-log space P_F can be obtained in closed-form, using a nonlinear static analysis and tabulated values of the uncertainty in S_e (Cornell et al, 2003; Dolšek and Fajfar, 2007).

The choice of the computational tool including the setting of possible steering parameters is of course essential for the accuracy of the result. The suitability of a method depends on the shape of the failure domain (defined by the set of limit state conditions) and the number and type of the (independent) random variables. Problems resulting in linear or almost linear limit state formulations are usually easy to solve (e.g. using FORM) while difficulties may arise in highly nonlinear and time dependent structural problems, in particular if the decisive limit state is reached after a sequence of plastic and brittle local failures. Also, the presence of spatial variability may be a complicating and time consuming factor.

In order to find out whether a method will lead to sufficiently accurate results or not, the characteristics of both the computational method and the reliability problem at hand should carefully be investigated. In this respect it is difficult to formulate generic rules. It may be helpful to apply several methods and compare the results and convergence behavior. The depth of such endeavours may depend on the decision making context..

Crude Monte Carlo is generally speaking always reliable and will converge to the exact result, but is of course time consuming. An interesting feature however is that the (statistical) uncertainty can be quantified explicitly on the basis of probability theory. This uncertainty may the also be incorporated in the final result in a similar way as described in EN1990:2023 Annex D for physical experiments.

5.3 Risk based decision procedures

In risk management usually the following steps are distinguished:

- Step 1 Qualitative Risk analysis
 - Description of the system and processes
 - Identification of hazard scenarios, failure modes and consequences
 - Selection of decisions to be taken
- Step 2 Quantitative Risk Analysis (QRA)
 - Definition of models, quantification of probabilities
 - Quantitative definition of consequences

-Effects of mitigation measures

Step 3 Risk evaluation, optimization, acceptance and communication

-Human safety, cost efficiency, environmental and sustainability issues

-Presentation of results and communication with stakeholders

The essence of a risk based or risk informed design (assessment) is that the decisions are taken with respect to a set of possible alternatives in such a way that the benefits minus the use of the resources needed is optimal. All models used should not be looked upon as absolute, but depend on the context of this decision making space. The steps will be discussed in more detail in the next section. For more information see also (ISO 2394, ISO31000, JCSS Risk analysis).

5.3.1 Step 1 Qualitative Risk analysis

As a first step in a risk informed decision making the system under consideration shall be identified, including the environment, all its components and processes, the limitations in space and time, the relevant hazard scenarios and the possible direct and indirect consequences. Special considerations shall be given to the interaction between the structure and its environment which may include natural constituents, possible mechanical systems as well as human and organizational procedures. Examples are operational and environmental loads, aggressive chemicals, human errors, poor quality of design, materials and construction etc., individually as well as in combination.

The inventory of hazard scenarios is in principle a creative process. In order to stimulate these processes systematic approaches like PHA, HAZOP, FMEA, fault and event trees etc. have been invented. A typical approach is to consider all relevant variables in the system under more or less normal conditions and answer the question what could be the origin of deviations of those values (or range) as well as the direct and indirect consequences. Very helpful may also be to use check lists like the examples in Table 5.1. One should however keep in mind that these lists always can be incomplete.

Table 5.1. Overview of hazards.

Normal loads	Accidental /natural	Accidental /manmade	Human influences	Human Errors
Self-weight	Earthquake ^{a)}	Internal explosion	Mining subsidence	Design error
Imposed loads	Landslide	External explosion	Demonstrations	Material error
Car park loads	Hurricane	Internal fire	Public disorder	Construction error
Traffic	Tornado	External fire	Vandalism	Misuse
Snow	Avalanche	Impact by vehicle etc	Terrorist attack	Lack of maintenance
Wind	Rock fall	Mining subsidence		Miscommunication.
Hydraulic	High groundwater			
Temperature	Flood/storm surge			"Unforeseen"
Physical attack	Volcanic eruption			
Chemical attack	Tsunami			
Earthquake ^{a)}				

a) depending on the region it is normal or accidental

Some comments with respect to this table might be given:

Generally, the **human error** is considered as the main cause of accidents. Most researchers give values in the order of 60-90 %. Errors may be made during the design (conceptual errors, misinterpretations of rules, calculating errors, software errors, drawing errors), during execution (misreading of specifications, bad workmanship, inferior materials) and use (operation, inspection, maintenance, refurbishment). The elaboration is closely related to EN 1990:2023 Annex B

The category of **vandalism and terrorist attack** is a type of action where there is typically a deliberate aspect involved. This complicates the modelling task. The intention of the action is destruction and for the destroyer the strength of a structure is the starting point. To some extent it does not help to make the structure stronger, as it can provoke more action on the loading side. Of course, by proper design and robustness, one can make it more difficult for persons who want to destroy the structure as a whole.

A very special category is the so-called **unforeseen** hazards. Even in the most careful QRA some hazards may be underestimated, unrecognized or objectively unforeseeable (sometimes referred to as black swans). By definition no specific information is available. The category "unrecognised or ignored" is a kind of human error, the "black swan category" is a shortcoming of the whole profession. Due to the fact that unforeseen hazards cannot be modelled directly another approach has to be followed. The usual way is to model the possible effect of the hazard in the form of a reduction in strength or complete (notional) removal of one or more structural members. Some of the approaches in EN 1990:2023 and EN 1991-1-7 and corresponding robustness requirements are related to this category.

Elaborating all hazards in combination with possible mechanisms in the structure (yielding, rupture, fatigue, chemical/physical mechanisms, etc) may lead to undesired behaviour and in the end to failures with serious consequences. The local structural damage may be referred to as the direct consequences, the further global effects on the structure, the environment, the users as well as the indirect consequences. See Figure 5.1 for illustration. Table 5.2 gives an overview of possible final consequences to be considered.

Table 5.2. Types of the possible consequences to be considered.

Health	Cost of damage	Business Continuity	Environmental	Social and Political
Fatalities	structures	Loss of income	Reversible damage to nature	Loss of reputation
Injuries	surrounding properties	Loss of customers	Irreversible damage to nature	Increase of public fears
Damaging vital facilities (e.g. hospitals),	contents	Inability to provide vital services		Loss of political support/.
Spread of diseases		detours and delays		stringent new measures
Delayed and long-term health effects		economy of a region		"Blight"/long-term evacuation
Psychological				

5.3.2 Step 2 Quantitative Risk Analysis (QRA)

In this step the hazards, the behaviour of the structural system and corresponding consequences are modelled and all uncertainties are quantified. For the modelling of hazards reference is made to Annex A of this report,

although for a number of models a direct further reference to Eurocodes EN 1991, EN 1997 and EN 1998 is made. Structural behaviour should be modelled as according to the material related model codes, although in some cases more advanced models like present in EN 1998 would be required. For models related to consequences (in terms of the expected number of casualties) reference is made to (Tanner et al. 2015, Hingorani et al. 2019).

Given all information, the risk R for a certain time reference interval can be assessed as the expected value of the sum of the consequences, where the expectation operation is taken over all uncertainties affecting the performance of the system:

$$R = \sum_{i=1}^{n_E} P_i C_i \quad (5.2)$$

where P_i and C_i are the probability and the consequence associated with event i respectively, all events being mutually exclusive. Both the number of possible events, their probabilities as well as the associated consequence generally depend on the decision (design) alternative chosen.

According to EN 1991-1-7, Annex B, the equation can be developed further to (see also Figure 5.1):

$$R = \sum_{i=1}^{N_H} p(H_i) \sum_j^{N_D} \sum_{k=1}^{N_S} p(D_j|H_i) p(S_k|D_i) C(S_k) \quad (5.3)$$

where:

H_i hazard scenario (or hazard combination i);

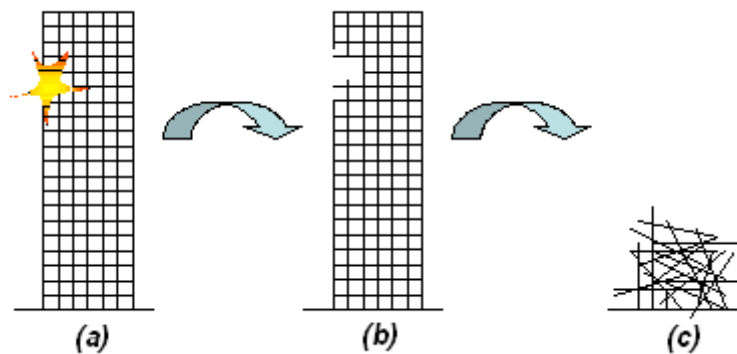
D_j structural damage state j ;

S_k the exceeding of adverse (limit) state k ;

$C(S_k)$ expected consequences related to S_k .

Additionally, N_H is the number of different hazards considered, N_D the number of selected damage states and N_S the number of adverse states. In principle all hazards H_i according to Table 5.1 and all consequences according to Table 5.2 should be considered, although in most cases a risk screening process will lead to a focus on the most relevant scenarios and consequences.

Figure 5.1. Illustration of factors risk analysis of structures subject to accidental actions. (a) Identification and modelling of relevant accidental hazards. (b) Direct structural damage (often damaged column or floor). (c) Further performance of the damaged structure and final indirect consequences. (source Eurocode 1991-1-7)



In case of human safety, the consequences 5.3 should be read as the expected number of casualties conditional upon the exceedance of the limit state S_k . The total risk R corresponds in that case to expected number of people being killed in the reference period (usually one year). In some cases, there may be a consideration of corresponding economic aspects (see Step 3).

For indicative statistical data, depending for instance on the role and characteristics of the structure, reference is made to annex A; some examples may be found in Annex B.

The goal in the end is to find an optimal solution, at least one that is feasible. For that reason, one has to evaluate the risk for a number of design alternatives. In particular one has to consider several levels and methods of risk mitigating measures (see Table 5.3).

Table 5.3. Examples of risk mitigating measures.

Elimination or reduction of the hazard
Protection of the structure, provision of sprinkler system, etc.
Controlling the hazard, for example, warning systems or monitoring.
Increased reserves of strength
Alternative load paths through structural redundancy
Controlled collapse of where the probability of injury or fatality may be reduced

5.3.3 Step 3 Decision making, optimization, risk acceptance and communication

In risk-based decision making one seeks the decision alternative that maximizes the utility of the decision maker AND is acceptable in regard to human safety from a societal perspective. In order to assess both criteria one has to (see ISO 2394, Annex G):

- Identify **an optimal decision** alternative. The optimisation considers the utility of the decision maker. The utility might be expressed in monetary terms and intangible assets might be translated to monetary terms reflecting their economic implication for the decision maker. Interest rates for discounting future costs and benefits should be chosen as representative for the type of economic endeavour.
- Check the optimal decision alternative for **societal acceptance**. The marginal lifesaving cost principle is employed and the cost surrogate for the representation of the societal preference for the prevention of the loss of human lives is obtained by the LQI (JCSS, 2008). Interest rates for discounting future costs should be chosen as representative for societal economic growth.
- Further criteria reflecting codes, standards and legislation might be added.

As already discussed in section 2.3.2 the utility of the decision maker should include:

all the relevant benefits;

all expected resources related to building activities, future maintenance (inspection/ monitoring/repair) and dismantling;

the cost (utility) expectation over all other possible random consequences of structural behaviour.

As far as human safety is concerned (and maybe also other intangible assets) there is the choice to include nominal compensation costs or costs based on a marginal lifesaving cost principle. The first option seems to be appropriate for the private decision maker, the second for the public decision maker. In the end of course the private decision maker is also bound to public constraints.

It can be concluded that the marginal lifesaving cost principle is applied in the optimization (by the public decision maker) and in the societal acceptance criterion (usually proposed by NSB's and endorsed by the relevant authorities). Typical notions related to the LQI (Life Quality Index) are:

The Societal Willingness To Pay (SWTP) and

The Societal Value of a Statistical Life (SVSL).

The results of the societal acceptance criteria are often presented in the form of minimum reliability levels (e.g. ISO2394, Table G3). Criteria concerning life safety risk acceptance are often specified in terms of the As Low As Reasonably Possible/Practicable (**ALARP**) format. The idea is that risks depending on their magnitude can be evaluated to be negligible, tolerable or non-acceptable, as shown in Figure 5.2 where the ALARP format is illustrated.

Figure 5.2. Illustration of the ALARP principle in life safety risk regulation.

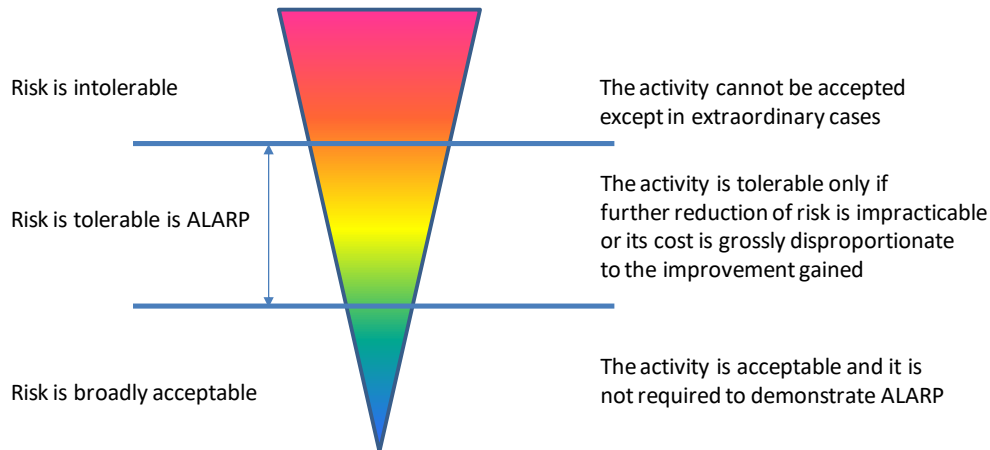


Table 5.4 shows an example of corresponding numbers for individual human life safety. In between the upper and lower limits, the cost optimization method is acceptable.

Table 5.4. Overview of risk criteria in terms of annual individual fatality rates.

Application	Fatality rate [a ⁻¹]
Upper limit extra ordinary cases	10 ⁻⁴
Upper ALARP limit for existing structures and installations	10 ⁻⁵
Upper ALARP limit for new structures and installations	3 x 10 ⁻⁶
Lower ALARP value	10 ⁻⁶

It should be kept in mind that criteria following from the above notions refer to averages. This means that within a group under consideration there may be persons with a less and a higher fatality rate than the average. From an ethical point of view it makes sense to have an additional check for the highest risk levels on an individual basis. An **individual** may be defined either as a real person (going all the time from one location to a next one) or as a hypothetical person spending all the time on one place. The second option may be considered as the most operational in regulatory concepts (Faber et al 2015).

In addition to the ALARP the so-called **precautionary principle** deserves mentioning. This principle has been formulated in the context of European risk regulation. The principle can be interpreted as a measure to safeguard society against (technical) developments having disproportionately large consequences for society. The precautionary principle finds application in relation to the assessment of activities for which there is no experience and for which there is suspicion that they might have unacceptable adverse consequences. For such situations it is required that more knowledge must be collected to ensure that such activities can be sufficiently controlled and managed using known best practice technologies.

An important issue is the **scale of an accident**. In general, one large accident attracts more attention in the news media and the public than a large number of small accidents. These perceptions may be called irrational but the effects on the general acceptance cannot be ignored and some extra weight to the consequence of large accidents may be needed to reflect the public risk aversion. The most common approach is to take the damage not proportional to N but to N^k , where k is a number between 1 and 2, reflecting increasing risk aversion.

Usually and for several reasons a structure is planned for some specified **period of time**. The principles in this chapter may hold for every possible period of anticipated design (working) life. One should however keep in mind:

The actual lifetime is a random variable, also depending on future events and decisions.

There may be a need for the structure after that period and that makes the design service life as such a design parameter in itself.

In the case of temporary structures, parts of it may be re-used for a next period of time; the continuous processes of mounting and demounting may involve special risks.

There is a limit to the time a structure can serve society in a certain set up; this may lead to demolishing (parts of) the structure after a certain period. When designing a structure, options for reuse should be kept in mind.

An essential aspect of risk management is **risk communication** before, during and after the undesired events. One should have in mind the various parties involved who may all have different interests and different perceptions. The degree of acceptance normally depends on the degree of exposure, influence and involuntariness. A particular aspect of risk communication is responsibility. Risk acceptance and correspondence is always the responsibility of the authority. The QRA-specialists are responsible only for presenting information in a transparent way on the basis of traceable up to date knowledge.

Finally, it should always be realized and also clearly mentioned that calculated risks are estimates of reality based on a given **state of information**. So, later, it might prove to be incorrect to some extent, but nevertheless it should now be accepted as the best basis for making decisions.

6 Concluding remarks

Achieving an adequate level of reliability is an essential part of design and assessment of structural systems. It requires a thorough knowledge of structural behaviour under the various possible loading conditions. In addition, all disciplines in the field of structural engineering have to deal with uncertainties. It is of great importance that in all action and material related Eurocodes the treatment of uncertainty is done in a transparent and standardized way. Otherwise, misunderstandings between experts working in different fields could occur and lead to dangerous or uneconomic situations.

The basis for the way to deal with uncertainties in structural engineering is formulated in EN 1990:2023, Basis of Structural and Geotechnical Design. The present report has been written to give background information to the various clauses and annexes of EN 1990:2023 as well as to present other state of the art material that might inform future developments. The aim has been to show how the contents of EN 1990:2023 are compatible with the relevant parts of other Eurocodes and to contribute to a common understanding of basic notions like uncertainty, reliability, safety, risk, code calibration and so on.

It turned out not to be an easy job and in some cases the report can only mention that discrepancies do exist between present practice and present knowledge or that present knowledge simply has no adequate answers with reliance therefore often placed on continuity of past practices. These of course are the topics for future research which hopefully will continue now and in due course support further future evolution of the Eurocode. Subject areas where further research could be helpful include:

- Annual versus lifetime reliability requirements;
- Target reliability: required resources versus risk reductions for all types of limit states;
- Use of nonlinear FEM methods;
- Partial Factors for prestressing;
- Background of load combinations;
- Risk based inspection and monitoring;
- More guidance on risk analysis and robustness;
- More background for requirements for existing structures;

An important (but not new) observation concerns the discrepancies between target and calculated reliabilities and real failure frequencies. To some extent these discrepancies are related to so-called “hidden safeties” in the design formula and the design process (see chapter 2). It is important to realise these hidden safeties, in particular when accepting design methods which may decrease the degree of conservatism like for instance certain wind tunnel tests. In those cases one might easily throw away a part of the hidden safety margin, and without realising, making the structure less safe than desired.

The calculations carried out in the process of writing this document gave indications that the recommended partial factors indeed on average lead to a reliability index of 3,8 for a 50 year reference period, as was taken as a standard. In Annex B example calculations are presented for the derivation of partial factors using the methods mentioned in Annex C.4.4 of EN1990:2023. The main conclusion is that NBS's should only aim for this approach if a more extensive analysis is performed. However, also on the level of CEN TC250/SC10 a continuation of the work started in this document is to be recommended. Possible issues for research could include reduction of uncertainties in climatic actions, performing combined optimization on loading and resistance side, setting new targets related to use of materials and modifying the system of reliability formats.

An important remark to be made is that the calculations in this report have been carried out using the exact definition of the characteristic values according to EN1990:2023. In some member states, however, values specified in National Annexes, for example environmental actions, appear to have been established on a different basis. This should be taken into account carefully when calibrating corresponding partial factors for inclusion in National Annexes. An example is given in Annex B.2.5.

In order to give guidance to reliability and risk based calculations, whether code oriented (as in Annex B) or for specific applications (as in chapter 5), a set of probabilistic models is presented in Annex A. The models presented reflect the state of the art in 2021. It is recommended that research in this field is continued and new data be gathered to update Annex A say every 5 years in collaboration with the JRC. Cooperation with organisations like ISO and JCSS could be helpful.

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List of abbreviations and definitions

AHG	Ad Hoc Group
ALARP	As low as reasonable possible/practicable
CC	Consequence class
CEN	Comité Européen de Normalisation (European Committee for Standardization)
EN	European Standard
ENV	European Preliminary Standard
FORM	First Order Reliability Method
ISO	International Standard Organisation
MC	Monte Carlo
NBS	National Bureau of Standards
QA	Quality Assurance
SC	Subcommittee
SLS	Serviceability Limit State
TC	Technical Committee
UC	Unity Check
ULS	Ultimate Limit State
WG	Working Group

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Annexes

Annex A. Probabilistic Models

A.1 Introduction

This annex contains recommended statistical (probabilistic) models to be used in structural reliability and risk calculations. The models may also be used as basis for the calibration of partial factors. The models are the basis for most of the reliability calculations in chapter 4 and Annex B of this document.

For all models the background to the information is presented. Where possible the models given in this annex correspond to the standardized models given in Eurocodes EN 1991 to EN 1999 or in the corresponding background documents. If no such information is available other sources like the JCSS Probabilistic Model code have been used. Some data models are partly based on engineering judgement. See also chapter 3.2.1.

The models represent the state of knowledge of 2021 and refer to the situation that no project-specific information is available. The model uncertainties are intended to reflect the uncertainty in normal engineering, numerical or experimental models. If in special cases more information is available the models in this Annex can be used as priors in a Bayesian updating procedure (see text below for detailed information).

Note further that this annex does not address the issue of climate change and the possible consequences for design and assessment. Some information can be found in chapter 4 of CEN-TC250-SC1 N1735 Interdependence of climatic actions (2021). However, at the moment there is no translation to the corresponding Eurocode sections, but this of course may be different in the future.

Further general technical introductory remarks:

Data for geometrical properties (thickness etc.) are given in the chapter on self-weight but may also be important for the resistance part.

The mean or bias in the tables is defined as the ratio of the actual mean value and the mean that follows theoretically from the characteristic value and the coefficient of variation.

Range in Tables may refer to different references, locations or circumstances; without further information medium values are recommended.

In some cases the user may have some local or material specific data, In that case the models described in this annex may be used as kind of prior. Typical values for the equivalent sample sizes of the priors are:

$n' = 1$ to 5 for the mean value

$v' = 3$ to 10 for the scatter

In most tables also values for the weight factors w_k are given; they should be considered as based on engineering judgement only. They reflect the relative importance of the load or material, both with respect to the presence in design as with respect to the possible consequences in case of failure.

A.2 Permanent actions

A.2.1 Self-weight

The self-weight of structural elements or structures of material i may be modelled as (JCSS PMC 2001):

$$G_{S,i} = \int_{Vol} \rho_i dV \quad (A.4)$$

where:

ρ_i is the weight density which is a random field;

Vol is the volume of the structure which is affected by deviations from the nominal values which are material-dependent.

The geometrical dimensions (thickness, cross-sectional area etc.) are modelled as $d=d_{nom}+\delta$ where d_{nom} is the nominal dimension (deterministic) and δ is the deviation which is represented by a normal distribution with parameters $\mu_\delta, \sigma_\delta$. It follows that d is normally distributed with parameters:

$$\mu_d = d_{nom} + \mu_\delta \text{ and } \sigma_d = \sigma_\delta. \quad (A.5)$$

Notes:

To cover (unsafe) conservative estimations of self-weight a bias of 1,05 is sometimes proposed.

Spatial correlation maybe important when self-weight has both large positive and negative contributions to a load effect. The JCSS PMC recommends $\rho=0,7$ for two different members.

Values for distribution parameters depend on the material and type of structural member.

A.2.1.1 Self-weight concrete structures

The self-weight of a box shaped concrete element with dimensions d_1, d_2 and l may be modelled as

$$G_s = d_1 d_2 l \rho \quad (A.6)$$

Reinforcement is not considered, usually around 1-2% of cross section and accounted for by increasing density of concrete.

The dimensions usually are considered as uncorrelated. The properties for the structural dimensions are presented in Table A.1 (JCSS- PMC, 2001).

Table A.1. Concrete elements self-weight models

	w_k	x	μ	v
Weight density		ρ	nominal	0,04
Large elements casted in situ $1000 \text{ mm} < d_{nom} < 2000 \text{ mm}$	30 %	d	$d_{nom} + 3\text{mm}$	$\frac{10\text{mm}}{d_{nom} + 3\text{mm}}$
Small elements casted in situ $100 \text{ mm} < d_{nom} < 1000 \text{ mm}$	70 %	d	$d_{nom}(1+0.003)$	$\frac{4+0.006d_{nom}}{d_{nom}(1+0.003)}$

The resulting $G_{s,i}$ for concrete elements may be considered to have a (on average) coefficient of variation $V = 0,046$.

References

JCSS (2001), *JCSS Probabilistic Model Code*, Joint Committee on Structural Safety, <https://www.jcss-lc.org/jcss-probabilistic-model-code/> (periodically updated, online publication)

A.2.1.2 Self-weight steel structures

The self-weight for steel members may be modelled as

$$G_s = A l p \quad \text{for profiles,}$$

$$G_s = l_1 l_2 t p \quad \text{for plates.}$$

If items like welds, bolts and stiffeners are not dealt with explicitly, one may on average take them into account by an additional 2 %.

The structural steel weight density may be taken as normal with mean equal to the nominal value and $V=0,01$.

Parameters for geometrical properties A and t may be taken as given in Table A2 (JCSS PMC, 2001).

Table A.2 Steel elements self-weight models

	X	μ	CoV
gross section area	A	$0,995 A_{nom}$	2,0%
net section area (bolt holes etc.)	A_{net}	$0,99 A_{net,nom}$	2,5%
section modulus (for bending capacity)	W	$0,995 W_{nom}$	2,0%

The resulting $G_{S,i}$ steel elements may be considered to have a coefficient of variation $V=0,025$ to $0,033$.

References

JCSS (2001), *JCSS Probabilistic Model Code*, Joint Committee on Structural Safety, <https://www.jcss-lc.org/jcss-probabilistic-model-code/> (periodically updated, online publication)

A.2.1.3 Self-weight timber structures

Timber element self-weight may be modelled as (steel joints and plates not accounted for);

$$G_s = d_1 d_2 l p \quad (A.7)$$

with (nominally):

$0,05m < (d_1, d_2) < 0,45m$ for sawn elements, and

$0,05m < d_1 < 1,55m$ and $0,05m < d_2 < 0,45m$ for glulam elements.

The timber weight density may be taken as normal with mean equal to the nominal value and $V=0,10$; where relevant a more detailed model taking into account means and fluctuations of moisture content may be appropriate.

Geometrical parameters may be taken from Table A.3.

Table A.3 Timber elements self-weight models

	w_k	Geometrical dimension	μ	V
Sawn	10 %	d_i	$d_{nom}(1+0,05)$	$\frac{2mm}{d_{nom}(1+0,05)}$
Glulam	90 %	d_i	d_{nom}	$\frac{1mm}{d_{nom}}$

The resulting $G_{S,i}$ timber elements may be considered to have on average a coefficient of variation $0,105$.

In some cases it may be necessary to add a certain weight (say 2 or 5 %) to take into account the weight of steel elements used in connections.

References

CIB-W81 (1989), *Actions on structures Self-Weight Loads*, Publication no. 115, CIB, Rotterdam

A.2.1.4 Self-weight aluminium structures

It may be assumed that the structural aluminium alloys weight density is normal with mean equal to the nominal value and $V=0,01$. Because deviation in density is non-existent for practical purposes, the self-weight of aluminium structural members is mainly influenced by their geometrical properties.

The member self-weight may be modelled as:

$G_s = A l p$ for profiles,

$G_s = l_1 l_2 t p$ for plates.

Statistical parameters, mean value (μ), coefficient of variation (V) and distribution types for available dimensional parameters (geometrical properties) may be taken from Table A.4. For dimensional parameters not specifically mentioned in Table A.4, it may be assumed that the mean values are equal to the nominal values and the standard deviations are equal to half of the interval between the nominal value and the lower bound of the applicable tolerance interval in EN 1090-3 or other relevant product standards.

Table A.4 Variability of available aluminium members' dimensional properties

Dimension type	Parameter / weight factor w_k	Mean value x_m	Coefficient of variation	Distribution type
Thickness	Extruded profiles / 75%	$0,95 t_{nom}$	5,0%	Lognormal
	Welded profiles / 5%	$1,00 t_{nom}$	4,0%	Normal
	Plates / 20%	$1,00 t_{nom}$	2,0%	Normal
Outer dimensions of cross-section	Depth h	$1,00 h_{nom}$	0,3 %	Normal
	Width b	$1,00 b_{nom}$	0,3 %	Normal
	Outer diameter d of circular hollow section	$1,00 d_{nom}$	0,5 %	Normal
Nominal dimensions according to the applicable product standard or specification.				

References

Dokšanović, T. (2017), *Lateral-torsional buckling of aluminium elements with open unsymmetrical cross section* PhD thesis, University of Zagreb (in Croatian)

A.2.1.5 Self-weight masonry

In general the mean value is taken as the nominal value and a coefficient of variation of 5% is assumed.

References

Rackwitz, R. (1996), *Einwirkungen auf Bauwerke*. In: Mehlhorn, G. (Ed), *Der Ingenieurbau: Tragwerkszuverlässigkeit, Einwirkungen*. Ernst & Sohn Verlag, Berlin, Germany (in German)

A.2.1.6 Self-weight ground

The weight of ground is usually not relevant for the self-weight of a structural element (G_s) but for the stress conditions in the ground surrounding foundations elements. It is also important for determining the load effects and resistances in other geotechnical structures such as embankments or retaining walls.

The coefficient of variation for the weight density of soil may be considered in the range of 2 to 5 % for both, clay and sand, considering the typical amount of site investigation and spatial averaging within the zone of influence. (5 to 10% CoV total variability according to prEN1997, i.e. without spatial averaging).

A.2.2 Non-structural permanent loads

Next to the permanent load caused by the weight of structural elements there may be other loads of (almost) permanent character, originating for instance from:

manufactured elements such as flooring systems, surfacing, facades and ceilings;

finishing elements connected with the structure, including hand rails, safety barriers, parapets, weight of cables, pipes and service ducts;

additional provisions like lifts;

machinery fixed permanently to, or within, the structure;

ventilation and colling equipment;

loads due to ballast systems;

earth loads on roofs and terraces.

Some of these loads may have a quite large uncertainty, this however may also depend on the type of analysis (generic or specific, long or short period). Table A.5 gives some guidance for numerical values to be taken into account.

Table A.5 Numerical values for non-structural permanent loads

Random variable	X	Distr. type	Mean(μ)	V	Char value (Fractile)
Permanent load	G_P	Normal	1,00	0,10	0,50
Permanent load (large V)	G_P^*	Normal	1,00	0,20	0,95

Some of the non-structural elements may be exchanged or removed during the design lifetime, e.g. ventilation and cooling equipment and could therefore be considered as a free permanent load not present when it acts favourable for the load effect considered.

A.3 Loads varying in space and time

A.3.1 General

The uncertainties on the action side in structural analysis are associated with:

The intrinsic variations in time and space;

The possible deviation of the adopted and simplified action model from the real action (load model uncertainties).

The time variability is accounted by considering maxima over a reference period T which in general is considered to be one year. If one has to change from one year to a period $T = n$ years, it is necessary to make the distinction between the varying in time and constant in time random variables:

$$F_{QT}(q) = \int F_{Q_1}(q;c)^n f_C(c) dc \quad (A.8)$$

where:

Q_T is the maximum load in period T;

Q_1 the annual maximum for the time varying variable and

C represents the time invariant variable(s).

The time variant part of most loads is dependent on the geographical location and the use of the building. Even if wind- or snow zones are distinguished there still may be an important systematic (but often randomized) variation within the zone.

Note finally that according to EN 1990:2023 the characteristics values for variable loads are defined as values corresponding to an annual exceedance probability of 0,02, including both time variant and invariant uncertainties. One should be aware of the fact that code values in EN 1991 and/or National Annexes may deviate from that definition.

A.3.2 Imposed load

In calculations the space variability of the loads is accounted by using an equivalent uniformly distributed load (EUDL) as described in the JCSS PMC. Live load is modelled as the sum of equivalent uniformly distributed sustained q and intermittent loads p where (see Figure A.1).

The sustained load q may be modelled as an FBC process with random time intervals; the mean occurrence rate of a load change is given by λ_q (that is, a mean interval duration equal to $1/\lambda_q$). The distribution of the arbitrary point in time load may be given by:

$$q \sim \text{Gamma} \left(\mu_q = m_q; \sigma_q = \sqrt{\sigma_V^2 + \sigma_U^2 \frac{A_0}{A} \kappa} \right) \quad (\text{A.9})$$

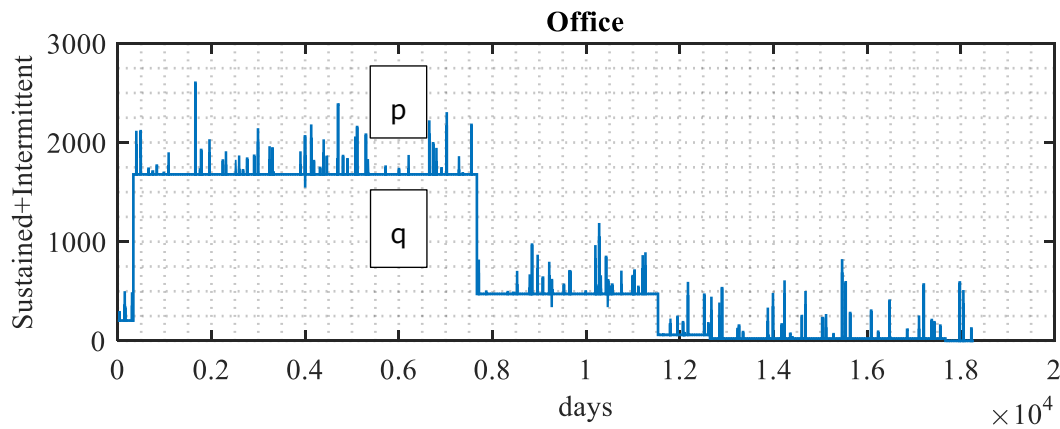
A is the tributary area and A_0 is the reference area usually taken as 10 m^2 (although depending on the type of use), keeping A_0/A at a maximum of 1.0; the value of κ follows from the shape of the influence factor. As an alternative one may choose the Gumbel instead of the Gamma distribution having a less “fat” tail.

The intermittent load intensity may be modelled as a Poisson pulse process having a mean occurrence rate λ_p (so the mean time between the occurrence of pulses is $1/\lambda_p$); the non-zero load realization may be modelled as:

$$p \sim \text{exponential}(\mu_p = \sigma_p = m_p) \quad (\text{A.10})$$

The duration of the non-zero loading period is d_p

Figure A.1. Arbitrary realization of the sustained intermittent load for an office space [N/m^2]



An approximation of the cumulative distribution function representing the maxima of the sum process $q+p$ over a period T is given by:

$$F_{(q+p)\max T}(r) \cong \exp \left\{ -\lambda_q T \left(1 - F_q(r) \right) - \lambda_p T \left(1 - F_{q+p}(r) \right) \right\} \quad (\text{A.11})$$

$$F_{q+p}(r) = \int_0^\infty f_q(x) F_p(r-x) dx \quad (\text{A.12})$$

The functions F and f represent, respectively, the cumulative distribution functions and the probability density functions for the various variables.

The upper tail of the cumulative density in equation A.11 may, for convenience, be approximated by a Gumbel and/or a Lognormal distribution with the parameters presented in Table A.6. This simplifies the code calibration work. Although the Gumbel distribution is best for representing extremes in general, the lognormal has the advantage of being defined on the positive real space only.

The model uncertainty for the live load model $\theta_{0,mod}$ is not known and probably dependent on the type of application; the present recommendation is a lognormal distribution with unit mean and with 0,10 coefficient of variation.

Table A.6 Parameters of the sustained and intermittent loads

	q-sustained load				p-intermittent load		
	Mean value			Mean duration of non-zero part	Mean value	Mean duration of non-zero part	Mean time between non zero part
	m_q	σ_v	σ_u	$\mu_{d,q}=1/\lambda_q$	m_p	$\mu_{d,p}$	$1/\lambda_p$
	N/m ²	N/m ²	N/m ²	years	N/m ²	days	Yr
Office	500	300	600	5	200	2	0,3
Lobby	200	150	300	10	400	2	1
Residence	300	150	300	7	300	2	1
Hotel room	300	50	100	10	200	2	0,1
Patient room	400	300	600	8	200	2	1
School	600	150	400	10	500	3	0,3
Merchant	900	600	1600	5	400	7	1

For office loading the maximum load in 50 years has been calculated which resulted in a Gumbel distribution with mean 2,22 kN/m² and V=0,26.

References

- JCSS (2001), JCSS Probabilistic Model Code, Joint Committee on Structural Safety, <https://www.jcss-lc.org/jcss-probabilistic-model-code/> (periodically updated, online publication)
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European Commission: Joint Research Centre, Markova, J., Pinto, A., Sousa, M., Dimova, S. et al. (2018), *Reliability of structural members designed with the Eurocodes NDPs selected by EU and EFTA Member States*, Pinto, A.(editor) and Dimova, S.(editor), Publications Office of the European Union, 2018, <https://data.europa.eu/doi/10.2760/426349>

A.3.3 Wind load

Eurocode 1 Part 1-4 (CEN, 2005) model for wind load is derived from the A. G. Davenport wind load chain. The characteristic peak velocity pressure is calculated in equation A.13 where, c_g , c_o , c_{dir} , c_{season} , c_r represent the gust, orography, direction, season, altitude and roughness coefficients, respectively; k stands for characteristic value and ρ for the air density. Here $c_o = c_{dir} = c_{season} = 1$ are considered deterministic.

$$q_{p,k} = \frac{1}{2} \rho \cdot c_{g,k} \cdot (c_{o,k} c_{dir,k} c_{season,k} c_{alt,k} c_{r,k} \bar{u}_{max, 1yr,k})^2 \quad (A.13)$$

Two methods are proposed in the Eurocode for calculating the characteristic wind action on a structure. The first, named *surface pressure method*, calculates the total force acting on a structure as the vectoral summation of the external, internal and frictional forces (included in order in equation A.14):

$$F_{w,k} = F_{w,e,k} + F_{w,i,k} + F_{fr,k} = c_{sd,k} \sum_i c_{pe,k}^i q_{p,k} A_{ref}^i + \sum_i c_{pi,k}^i q_{p,k} A_{ref}^i + c_{fr} q_{p,k} A_{fr} \quad (A.14)$$

where:

c_{pe} , c_{pi} , c_{fr} are the external pressure, internal pressure and friction coefficients, respectively;

c_{sd} is the structural factor which is equal to unity for structures which are not sensitive to dynamic effects and have small dimensions.

Friction forces are not considered further since c_{fr} in most cases is one or two orders of magnitude smaller than the other pressure (force) coefficients.

The second method named *force coefficient method* evaluates the characteristic force acting on a structure by means of an overall force coefficient c_f , see equation A.15.

$$F_{w,k} = c_{sd,k} \sum_j c_{f,k}^j q_{p,k} A_{ref}^j \quad (A.15)$$

The summations in equation A.14 are performed over different areas which the structure surface is divided in; the one in equation A.15 is performed over different elements composing the structure. The structural factor may be written as $c_{sd} = C_s C_d$ and takes into account the non-simultaneous occurrence of peak wind pressures on the surface (C_s) and the dynamic effects (C_d).

According to equation (5.3.1) in Davenport, 1987, the annual maximum wind pressure is obtained as

$$q = c_p (c_g c_r c_o)^2 q_{b1} \quad (A.16)$$

in which:

c_p is the external or internal coefficient of pressure;

c_g is the gust factor equal to $1 + k_u I_u$ with k_u expected peak factor and I the turbulence intensity;

c_r is the roughness factor accounting for the variability of the mean wind velocity at the site of the structure due to the height above ground level and the ground roughness of the terrain upwind of the structure for the wind direction considered;

c_o is the orography factor which takes into account the orography effect (e.g. hills, cliffs, etc.);

q_{b1} is the basic velocity pressure associated with the basic wind velocity $\bar{u}_{max, 1yr,k}$, defined as a function of wind direction and time of the year at 10 m above ground of terrain category II.

The air density is modelled as a deterministic variable since its scatter at large wind speeds is small, see e.g. Kasperski, 2009 for an example of real measurements. The roughness factor C_r describes the variation of the pressure with height. The gust factor C_g is the ratio between the peak velocity pressure to the mean velocity pressure. By approximating the longitudinal velocity pressure fluctuation as a linear function of the velocity fluctuation the mean and standard deviation of C_g are obtained as equal to the ones of the gust velocity, see e.g. JCSS, 2001. The aerodynamic shape factor C_a is standing for C_{pe} , C_{pi} , C_{fr} or C_f depending on the force component under consideration.

C_{pe} and C_f are best represented by a Fisher-Tippett type I distribution, see e.g. Cook and Mayne, 1979. Holmes and Cochran, 2003, propose a GEV distribution with slightly positive ξ (sub-exponential tail). Table A.7 summarizes the basic random variables representation, characteristic fractiles and partial safety factors.

Table A.7 Basic variables properties from and partial safety factors

	Description	Distribution	Bias	V	Characteristic fractile
$(c_r)^2$	Roughness factor	Lognormal	0,8 ⁽⁹⁾	10 - 20% ⁽⁵⁾⁽⁹⁾	$F_{C_r}(\mu_{C_r})$
$(c_g)^2$	Gust factor	Lognormal	1,00 ⁽⁹⁾	10 - 15% ⁽⁵⁾⁽⁹⁾	$F_{C_g}(\mu_{C_g})$
C_{pe}	External pressure coefficient	Gumbel	1,00 ⁽⁹⁾	10-30% ⁽⁶⁾ (14% for wind tunnel test)	80 % ⁽⁷⁾
C_{pi}	Internal pressure coefficient	Gumbel		10-30% ⁽⁵⁾	78 %
C_{fr}	Friction coefficient		1,00		78 %
C_f	Force coefficient	Gumbel	1,00 ⁽⁹⁾	10 - 15 % ⁽⁵⁾⁽⁹⁾	78 % ⁽⁵⁾
$\bar{U}_{max, 1yr}^2$	Mean wind pressure (1yr max)	Gumbel	1,00	20 - 40%	0,98

According to [6] the bias in the c_{pe} and c_f -coefficients are size dependent and given by:

20% underestimation for c_{pe1} (bias of approx. 1,20 for 1 m²)

20% overestimation for c_{pe10} (bias of approx. 0,80 for 10 m² and larger)

According to EN 1991-1-4 the coefficient of variation of $\bar{U}_{max, 1yr}^2$ may be taken as 0.23 if no site specific information is available. Further, the stochastic model for the wind action is calibrated such that the annual probability of exceedance of the calculated characteristic value does not exceed 0.02.

A simplification of the model may be obtained by combining all model uncertainties in one model uncertainty parameter θ :

$$q = \theta \left(\frac{1}{2} \rho \bar{U}_{max, 1yr}^2 \right) \quad (A.17)$$

where, for uncorrelated random variables (or weakly correlated), θ is lognormal distributed for the central limit theorem with mean equal to the product of the mean values, characteristic value equal to the product of the characteristic values and the coefficient of variation satisfying the following equation:

$$(1 + V_{\theta}^2) = (1 + V_{c_a}^2) (1 + V_{(c_r)^2}^2) (1 + V_{(c_g)^2}^2) \quad (\text{A.18})$$

This gives $V_{\theta q}$ between 0,17 (with all minimum coefficients of variation) and 0,40 (with all maximum coefficients of variation). The expected coefficient of variation may be taken around 0,28.

Note: the terrain category should also be introduced, variation of mean wind with height and intensity of turbulence, see JCSS PMC.

Model uncertainty does not cover uncertainty in thermal effects for special cases (buildings where temperature is intentionally kept below 0 °C, roofs with high thermal transmittance etc.) which should be noted.

References

JCSS (2001), JCSS Probabilistic Model Code – Part 1 – Basis of Design, Joint Committee on Structural Safety, <https://www.jcss-lc.org/jcss-probabilistic-model-code/> (periodically updated, online publication)

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A.3.4 Snow load

The model in accordance with prEN 1991-1-3:2020 (CEN, 2020a-b) is given by:

$$s_r = \theta \mu_i(C_e) C_t s_g \quad (\text{A.19})$$

where:

s_g snow load on the ground;

θ model uncertainty

μ_i conversion factor or snow load shape coefficient, depending on the roof’s geometry and the wind exposure of the site (through the exposure coefficient C_e)

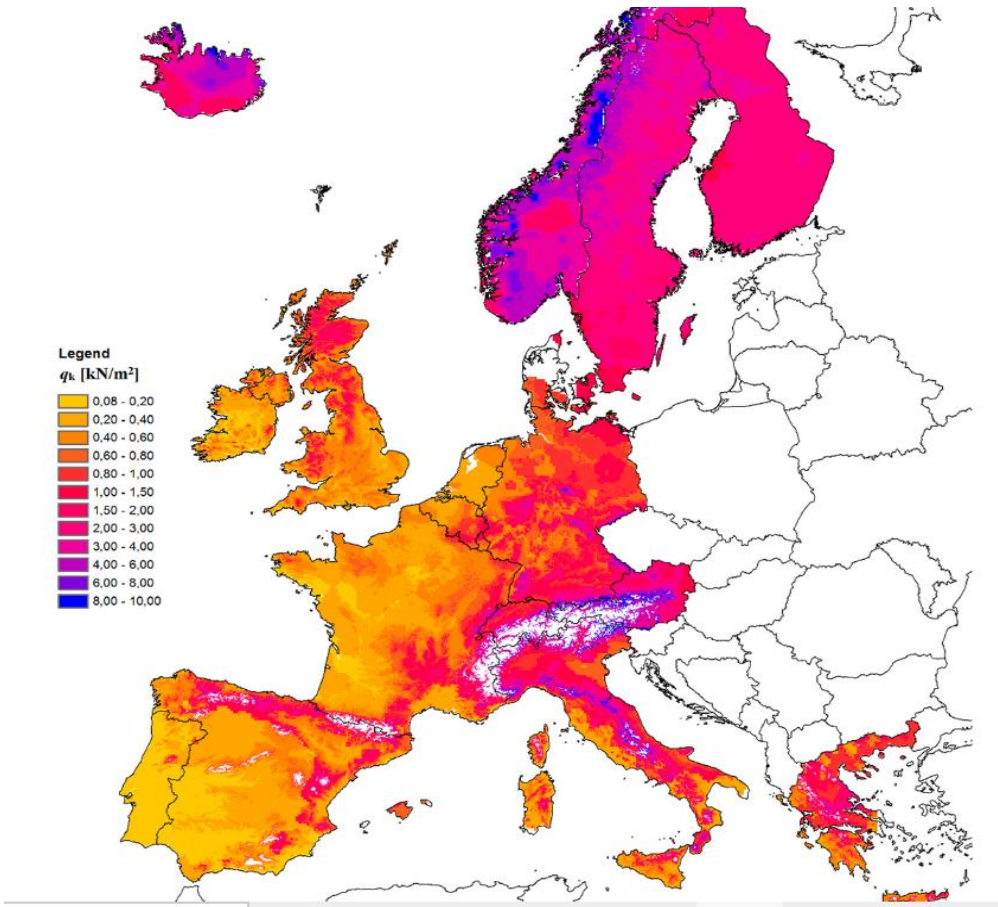
C_t is the thermal coefficient, to take account of the effects of heat flux through the roof.

The model uncertainty does not cover uncertainty in thermal effects for special cases (buildings where temperature is intentionally kept below 0°C, roofs with high thermal transmittance etc.) which should be noted.

Values for use in design calculations can be found in prEN 1991-1-3:2020 and or in the corresponding NAs. The following tables show the corresponding statistical properties. All values may be adjusted based on local data. Reference is made to the Background documents of SC1.T2 (CEN, 2020c) and to the Final Reports of the European Snow Load Research Project (Sanpaolesi et al, 1998 and 1999). Figure A.2 gives an impression of the

distribution of the load intensity over Europe. Note that here the characteristic values are presented which depend on the definition and rounding off rules used per country or region.

Figure A.2. European Ground snow load map (indicative characteristic values) defined by EN 1991-1-3, 2005 (source: Croce et al., 2019).



Source: Croce at al. 2019

The model for the time-invariant part of the action, i.e. the shape coefficient, which can be calibrated on the basis of available data for roof snow loads, is significantly affected by the shape of the roof and its wind exposure conditions and the climatic region. Therefore, uncertainties in the snow load model, which are related to the time-invariant part of the action, will be evaluated in two different ways depending on the level of detail:

CASE A:

By means of a single random variable, which covers all the uncertainties in the time-invariant part and will be represented by the model uncertainty θ assuming deterministic the shape coefficients. Statistical parameters of this random variable are evaluated considering the ratio between measured and modelled data of snow load on roofs for all available measurement collected in Thiis and O'Rourke, 2015. In the table below the statistical data for snow load model are illustrated.

Table A.8 Statistical data for snow load (CASE A)

Variable	X	Distrib.	Bias	V	X _k	Comments
Snow load on the ground (annual maximum)	S _{g1}	Gumbel	1,00	See Table A.10 Table A.10	X _{0,98}	See Thiis and O'Rourke, 2015 for ground snow load maps in Europe according to EN 1991-1-3 and National Annexes.

model uncertainty	θ	lognormal	0,81	0,26	$\mu + \sigma$	See N.71 "NewCEN model for predicting snow load on gable roofs" (Croce et al. 2019). It is estimated considering the ratio between measured and modelled data of snow load on gabled roofs in the Norwegian Case History Database (Thiis and O'Rourke, 2015) under different exposure conditions.
conversion from ground to roof snow load	$\mu_i(C_e)$	det	EN 1991-1-3:2020			
thermal coefficient	C_t	det	1,00		--	

CASE B:

Considering the specific statistical distribution of shape coefficients for the investigated roof shape, wind exposure condition, and climatic region, derived from available measurements. In the table below, statistical data are given for flat roofs (measurements taken from Sanpaulesi et al, 1998 and 1999, or CEN, 2020d) and gabled roofs in case of balanced load and unbalanced load (measurements taken from CEN, 2020d).

Table A.9 Statistical data for snow load (CASE B)

Variable		X	Distrib.	bias	V	X_k	Comments
Snow load on ground(annual maximum)		S_{g1}	Gumbel	1,00	See Table A.10 Table A.10	$X_{0,98}$	See (Croce et al., 2019) for ground snow load maps in Europe according to EN 1991-1-3 and National Annexes.
model uncertainty		θ	-	-	-	-	Model uncertainties in the time-invariant part of the snow load are directly taken into account through the statistical distribution of shape coefficients.
conversion from ground to roof snow load (flat roofs)*	Sheltered	$\mu_i(C_e)$	LN	0,94	0,09	$X_{0,58}$	*Flat roofs according to the data collected during the ESLRP (Sanpaulesi et al, 1998 and 1999). SC1.T2 DOC_087 (CEN, 2020b) shows the dependence of design conversion factors with the coefficient of variation of ground snow loads (CoV_g). X_k are evaluated considering a $CoV_g = 0,6$.
	Normal			0,92	0,39	$X_{0,76}$	
	Exposed			0,91	0,32	$X_{0,73}$	
conversion from ground to roof snow load (flat roofs)**	Sheltered	$\mu_i(C_e)$	LN	0,78	0,16	$X_{0,63}$	**Flat roofs according to the data collected in the Norwegian Case History Database (Thiis and O'Rourke, 2015). Data are available only for sheltered and normal exposure and are characterized
	Normal			0,83	0,16	$X_{0,63}$	

	Exposed			-	-	-	by lower CoV for normal exposure than that collected during the ESLRP (Sanpaolesi et al, 1998 and 1999). Design conversion factors is almost constant for both sheltered and normal conditions (CEN, 2020b).
conversion from ground to roof snow load (gabled roofs 25°<α<30°) Balanced load (case (i) prEN 1991-1-3: 2020)***	Sheltered	$\mu_i(C_e)$	LN	0,71	0,24	$X_{0,69}$	***Gabled roofs according to the data collected in the Norwegian Case History Database (Thiis and O'Rourke, 2015).
	Normal			0,73	0,29	$X_{0,71}$	
	Exposed			0,78	0,37	$X_{0,75}$	
conversion from ground to roof snow load (gabled roofs 25°<α<30°) Unbalanced load (case (ii) and (iii) prEN 1991-1-3:2020)***	Sheltered	$\mu_i(C_e)$	LN	0,73	0,25	$X_{0,68}$	***Gabled roofs according to the data collected in the Norwegian Case History Database (Thiis and O'Rourke, 2015).
	Normal			0,70	0,27	$X_{0,70}$	
	Exposed			0,70	0,35	$X_{0,74}$	
thermal coefficient		C_t	det	1,00		-	

LN stands for lognormal distribution

Table A.10 Coefficient of variation for yearly maximum snow load on ground (from Annex 3 in Sanpaolesi et al. 1998)

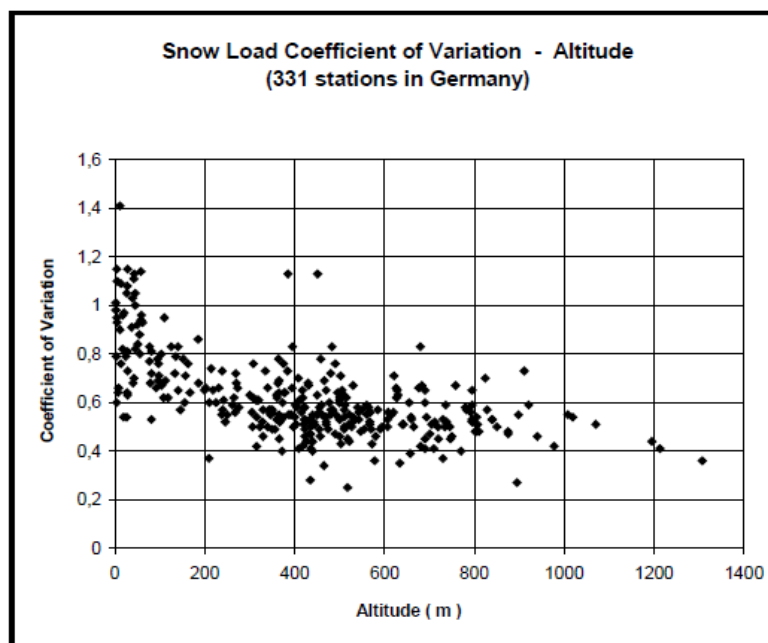
Region	min CoV_{S_G} min	m CoV_{S_G} mean	max CoV_{S_G} max	Number of repetitions in a year	Comments
Switzerland	0,4		1,0	2	<1000m
	0,2 (seldom below 0,3)		0,8	1	>1000m
Italy	0,5		1,0	1<r<2	<1000m
	0,4		0,7	1<r<2	>1000m
Germany	0,3 (seldom below 0,4)		0,7	1	No altitude dependency
Austria	0,3		0,8	1<r<2	Weak altitude dependency
France	0,4		0,45	1<r<3	Only two stations analysed
UK and EIRE	1,0		1,8	1	
	0,7		1,6	2	
	0,6		0,8	>3	
Iberian Peninsula	0,5		6,0	1	Mean value of V is equal to 2; majority of stations have CoV between 1,0 and 2,0
Mediterranean region	0,1 (0,4 for most stations)		1,5 (1,1 for most stations)	1<r<3	Do not depend on altitude
Central East	0,5		1,2		<100m
	0,3/0,4	0,5/0,6		1<r<6	>100m
Central West	0,6 (0,7 for most stations)		1,2 (1,0 for most stations)	1<r<2	
Greece	0,4	0,67	1,2	1<r<2	
Norway	0,2		0,6	1	
Finland Sweden	0,2		0,7	1<r<2	Weak altitude dependency
Iceland	0,3		1,3	1	No altitude dependency
Croatia*	0,5		0,7	1	Pannonian region
	0,4		0,7	1	Dinaric Alps region

	0,4		1,0	1	Mediterranean region
Czech Republic	0,3	0,6	1,2	$1 < r < 2$	Altitude dependent

* Croatian Meteorological and Hydrological Service (DHMZ) ** Czech Hydrometeorological Service (CHMI)

Sanpaolesi et al., 1998 concluded that, although there is a wide deviation, the value of 0,6 is calculated as the average from all the data. Also Soukhov, 1998 seems to support 0,6 as a representative value of CoV of annual ground snow load maxima for Germany. Aldea et al, 2016 indicate CoV between 0,35 and 1,38 for Romania, with an average value of 0,59. See also Figure A.3 for information.

Figure A.3. Coefficient of variation of annual maximum of snow load versus altitude



Source: Soukhov, 1998

References

CEN – European Committee for Standardization (2020a), *Eurocode 1: Actions on structures - Part 1-3: General actions - Snow loads*, EN 1991-1-3:2020 (Final draft)

CEN – European Committee for Standardization (2020b), *Background on design values of roof snow loads*, SC1.T2 DOC_087

CEN – European Committee for Standardization (2020c), *Background document to prEN 1991-1-3 (Final PT Document - April 2020)*, SC1.T2 DOC_095

CEN – European Committee for Standardization (2020d), *New CEN model for predicting snow load on gable roofs*, SC1.T2 DOC_071

Sanpaolesi L., Del Corso R., Formichi P., Currie D., Sims P., Sacré C., Stiefel U., Lozza S., Eiselt B., Peckham R., Solomos G., Holand I., Sandvik R., Gränzer M., König G., Sukohv D. (1998), *Phase 1 Final Report to the European Commission, Scientific Support Activity in the Field of Structural Stability of Civil Engineering Works: Snow Loads*, Department of Structural Engineering, University of Pisa, Italy

Sanpaolesi L., Del Corso R., Formichi P., Brettell M., Currie D., Dillon P., Sims P., Delpech P., Dufresne M., Sacré C., Stiefel U., Lozza S., Eiselt B., Peckham R., Solomos G., Holand I., Leira B., Sandvik R., Gränzer M., König G., Sukohv D. (1999), *Phase 2 Final Report to the European Commission, Scientific Support Activity in the Field of Structural Stability of Civil Engineering Works: Snow Loads*, Department of Structural Engineering, University of Pisa, Italy

Thiis T. K., O'Rourke M. (2015), 'Model for Snow Loading on Gable Roofs', *Journal of Structural Engineering*, Vol. 141, Issue 12, pp. 1-9.

Croce P., Formichi P., Landi, F. Marsili F. (2019), 'Harmonized European ground snow load map: Analysis and comparison of national provisions', *Cold Regions Science and Technology*, Vol. 168, 102875.

Soukhov, D. (1998), "The Probability Distribution Function for Snow Load in Germany", *LACER*, Volume 3

Aldea, A., S. Demetriu, D. Lungu, C. Neagu, R. Vacareanu and C. Arion, (2016), The snow loads code in Romania, in: Proceedings of ICSE 2016, CSTB - Centre scientifique et technique du bâtiment, Nantes, France

A.3.5 Traffic loads

For information reference is made to the JCSS Probabilistic Model Code.

References

JCSS (2001), JCSS Probabilistic Model Code - Part 2 (under development), Joint Committee on Structural Safety, <https://www.jcss-lc.org/jcss-probabilistic-model-code/> (periodically updated, online publication)

A.3.6 Accidental actions

Probabilistic models for accidental actions, in particular impact from vehicles, trains, ships and aircrafts and loads due to internal and external explosions may be found in Eurocode EN 1991-1-7 and ISO 10252. Relevant Background information is presented in (Vrouwenvelder and Stiefel, 2012). Table A.11 and Table A.12 give a summarized and indicative information on models for vehicle impact and internal natural gas explosions.

Table A.11 Indicative data for car impact

Random variable	X	Distr. type	Mean	V
Vehicle velocity - highway	v_0	Lognormal	80 km/h	0,12
Vehicle velocity - urban area		Lognormal	40 km/h	0,20
Vehicle velocity - courtyard		Lognormal	15 km/h	0,33
Vehicle velocity - parking garage		Lognormal	5 km/h	1,00
Deceleration after leaving the track	a	Lognormal	4,0 m/s ²	0,33
Vehicle mass – lorry	m	Normal	20 000 kg	0,6
Vehicle mass – car		Deterministic	1 500 kg	
Vehicle stiffness	k	Deterministic	300 kN/m	--
Angle of leaving the track	φ	Raleigh	10°	1,0°
Vehicle failure intensity	λ	Poisson	10 ⁻⁷ / m	

Table A.12 Indicative data for internal Natural Gas explosion

Random variable	X	Distr. type	Mean	V
Explosion pressure in buildings	p	Lognormal	EC-Model	0,7
Explosion rate	λ	Poisson	2x10 ⁻⁶ /a	

References

Vrouwenvelder A. and Stiefel U. (2012), *Eurocode 1, Part .1-7, Accidental Actions Background Document*, TNO, the Netherlands

A.3.7 Seismic actions

Seismic action is described as a response spectrum (usually spectral acceleration $S_a(T)$). This may be a so-called uniform hazard spectrum, or, more recently, a conditional spectrum. The former is the collection of spectral ordinates characterized by the same probability of exceedance (collection of fractile from their respective marginal distributions), while the latter is obtained from the joint distribution of spectral ordinates (i.e., it requires a correlation model) and represents more closely the spectrum of real motions conditional on a spectral ordinate taking a specific value. The distribution of spectral acceleration at a given vibration period T at a chosen site is obtained through probabilistic hazard analysis (Cornell, 1968, McGuire, 1995, Araya and Der Kiureghian, 1988, Bazzurro and Cornell, 1999, Woessner et al., 2015, GEM 2019), which employs the total probability theorem (Benjamin and Cornell, 1970) to obtain the probability of $S_a(T)$, being exceeded.

The classic probabilistic seismic hazard analysis (PSHA) describes the earthquake occurrence in time as a Poisson process, and the interarrival time τ as an exponential distribution, both having the parameter λ_0 when considering all seismic sources. The return period is the inverse of the mean annual rate of exceedance of a given ground motion level at a site, product of λ_0 times the probability of exceedance given an event of any magnitude on any of the sources affecting the site. The return periods for the significant damage (SD) and near collapse (NC) limit states are Nationally Determined Parameters in EN 1998, with recommended values equal to 475 years and 1600 years, respectively.

Table A.13 presents the relevant distributions to model earthquake occurrence in time, magnitude and space and the conditional distribution of $S_a(T)$, given a seismic event, describing the attenuation laws or ground motion prediction equations.

Table A.13 Seismic actions

Random variable	X	Distribution type	Distribution parameters ¹	X_k
Number of events in t	N_t	Poisson: $P(N_t=n) = \frac{e^{-\lambda_0 t} (\lambda_0 t)^n}{n!}, n=0,1,2,\dots$	λ_0 - annual mean rate of earthquakes occurrence	
Time till the first event	τ	Exponential: $P(\tau \leq t) = 1 - e^{-\lambda_0 t}$		
Magnitude	M	Gutenberg-Richter magnitude recurrence model for a seismic source $f_M(m) = \beta e^{-\beta(m-m_{min})}$ (Gutenberg and Richter, 1942),	m_{min} – lower bound magnitude. β - parameter governing the relative likelihood of earthquakes of various magnitudes	
Site-to-source distance	R	Uniform distribution for area seismic sources (simplified models)		

Spectral acceleration for a given event	$S_a(T)$	Lognormal distribution depending on the natural period T, for a given magnitude m and site-to-source distance r $P[S_a(T) < s m, r] = \Phi\left(\frac{\ln s - \mu_{\ln S_a}(m, r)}{\sigma_{\ln S_a}}\right)$	Mean - $\mu_{\ln S_a}$ Standard deviation - $\sigma_{\ln S_a}$	
Spectral acceleration due to all sources affecting the site	$S_a(T)$	Mixture of lognormal distributions obtained through PSHA		Return period [year] SD: 475 NC: 1600

¹ Seismic action parameters are site dependent.

References

- Araya, R., and Der Kiureghian, A. (1988), *Seismic hazard analysis improved models. Uncertainties and sensitivities*, Rep. UCB/EERC 90/11. California University, Berkeley, USA
- Bazzurro, P. and Cornell, C.A. (1999), 'Disaggregation of seismic hazard', *BSSA*, Vol. 89, pp. 501-520.
- Benjamin, J.R. and Cornell C., (1970), *Probability, statistics, and decision for civil engineers*, McGraw-Hill, New York, USA
- Cornell, C.A. (1968), 'Engineering seismic risk analysis', *BSSA*, Vol. 8, pp. 1503-1606.
- Global Earthquake Model Foundation – GEM (2019), The OpenQuake-engine User Manual, version 3.7.1., Global Earthquake Model Foundation, Pavia, Italy, doi: 10.13117/GEM.OPENQUAKE.MAN.ENGINE.3.7.1.
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- McGuire, R.K. (1995), 'Probabilistic seismic hazard analysis and design earthquakes: closing the loop', *BSSA*, Vol. 85, pp. 1275-1284.
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A.4 Material Properties

The statistical properties in this section are as much as possible in accordance with the data presented in material related Eurocodes or their background documents and may be considered as appropriate for performing reliability analysis in combination with the corresponding structural models.

A.4.1 Concrete

Table A.14 Concrete properties

X	description	Mean or bias factor	CoV V
	Concrete bending		
f_c	Compressive strength	$f_{cm}/f_{ck} = \exp(1,645V_{f_c})$	0,100
	Concrete shear		

θ_5	Model uncertainty (without shear reinforcement.)	$\mu = 1,10$	0,107
	Residual uncertainties (??)	-	0,046 ^{b)}

a) In-situ factor θ_5 accounts for the difference between the actual in-situ concrete strength in the structure $f_{c,ais}$ and the strength of the control specimen f_c .

b) The residual uncertainties (??) refer to aggregate size, reinforcement area and spacing and column size.

All Model Uncertainties for concrete bending have been removed and put together in A6.1

The higher qualities have usually a lower coefficient of variation.

References

CEN – European Committee for Standardization (2021), prEN 1992-1:2021 Part1, Annex A, Table A,2

A.4.2 Reinforcement Steel

Table A.15 Basic variables reinforcement

X	Description	Distr	Bias	V
f_y	yield strength	LN	$f_{ym}/f_{yk} = \exp(1,645V_{fy})$	0,045

References

prEN 1992-1 Part1, Annex A, Table A,2 (2021)

A.4.3 Structural Steel

Table A.16 Basic variables steel

X	Description	Mean or bias factor	CoV - V
f_y	yield stress – steel grades S235 to S420	$f_{ym}/f_{yk}^{a)} = 1 + 4,00 V_{fy}$	5,0%
	yield stress – steel grades S460 and above ^{b)}	$f_{ym}/f_{yk}^{a)} = 1 + 3,00 V_{fy}$	4,5%
f_u	tensile strength – steel grades S235 to S420	$f_{um}/f_{uk}^{a)} = 1 + 4,00 V_{fu}$	5,0%
	tensile strength – steel grades $\geq$ S460 ^{b)}	$f_{um}/f_{uk}^{a)} = 1 + 3,00 V_{fu}$	4,5%

^{a)} The design and the assessment of steel structures according to EC3 or EC4 do not make use of the 5% fractile as characteristic value of material strengths. The minimum guaranteed values are used instead, which are checked after production through specific testing. Due to quality control measures aimed at minimizing the sorting out of material, the minimum guaranteed value typically corresponds to a very low fractile of a PDF, below the 0,1% fractile.

^{b)} The current market share of steels of this strength in construction is below 10%.

For the E modulus the usual value is 210 000 N/mm².

References

CEN – European Committee for Standardization (2022), EN 1993-1-1:2022 Annex E (small differences possible)

A.4.4 Timber

The strength and stiffness properties of structural timber products are specified in the product standards EN 338 for solid timber and EN 14080 for glued laminated timber. Structural timber is graded according to different indicators and assigned to the different strength classes in the product standards based on the reference properties bending strength, bending MOE and density. The coefficient of variation differs for the

different timber products, strength grades and is dependent on the grading method. The strength classes C24 for solid timber and GL24h for glulam are typically taken as a reference.

Table A.17 Timber Strength models and parameters

Mechanical property		Distribution type	μ	V	Comment
Solid timber (EN 338)					
Bending strength	f_m	Lognormal	1,0	0,25	
Tension parallel to grain	$f_{t,0}$	Lognormal	1,0	0,30	
Tension perp. to grain	$f_{t,90}$	2-p Weibull	1,0	0,25	
Compression parallel	$f_{c,0}$	Lognormal	1,0	0,20	
Compression perp.	$f_{c,90}$	Normal	1,0	0,10	
Shear (parallel)	f_v	Lognormal	1,0	0,25	
MOE parallel (bending)	E_m	Lognormal	1,0	0,13	
MOE parallel (tension)	E_0	Lognormal	1,0	0,13	
MOE perpendicular	E_{90}	Lognormal	1,0	0,13	
Shear modulus	G_v	Lognormal	1,0	0,13	
Density	ρ	Normal	1,0	0,10	
Glulam (EN 14080)					
Bending strength	f_m	Lognormal	1,0	0,15	
Tension parallel to grain	$f_{t,0}$	Lognormal	1,0	0,10-0,15	(Frese 2016), JCSS
MOE (bending)	E_m	Lognormal	1,0	0,05-0,10	(Fink 2014; Frese 2016)
Density	ρ	Normal	1,0	0,10	

Source: [JCSS PMC Section 3.5.3.2](#) Table 2, if not otherwise stated

The coefficients of variation for engineered wood products are typically lower due to homogenization effects. Ranta-Maunus et al., 2001 report $V=0,10$ for softwood LVL in edgewise bending and $V=0,14$ for softwood LVL in flatwise bending.

References

- Fink, G. (2014), *Influence of Varying Material Properties on the Load-Bearing Capacity of Glued Laminated Timber*, PhD Thesis Nr. 21746, ETH, Zurich, Switzerland
- Frese, M. (2016), *Computergestützte Verfahren zur pragmatischen Beurteilung der Tragwiderstände von Brettschichtholz: Zusammenfassung exemplarischer Simulationsstudien*, KIT Scientific Publishing, <https://doi.org/10.5445/KSP/1000052710> (in German)
- Ranta-Maunus, A., Fonselius, M., Kurkela, J., and Toratti, T. (2001), *Reliability Analysis of Timber Structures - VTT Research Notes, 2109*, Technical research centre (VTT) of Finland, Espoo, Finland

A.4.5 Aluminium

Wrought aluminium alloys for structural applications according to EN 1999-1-1:2007 are shown in Table A.18.

Table A.18 Wrought aluminium alloys to be used in structural applications according to EN 1999-1-1:2007

Aluminium Alloys	Common tempers	Structural use in Europe
EN AW-3004	–	Not common
EN AW-3005	–	
EN AW-3103	–	
EN AW-5005/5005A	–	
EN AW-5049	–	
EN AW-5052	–	
EN AW-5083	O/H111, H24, H36, H116/H321	Common
EN AW-5383	H12/H22/H34	
EN AW-5454	–	
EN AW-5754	O/H111, H12/H22/H34	
EN AW-6060	T6	
EN AW-6061	T6	
EN AW-6063	T6, T66	
EN AW-6005A	T6	
EN AW-6082	T6, T4	
EN AW-6106	T6	
EN AW-7020	–	Not common
EN AW-8011A	–	

Statistical parameters: mean value (μ), coefficient of variation (V) and distribution types for material strength models of available aluminium alloys that are commonly used for structures in Europe may be taken from Table A.19. These statistical parameters have been obtained from a recent study performed within SC9 on more than 12 000 tension test results collected from producers, manufactures and design offices all over Europe.

Table A.19 Variability of available material properties

Parameter	Alloy EN AW	Temper	Mean value	Coefficient of variation	Distribution type
	5754	H111	1,56 $R_{p0,2,min}$	11 %	Lognormal

Y be taken from 2% proof strength, f_o	6005A	T6	1,18 $R_{p0,2,min}$	4,7 %	Normal
	6060	T6	1,25 $R_{p0,2,min}$	5,0 %	Lognormal
	6060	T66	1,28 $R_{p0,2,min}$	8,8 %	Lognormal
	6063	T6	1,26 $R_{p0,2,min}$	4,4 %	Normal
	6063	T66	1,19 $R_{p0,2,min}$	4,1 %	Normal
	6082	T6	1,20 $R_{p0,2,min}$	6,2 %	Normal
Ultimate tensile strength, f_u	5754	H111	1,14 $R_{m,min}$	3,4 %	Lognormal
	6005A	T6	1,07 $R_{m,min}$	2,8 %	Lognormal
	6060	T6	1,15 $R_{m,min}$	5,3 %	Lognormal
	6060	T66	1,07 $R_{m,min}$	6,5 %	Lognormal
	6063	T6	1,09 $R_{m,min}$	3,0 %	Lognormal
	6063	T66	1,07 $R_{m,min}$	3,3 %	Lognormal
	6082	T6	1,12 $R_{m,min}$	5,0 %	Lognormal
Modulus of elasticity, E	All	All	69943 MPa	5,5 %	Lognormal
Poisson ratio, ν	All	All	0,33	3,3 %	Normal
$R_{p0,2,min}$ and $R_{m,min}$ are the minimum yield strength $R_{p0,2}$ and the lower bound of the ultimate tensile strength R_m , according to the applicable product standard, e.g. of the EN 755 or EN 485 series.					

References

Skejić, D.; Dokšanović, T.; Čudina, I.; Mazzolani, F.M. (2021), 'The Basis for Reliability-Based Mechanical Properties of Structural Aluminium Alloys', *Applied Sciences*, Vol. 11, 4485, <https://doi.org/10.3390/app11104485>

A.4.6 Masonry

For the analysis of masonry structures the data in Table A.20 may be used.

Table A.20 Probabilistic model for generic Masonry

X		description	Mean/char value	CoV
f_c		Masonry Compressive strength	$f_{cm}/f_{ck} = \exp(1,645 V_{f_c})$	0,16

References

Jäger, W. and Pech, A. (2015), 'Compressive strength of masonry – statistical evaluation of test data by power functions as basis of EC6, in: Jäger, W. (ed), *Mauerwerk-Kalender 40*, Ernst & Sohn, Berlin, Germany pp. 423 - 448

A.4.7 Ground

The prEN1997 provides ranges of the coefficient of variation for various ground properties, see Table A.21.

Table A.21 Indicative ranges of coefficient of variation of ground properties from EN1997-1 Annex A (representing spatial variability, i.e. no spatial averaging is included yet)

Ground/rock type	Ground property	Symbol	CoV (X)
Fine-grained soils	Shear strength in total stress analysis	c_u	30-50
All soils and rocks	Peak or residual effective cohesion	c'_p, c'_r	30-50
All soils and rocks	Angle of friction	φ	5-15
All soils and rocks	Shear strength at failure	z_f	15-25
All soils and rocks	Unconfined compressive strength	q_u	20-80
All soils	Modulus of deformability	E or G	20-70
Fine-grained soils	Vertical or horizontal consolidation coefficient	c_v or c_h	30-70
All soils	Hydraulic conductivity	k	70-250

These indicative CoV values) have recently been supported in broad literature study by ISSMGE-TC304 (2020) to reasonably represent the point-to-point variability of individual sites. It is important to notice, however, that a geotechnical design parameter refers to larger volumes of ground involved with the occurrence of a limit state. Since spatial averaging takes place, the geotechnical design parameter typically refers to the spatial mean of a larger volume (with a few exceptions), including the related statistical uncertainty. Notice that the amount of averaging is therefore application dependent, depending on the volume of ground involved. See section 3.5.6 in the report for more details.

Table A.21a presents recommended values for coefficients of variation of the most important ground strength parameters for calibration purposes, taking into account that the variables of interest in calibration studies more often than not refer to spatial averages. That is the reason why the recommended CoV values in the third column are sometimes lower than the ranges of the values reported in Table A.21a.

Table A.21a Typical coefficients of variation representing the uncertainty in spatial average ground strength parameters, recommended for calibration of partial factors

	Distribution	CoV*
Angle of friction φ'	Lognormal	0,10
Effective Cohesion c'	Lognormal	0,20
Undrained shear strength c_u	Lognormal	0,20

* The CoV refers to the uncertainty in the geotechnical parameter, meaning that spatial averaging and statistical uncertainty are considered.

It is important to notice that there is a large variety of (strength-related) parameters involved with geotechnical design, and that the uncertainty is related to the type and amount of site investigation (e.g. laboratory versus in-situ testing). Table A.21 merely reflects typical values of the most important properties. More detailed information on statistics of soil properties can be found in the literature; the literature review by ISSMGE-TC304 (2020) provides a useful overview.

References

International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) - Technical Committee TC 304 'Engineering Practice of Risk Assessment and Management' (2020), *State-of-the-art review of inherent variability and uncertainty in geotechnical properties and models*, Download:

A.5 Imperfections and eccentricities

For the time being calculations will be based on current design values in the codes.

A.6 Model Uncertainties for load effect calculation

The model uncertainty for the load effect calculation θ_E is primarily depending on the type of internal force (axial forces, shear forces etc.) considered.

As far as the global load effects are concerned and assuming the use of the computer codes according to the state of the art (common practice, in principle Eurocode models), the values according to Table A.22 may be considered as recommended.

Table A.22 Model uncertainties for the global analysis; the correlation ρ is between members

Structural type	Load effect type	Bias	CoV	ρ
Frames statically determinate	N, M, D	1,0	0,05	0,8
Frames statically indeterminate	N, M, D	1,0	0,10	0,8
Plate structures	m	1,0	0,20	0,8
Plate structures	n	1,0	0,10	0,8
Shells	m, n	1,0	0,20	0,8
2D solids	σ	1,0	0,10	0,8
3D solids	σ	1,0	0,15	0,8

All model uncertainties may be assumed to have a lognormal distribution.

It is assumed here that applied FEM models have appropriate geometrical and physical linear or nonlinear features. In case of geometrical nonlinear analysis relevant initial imperfections should be included. In case of physical nonlinearities care is taken of deformation capacity and possible path dependence. Usually the fulfilment of code requirements avoids buckling and/or brittle behaviour with sufficient reliability.

References

JCSS (2001), JCSS Probabilistic Model Code, Joint Committee on Structural Safety, <https://www.jcss-lc.org/jcss-probabilistic-model-code/> (periodically updated, online publication)

A.7 Model Uncertainties for (local) resistance models

Model uncertainties are defined according to: (not including parameter uncertainty in principle)

$$\theta_R = R_{\text{experiment}} / R_{\text{model}}$$

The statistical properties of a model uncertainty are always connected to a specific model and a well-defined population of structures for which it has been derived and will be used. For the determination of the statistical properties reference is made to EN 1990:2023, Annex D.

Note that theoretically model uncertainties do not include the uncertainties or scatter in the other basic variables. However, in practice this is not always possible resulting in some double counting of uncertainties.

Uncertainties regarding resistance, θ_R , are dependent on both the material and the type of internal force considered. All model uncertainties in this chapter are assumed to have a lognormal distribution.

A.7.1 Reinforced concrete

Table A.23 Model uncertainty related to the resistance model for concrete

	Relative frequency	Random variable	Mean	CoV
	w_k		μ_θ	V_θ
Bending moment	30%	$\theta_{R, mod, ik}$	1,035	7% *
Shear	30%	$\theta_{R, mod, ik}$	1,40	25,0%
Axial force	10%	$\theta_{R, mod, ik}$	1,00	5,0%
Bending in plates	10%	$\theta_{R, mod, ik}$	1,20	15,0%*
Buckling	20%	$\theta_{R, mod, ik}$	1,40	25,0%

*Including position of the reinforcement

More in detail, for bending use has been made of (see EN 1992-1, Annex A):

Table A.24 Detailed background information for resistance model for concrete

X	Description	Distr	Bias	CoV
bending				
d	Effective depth	LN	$md = 1 - 0,05(200/d)^{2/3}$	$Vd = 0,05(200/d)^{2/3}$
d	Effective depth d	LN	$\mu = 0,95$	0,050
θ	Model uncertainty	LN	$\mu_{\theta 5} = 1,09$	0,045
Compression				
η_s	In-situ factor ($= \frac{f_{c, als}}{f_c^a}$)	LN	$\mu = 0,95$	0,120
A_c	Concrete area	LN	$\mu = 1,00$	0,040
θ_c	Model uncertainty steel	LN	$\mu = 1,02$	0,060'

In total resulting in $m = 1,07$ en $CoV = 0,14$, as indicated in the first table

References

CEN – European Committee for Standardization, EN 1992-1, Annex A

CEN – European Committee for Standardization, *Background document FprEN 1992-1-1 (final version), 2023-04-05*, CEN-TC 250-SC 2_N2087

A.7.2 Steel

Parameters for the model uncertainty in steel structures designed to EC3 or (where pertinent) EC4 are presented in Table A.25; the values are based on data found e.g. in the reference list.

Table A.25 Model uncertainty related to the resistance model for steel

Load or ULS limit state	effect	Relative frequency	Random variable	Mean	CoV
		$W_k^{a)}$		$\mu_\theta^{b)}$	$V_\theta^{b)}$
Cross-sectional resistances – no buckling, associated with f_y and γ_{M0}					
Bending – plastic, compact, elastic sections	15%	$\theta_{R,mod,ik}$	1,225	7,5%	
Shear	3%		1,40	10%	
Axial force - yielding	15%		1,00 ^{d)}	– ^{d)}	
Torsion	2%		1,40	10%	
Cross-sectional resistances – slender sections, local buckling, associated with f_y and γ_{M1}					
Bending moment and shear	7,5%	$\theta_{R,mod,ik}$	1,15	5%	
Axial compression	7,5%		1,10	4%	
Member resistances - global buckling, associated with f_y and γ_{M1}					
Column or compression member buckling	15%	$\theta_{R,mod,ik}$	1,15	5%	
Lateral-torsional buckling	15%		1,20	7,5%	
Local section or connector resistances - associated with f_u and γ_{M2}					
Net section and bolt bearing failure	7,5%	$\theta_{R,mod,ik}$	1,10	5%	
Welds	7,5%		1,10	5%	
Bolts, studs and other mechanical connectors	5%		1,10	5%	

^{a)} Weights are based on best estimates of dominant ultimate limit states in common building, bridge and industrial structures

^{b)} The mean value of the model uncertainty is defined as the average of the ratio between available test results and strength predictions given by the corresponding Eurocode design model, the corresponding CoV as the coefficient of variation of this ratio

^{c)} Recommended values for the partial factors are: γ_{M0} =1,0 ; γ_{M1} =1,0 or 1,1 (depending on application); γ_{M2} =1,25

^{d)} The model uncertainties associated with axial force yielding are covered by the uncertainties for material and geometry

The resulting weighted $\theta_{R,mod,ik}$ for structural steel has a mean of ca. $\mu=1.15$ and a CoV of 5%.

Note: Model uncertainties for buckling may depend on the eccentricities used in the model: a large value for the eccentricity will lead to a higher average of the model uncertainty.

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A.7.3 Timber

(Sørensen, Svensson, and Stang 2005) report values $CoV_{\theta}=0,05$ for bending and short term load duration. (Kohler 2006) reports $CoV_{\theta}=0,05-0,10$ for bending and short term load in addition $CoV_{\theta}=0,10$ for bending and long term load duration including damage accumulation.

Table A.26 Model uncertainty related to the resistance model associated with the load effect for timber

	w_k	variable	μ_{θ}	CoV_{θ}
Bending moment	30 %	$\theta_{R, mod, ik}$	1,00	10,0%
Shear	30 %	$\theta_{R, mod, ik}$	1,00	15,0%
Axial force	10 %	$\theta_{R, mod, ik}$	1,00	5,0%
Buckling	30 %	$\theta_{R, mod, ik}$	1,00	15,0%
Joints with laterally loaded dowel type fasteners / European yield model:				
-embedment failure		$\theta_{R, mod, ik}$	0,8	10,0%
-failure with two plastic hinges in the fasteners per shear plane		$\theta_{R, mod, ik}$	1,25	10,0%

The resulting $\theta_{R,i}$ for timber has mean 1,00 and coefficient of variation 0,15.

A considerable model uncertainty is reported for connections including a bias ($\mu_{\theta} \approx 0,8-1,3$ for single fasteners and $\mu_{\theta} \approx 1,25$ for multiple fasteners) and variability ($CoV_{\theta} \approx 0,07-0,10$ for single fasteners and $CoV_{\theta} \approx 0,20$ for multiple fasteners), e.g. Kohler, 2006 and Jockwer, Fink, Köhler, 2018.

Other examples are the design of notched beams with fracture in tension perpendicular to grain and shear (Jockwer et al. 2011) with $\mu_{\theta} \approx 0,8$ and $CoV_{\theta} \approx 0,25$.

References

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A.7.4 Masonry

Information on the model uncertainty for masonry under compression and buckling can be found in Bakeer & Salehi, 2019 and Brehm, 2011; a summary is presented in Table A.27.

Table A.27 Summary of the stochastic models of the model uncertainties for masonry

Failure mode	Unit	Model	Dist.	m	σ	CoV
Compression θ_{fc}		Bakeer & Salehi 2019	Lognormal	0,97		16%
Diagonal tension θ_{dt}						
	AAC	DIN EN 1996-1-1/NA		1,14	0,20	17%
	CB			1,03	0,18	17%
Sliding shear θ_s	CS	DIN EN 1996-1-1/NA		1,15	0,24	21%
	CB	DIN EN 1996-1-1/NA		1,24	0,24	19%
Flexure θ_f	CS	Fully-plastic ^a		1,00	0,18	18%
	AAC			1,05	0,16	15%
	CB			1,10	0,20	18%
Shear crushing θ_c	all	DIN EN 1996-1-1/NA		1,00	0,20	20%
^a stress-strain relationship						

^a See also (Salehi 2018).

References

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Salehi, H. (2018), 'Determination of partial factor for model uncertainty for unreinforced masonry sheer wall', *European Masonry Journal*, Vol. 22, Issue 2, pp. 103 -112

A.7.5 Aluminium

A list of model uncertainty statistics related to the resistance models associated with the load effects, i.e., typical failure modes of aluminium members is given in Table A.28. Relative frequency values are assumed, and they should be obtained from real design situation when considering relevant structural member.

Table A.28. Model uncertainty related to the resistance model associated with the typical failure modes of aluminium member

Resistance / failure mode	Relative frequency	Random variable	Mean	CoV
	w_k^1		μ_θ	V_θ
Axial resistance of cross-section	10%	$\theta_{R, mod, ik}$	1,00	0,0%

Bending resistance of cross-section ²	30%	$\theta_{R, mod, ik}$	1,27	14,9%
Shear resistance of cross-section	5%	$\theta_{R, mod, ik}$	1,00	5,0%
Buckling resistance of members in compression ³	20%	$\theta_{R, mod, ik}$	1,18	14,6%
Buckling resistance of members in bending ⁴	25%	$\theta_{R, mod, ik}$ ⁴	1,00	10,0%
¹ Each load effect is sampled with a different relative frequency, w_k . The values are assumed. ² Including the effects of normal and shear forces. The values are obtained from tests related to bending of extruded tubes (EN AW 6061-T6 and 6063-T5). ³ The values are obtained from tests related to buckling of extruded tubes and I-sections (EN AW 6061-T6, 6063-T5 and 6082-T6). ⁴ Including the effects of axial compression. ⁵ For lateral-torsional buckling of unsymmetrical extruded profiles, for resistance model, $\theta_{R, mod, ik}$, the mean of 1,19 and coefficient of variation of 0,08 are obtained (Doksanovic, PhD). The distribution is lognormal. When results are separated by relative slenderness areas, the values are 1,27/0.06 (less than 1,2) and 1,13/0.03 (higher than 1,2).				

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A.7.8 Ground

Resistance models for geotechnical structures refer to a large variety of structures, some of which are part of larger structures (e.g. foundations of buildings and bridges), while others are structures on their own (e.g. embankments, retaining structures). For the main purpose of this annex, namely calibration of partial factors for typical structural members, Table A.29 contains a list of model uncertainty statistics for foundation elements (expert judgement). More information on model uncertainty for geotechnical applications for broader use in reliability and risk analyses can be found in the literature; ISSMGE-TC304 (2020) contains a valuable literature review.

Table A.29 Model uncertainty related to the resistance model associated with the load effect for ground

	w_k	Random variable	μ_θ	COV_θ
Spread foundation – bearing capacity		$\theta_{R,mod,ik}$	1,00	0,15
Piled Foundation – vertical bearing capacity (*)		$\theta_{R,mod,ik}$	1,00	0,20

Reference

International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) - Technical Committee TC 304 'Engineering Practice of Risk Assessment and Management' (2020), State-of-the-art review of inherent variability and uncertainty in geotechnical properties and models, Download: http://140.112.12.21/issmge/2021/SOA_Review_on_geotechnical_property_variability_and_model_uncertainty.pdf

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Annex B. Applications and Examples

B.1 Introduction

This annex contains examples and applications intended to clarify parts of the main report. It is intended to be helpful in checking or modifying reliability related National Determined Parameters by NSB's. Some examples might also prove useful when advanced reliability methods are directly applied in practical design and assessment procedures.

B.2 Reliability based background information on partial factors

B.2.1 Results for design format EN 1990:2023-(8.12)

B.2.1.1 Optimization method

In chapter 3.4 of the main text it was concluded that using the partial factors currently proposed in Eurocode EN1990 the average reliability level for a reference period of 50 year is close to $\beta_t = 3,8$ which is formulated as the target value. However, the scatter around this value is quite large and the question arises whether another choice of partial factors could lead to the same average but less scatter, that is could lead to less error in a number of individual cases.

In this section an example of that type is presented for the derivation of partial factors for the permanent and variable loads, on the basis of chapter 2.6.2 in the main part of this document and introduced in EN1990 C.4.4.3. The object of the procedure is to minimize the weighted difference of squared deviations::

$$\min \sum w_i (\beta_i(\gamma) - \beta_t)^2 \quad (B.1)$$

The data for this example are the same as presented in the Tables 3.2, 3.3 and 3.7 of the main text and used for the calculations in chapter 3.4. That is:

Design format EN 1990 equation (8.12)

Partial resistance factors γ_M according to the current Eurocodes as presented in Table 3.2;

Load ratios a_Q , a_G and weight factors w as presented in Table 3.3;

Probabilistic models according to Table 3.7.

The target reliability is $\beta_t = 3,8$ for a period of 50 year as is the basis in the Eurocode. The resulting partial factors on the basis of equation B.1 and the data from chapter 3.4 mentioned above are:

$\gamma_G = 1,20$ for the self-weight and

$\gamma_Q = 1,94, 2,35$ and $1,60$ for respectively wind, snow and imposed load.

Figure B.1 shows the violin plots for the calibrated values (right hand side and green bars) and the standard Eurocode values $\gamma_G = 1,35$ and $\gamma_Q = 1,50$ (left hand side and blue bars). As could be expected, the scatter for the calibrated values is lower than for the classical Eurocode values because for the assumptions inherent in this example, the partial factors have been optimised to minimise the scatter.

If a system with only one common partial factor for variable loads is chosen, the optimal values $\gamma_G = 1,20$ for the self-weight and $\gamma_Q = 1,75$ for wind snow and imposed load are obtained. In that case the scatter is in between two results shown in Figure B.1.

Figure B.1. Violin plots for the scatter in resulting β -values

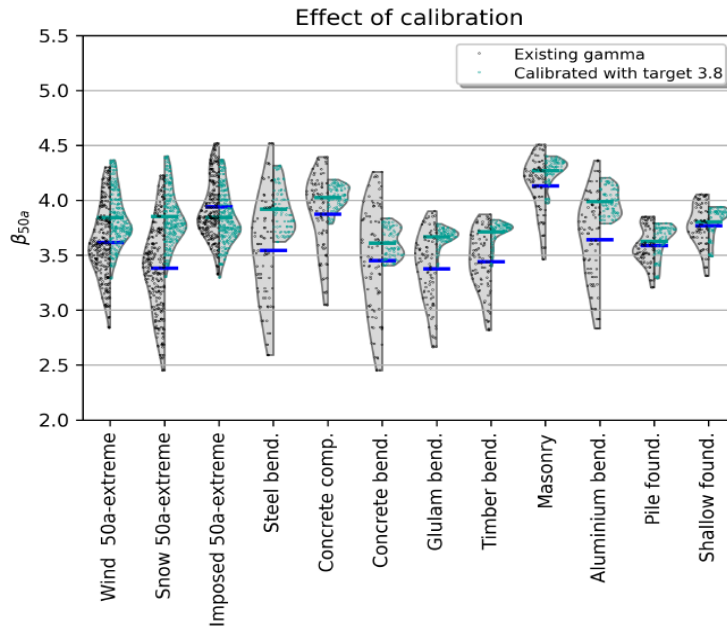
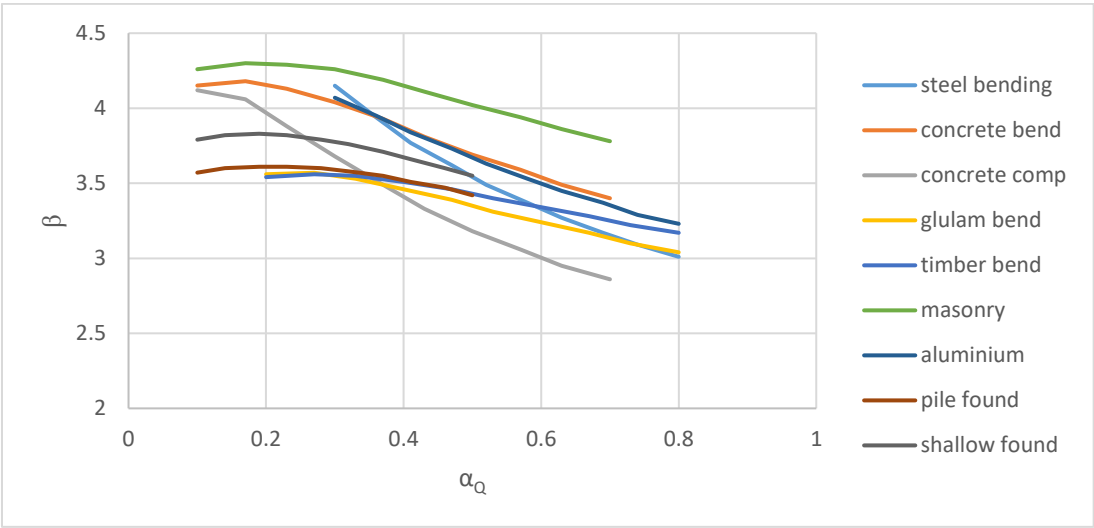
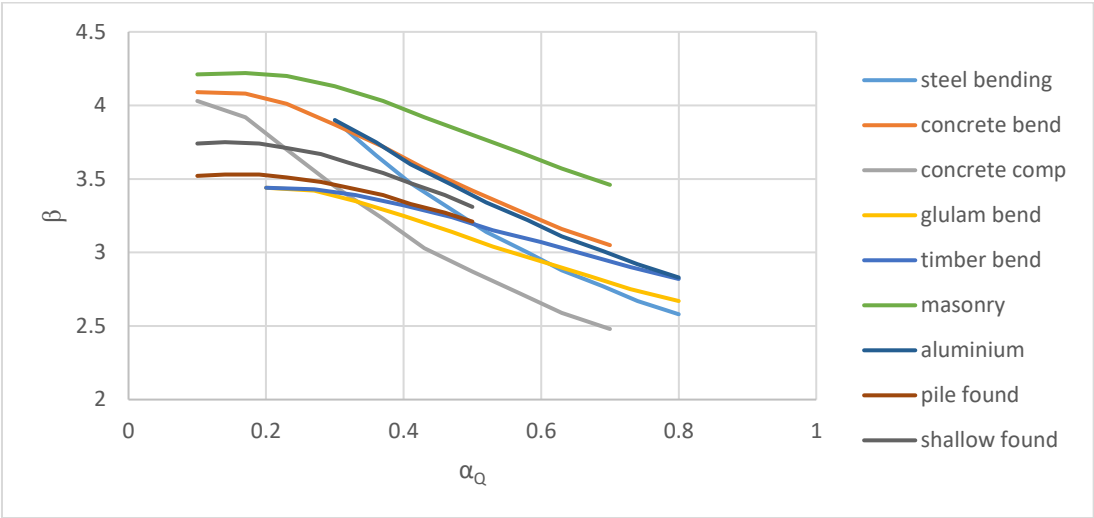


Figure B.2 shows the corresponding β -values as function of a_Q for wind, snow and live load for the non-optimized case. Each line corresponds to one of the materials.

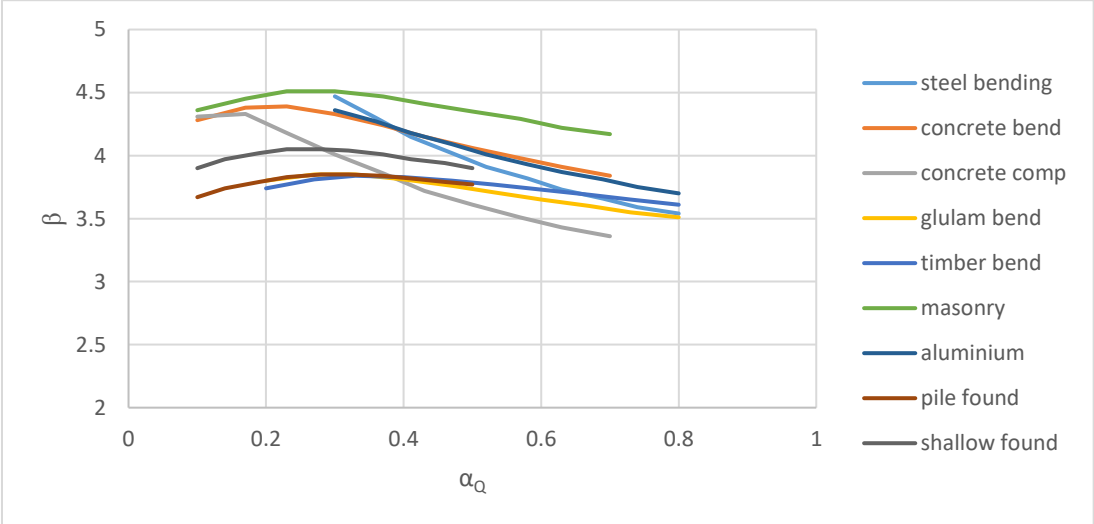
Figure B.2. Non-optimized Reliability index β as function of α_Q for wind, snow and live load ranging from 2.5 to 4.5



wind



snow



Imposed load

B.2.1.1 Design Value Method

As mentioned in chapter 2.6.1 of the main text and as introduced in EN 1990 C.4.4.2 partial factors can also be obtained using the **Design Value method (DVM)** which in the Eurocode is commonly used to derive the partial factors on the resistance side. Based on the same probabilistic models as above the following results for the partial factors for variable loads are obtained (the values in parenthesis are with the load effect uncertainty θ_{cl} included):

Wind load:

Characteristic value $Q_k = 1,08$, Design value $Q_d = 2,05$ implying a partial factor $\gamma_Q = 1,89$ (1,96)

Snow load:

Characteristic value $Q_k = 0,82$, Design value $Q_d = 1,86$ implying a partial factor $\gamma_Q = 2,27$ (2,31)

Imposed load:

Characteristic value $Q_k = 1,35$, Design value $Q_d = 2,08$ implying a partial factor $\gamma_Q = 1,54$ (1,60)

where:

The characteristic value Q_c for the wind and snow loads are based on $P(\theta_Q Q_{1a} \leq Q_c) = 0,98$ i.e. the annual probability of exceedance of the load $[\theta_Q Q_{1a}]$ is 0,02, where Q_{1a} is the annual maximum of the time-variant part of the load and θ_Q is the time independent load model uncertainty.

The design value Q_d for loads is based on $P(\theta_Q Q_{50a} \leq Q_d) = \Phi(0,7 \beta_t)$, where the 50 year target reliability index $\beta_t = 3,8$.

It can be concluded that the results are close to the result of the optimization method. For further comments reference is made to section B.2.2.5. Note also that the results are similar to the results in the Background document to the earlier version of the Eurocode (PT Eurocode 1.1 and JCSS, Background Documentation, Eurocode 1 (ENV1), Part1 Basis of Design, ECCS No 94, March 1996).

B.2.2 Results for design formats EN1990:2023 (8.13a/b)

Using the same models as in section B.2.1 we may derive reliability levels for designs based on EN 1990 (8.13a/b) with $\xi = 0,89$ (which corresponds to $\xi \gamma_G = 1,20$, as used in some countries), $\gamma_Q = 1,5$ and ψ_{01} values equal to 0,6, 0,5 and 0,7 for respectively wind, snow and imposed loads.

Table B.1 Average reliability indices per material and action

	wind	snow	imposed
Steel bending	3,26	2,88	3,71
Concrete compression	3,55	3,31	3,90
Concrete bending	3,04	2,75	3,46
Glulam bending	3,08	2,80	3,48
Solid timber bending	3,17	2,91	3,54
Masonry	3,86	3,65	4,15
Aluminium bending	3,38	3,05	3,79
Pile foundation	3,24	3,09	3,52
Shallow foundation	3,41	3,24	3,69

The overall result can be characterized by $\beta = 3,45$ (crude average) and $\beta = 3,37$ (using the weights w_i as specified in the Table 3.3). As expected the result is lower than for design based on (8.12).

Calibrating the partial factors in this case leads to:

$\gamma_G = 1,30$ for the self-weight and

$\gamma_Q = 2,05, 2,50$ and $1,65$ for respectively wind, snow and imposed load.

When imposing a common partial factor for all variable loads one obtains:

$\gamma_G = 1,33$ for the self-weight and

$\gamma_Q = 1,78$ for wind, snow and imposed load.

The target reliability is $\beta_t = 3,8$ for a period of 50 year as is the basis in the Eurocode.

B.2.3 Results for annual reliability and design formats EN1990:2023-(8.12)

In section B.2.1 the reliability index for the 50 year period was calculated. As explained in EN 1990-Annex C the $\beta = 3,8$ for this period is the anchor point for the Eurocode reliability target. However, in some applications and also as a choice in some member states, the annual reliability level is preferred. Using the same models as in section B.2.1 (but of course adjusted to annual probabilities where relevant) we may calculate for the annual reliability levels according to Table B.2.

Table B.2 Average reliability indices per material and action

	wind	snow	imposed
Steel bending	4,36	4,10	4,44
Concrete compression	4,50	4,37	4,60
Concrete bending	4,25	4,08	4,33
Glulam bending	4,10	3,95	4,21
Solid timber bending	4,11	3,98	4,23
Masonry	4,67	4,57	4,77
Aluminium bending	4,42	4,22	4,50
Pile foundation	4,07	4,02	4,20
Shallow foundation	4,27	4,22	4,39

The overall result can be characterized by $\beta = 4,36$ (crude average) and $\beta = 4,29$ (average using the weights w_i as specified in the Table 3.3 of the main text). The average annual reliability index is close to 4,3, i.e. much lower than the upper bound value 4,7 mentioned in EN 1990 Annex C. As to be expected the lowest values close to 4,0 occur for foundations and the highest (masonry under imposed loads is close to the upper bound).

B.2.4 Results sensitivity studies

In order to get a feeling for the sensitivity of the calibration results to some of the input assumptions, a number of additional analyses has been made based on modifications in section 3.4. The results have been summarized in table B.3:

Table B.3 Overview of sensitivity studies

	COV wind	α_Q distribution	Procedure	Gamma perm loads	Gamma variable loads	WEIGHTED AVERAGE BETA	AVERAGE BETA
All materials/all loads							
1	0,14	uniform	fixed	1,35	1,50	3,74	3,64
2	0,12	uniform	fixed	1,35	1,50	3,79	3,68
3	0,17	uniform	fixed	1,35	1,50	3,59	3,67
4	0,14	uniform	optimized ^a	1,20	1,75	3,78	3,68
5	0,12	uniform	optimized	1,20	1,70	3,78	3,67
6	0,17	uniform	optimized	1,2	1,85	3,81	3,73
7	0.14	uniform	optimized	>1,25	1,70	3,80	3,70
8	0.14	uniform	optimized	>1,30	1,65	3,83	3,73
9	0.14	triangular	optimized	1,20	1,75	3,81	3,68
10		linear	optimized	1,25	1,65	3,82	3,65
Combination "Steel-wind" only							
11	0,12	uniform	fixed	1,35	1,50	3,71	3,71
12	0,12	Uniform	Optimized	1,10	1,75	3,78	3,78

^a optimized is always for the weighted average

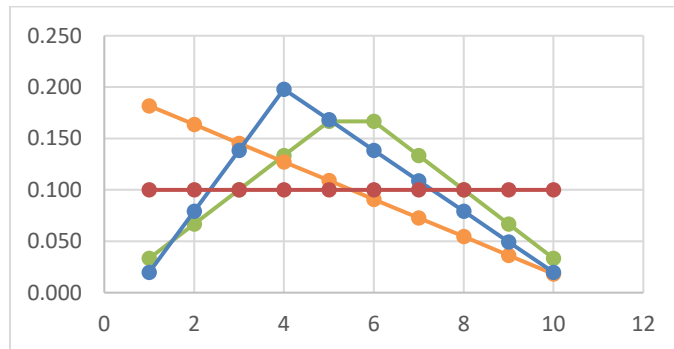
Calculation 1 (C1 for short) on the first line is the standard calculation as presented in Chapter 3.4. It shows that the Eurocode partial load factors 1,35 and 1,50 lead to an average reliability index close to the target 3,8, both for the assumed and for the constant w-factors (last column). This shows the relative moderate influence of the weight factors. The lines C2 and C3 show the sensitivity to the uncertainty in the wind model by changing the CoV from 0,14 (standard) respectively down to 0,12 and up to 0,17. As expected the beta-values go up for a lower wind uncertainty and down for a larger one. However, the result is not dramatic.

The lines 4-6 give the result of the partial factor optimization procedure for the same cases with optimized partial factors rounded off to a discretization with 0,05. The factors for snow, wind and imposed load are forced be the same like in the Eurocode EN1990:2021. The resulting beta-values are a bit closer to the target of 3,8, but also the scatter in the results is less. This can be seen in the violin plots in figure B1 where the scatter of Beta of the EN1990 factors is on the left side and the optimized values on the right hand side, although here all partial factors for the variable loads are optimized individually.

Calculations C7 and C8 show the results if in the optimization procedure the factor for the permanent load is constrained to have values larger than 1,25 respectively 1,30. The optimization procedure then leads to the minimum allowed values for the permanent loads and to 1,70 and 1,65 for the variable loads. Of course the scatter in the results increases if these higher values for the permanent load is imposed.

In the basic calculations the α_Q values (governing the ratio of permanent and variable load) for all have been given an equal weight. In C9 and C10 the shape of the α_Q distribution is changed from uniform to triangular and linear (see Figure B.3). Only assuming a full linear distribution has a substantial influence and makes the partial factor for self-weight rise. Note that this distribution gives a high weight to a very low relative contribution of the total load and is only valid for soil and a specific subset of concrete structures.

Figure B.3. Examples of α_Q distribution (uniform, triangular symmetric, triangular 0,4, linear)



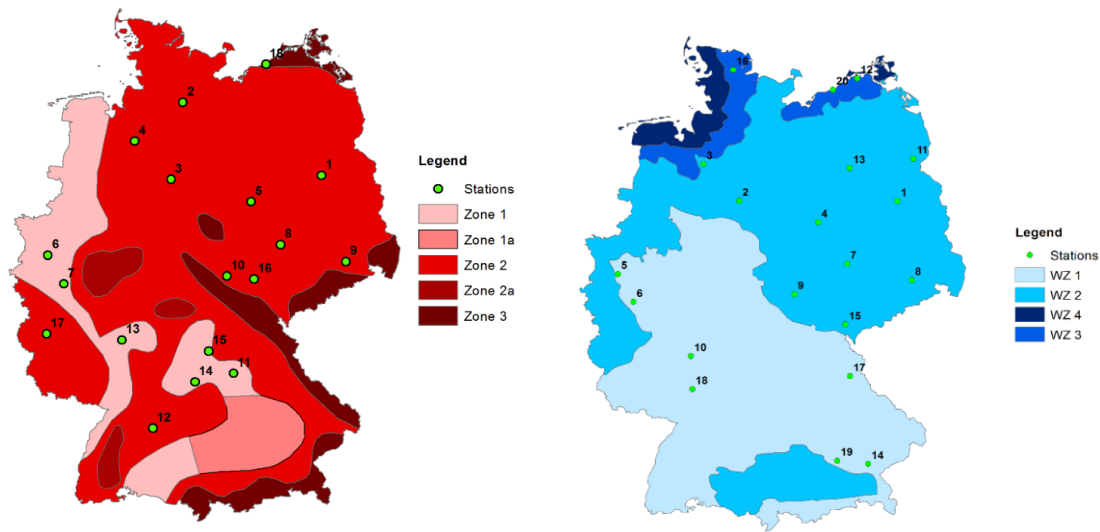
Finally, the calculations C11 and C12 show the results for the standard case (CoV wind =0,14 and uniform α_Q values) for the case of only one material (steel) and one variable load (wind). The result is that the partial factor for permanent load should be lowered and the factor for the variable loads should be increased compared to the overall average.

B.2.5 Effect of bias in characteristic value on partial factors

The example calculations performed in Chapter 3.8 of the main text as well as in Annex B2 are always based on the definitions of the characteristic values according to EN1990 clause 6.1.2.3. However, one should keep in mind that National Annexes to the relevant Eurocode parts may (and often do) provide slightly different characteristic values if compared to the results of extreme value analysis of measured data. This holds in particular for the characteristic values of climatic loads and it is mainly due to the needs of mapping procedures in the derivation of climatic maps, as well as to past practice. This section will show the possible effects on the partial factors.

As an example we will consider a bias in wind and snow loads. Starting from data collected and made available by the German Meteorological Office, (Formichi and Landi, 2022) calculated a preliminary estimation of the mean value of the bias for ground snow loads and wind pressure at approximately 20 weather stations in Germany, spread over the country as illustrated in Figure B.4, where the location of the selected weather station is plotted against the corresponding climatic zone, for both snow and wind actions as given in the German NAs to the pertinent Eurocode parts.

Figure B.4. Selected weather stations and snow zones (left) and wind zones (right) (with courtesy of the University of Pisa) (Formichi et al, 2023)



It turned out that the mean value and standard deviation of the bias (ratio between the characteristic value resulting from EV analysis and the corresponding NA's value) are given by:

-for wind pressure : mean(b) = 0.83 and st dev(b)=0.23

-for snow load : mean(b) = 0.72 and st dev(b)=0.12

where $b = Q_k/Q_k^*$ with Q_k the characteristic value according to EN1990 and Q_k^* the characteristic value provided by the regional code.

Note:: the data were extracted from the website of the German Meteorological Office: Deutscher Wetterdienst (DWD) Climate observations. 2022 (https://opendata.dwd.de/climate_environment/CDC/observations_germany/climate/). Comparisons with NAs data were made against: DIN EN 1991-1-3/NA. Eurocode 1 – Actions on Structures – Part 1–3: General Actions – Snow Loads –National Annex (in German). and DIN EN 1991-1-4/NA. Eurocode 1 – Actions on Structures – Part 1–4: General Actions – Wind Actions –National Annex (in German).

If the bias b has no scatter (just single values over the whole area) and a value lower than 1.0, it means that the design load used in the design equation (see chapter 3) using the partial factor derived according to the procedure in chapter 3 is higher than required. So, in that case we might actually multiply the partial factor by the bias b and end up with the desired result. Characteristic values and partial factors are communicating vessels: higher characteristic values for loads mean lower partial factors.

If the bias is random (meaning that we do not know the exact value in every location within a zone or neglect it for simplicity) we can split up the bias into a mean part and a fluctuating part. The mean part may be treated as before. The zero mean fluctuating part does not affect the dimensions but leads to a larger uncertainty. It has to be taken care of as an additional random factor in the LSF in the reliability calculation. If the effects of the fluctuating part is significant (i.e. high C.O.V.), this will reduce beta which has to be compensated by a larger partial factor.

In the present example case of climatic loads in Germany we find the following results for the optimized partial factors (see Formichi et al, 2023)

Table B.4 Calibrated partial factors for different actions and for different treatment of bias in climatic actions

	γ_G	γ_{wind}	γ_{snow}	$\gamma_{imposed}$
No bias, no scatter, as in Annex B2	1,2	1,9	2,35	1,6
With bias, no scatter	1,2	1,6	1,65	1,6
With bias and scatter	1,2	2,1	1,85	1,6

The values presented in the line “with bias, no scatter” have no direct meaning at regional scale, but make sense at local scale where it can be assumed that the characteristic value is known from available measurements. The values are also intended to illustrate the contribution of the various effects. The reason for deviating from the formal definition of the characteristic value in the NA could be that codes keep definitions and corresponding values from previous codes. In those cases one should be aware of the fact that the characteristic value and the partial factor are communicating vessels: high characteristic values may be compensated by lowered partial factors and vice versa. But it is essential to keep track of the difference between the average part and the fluctuating part.

From this example it follows a final recommendation to the National Standard Bodies in charge of the definition of National Annexes to the Eurocodes to be used in a country: the calibration of partial factors is highly dependent on the choices taken in the definition of the characteristic values of actions (mainly climatic actions) in the NA's to the relevant EN1991 parts. This aspect should be carefully dealt with in the calibration process since, as it is shown in the example, overlooking it could introduce overestimation of the actions' effects. So it is important to have the background of action models available in accessible documents.

B.2.6 Summary, conclusion and reflection

In this annex the derivation of partial factors for actions was demonstrated on the basis of both the DVM method and the Code Optimization Method (minimizing deviations from the target reliability) as described in Annex C.4.4.2 respectively C.4.4.3 of EN1990:2022. The calculations should primarily be considered as “examples” showing how the methods work for a given input. It should be realized that for the Code Optimization Method the calculations were based on simple limit state functions for bending of beams only with mean values and coefficients of variation estimated to be representative for a large class of member verifications. These limitations can be considered as acceptable for the demonstration purposes of this Annex, but need further development to reach final conclusions.

Member states that want to apply the methods for evaluating Eurocode recommendations or developing national determined parameters in their National Annexes are advised to review the various inputs for relevance, and how representative they are, for their applications and balance their consideration of the findings of reliability analysis with past performance and practice.

The **DVM** was already presented in the first editions of the Eurocode EN1990:2002 as well as in the preceding ENV1991 version. In the EN1990:2002 the method was described in the informative Annex C and served as the basis for Annex D (Design by Testing). As such it has been used widely to derive the partial factors for the resistance side of the design equations in the material related Eurocodes. In the Background Document to the EN1990:2002 [REF] the method was for reasons of consistency and demonstration also used to derive partial factors on the loading side. The numerical basis of the method for use in the Eurocode can be described as:

- a target reliability $\beta = 3,8$ for a reference period of 50 year.
- a sensitivity factor $\alpha_Y = 0,8$ for dominating resistance parameters and $0,4 \times 0,8 = 0,32$ for others
- a sensitivity factor $\alpha_Y = 0,7$ for dominating and $0,4 \times 0,7 = 0,28$ for accompanying actions
- probabilistic models for actions and structural properties (resistances).

The split α -values for load and resistance have the advantage that they allow action and resistance side to be treated independently. This allows also an optimisation for each material individually based on refined calculations concerning the different failure modes assuming adapted resistance function. Within the revisions for the second generation of Eurocodes a systematic review of the chosen partial factors and the belonging resistance functions has been done tuning those where deficiencies were recognised and allowing also for a differentiation of “more critical” failure modes in comparison to each other. Such a procedure is much more cumbersome when using the optimization method.

The DVM also has the advantage that no explicit assumptions need to be done on the choice of a portfolio and corresponding weight factors. One may of course claim that these assumptions implicitly are reflected in the above α -values. The α -present values have been determined and agreed long time ago and form the basis of the existing derivation of most partial resistance factors, see 3.1.3. Also, many proposals dealing with the loading side have been published. However, these papers do not always correspond to the current definition of the characteristic values in EN1990 and EN1991 and do not make use of up-to-date probabilistic models.

The **Code Optimization Method** [Kohler] is an alternative to the DVM and has been added in the EN1990:2022. The code optimisation in principle deals with all materials and all actions in one straight forward optimization procedure, although it also may be used to optimize a subset (say for one material, like the lines 11 and 12 in Table B.3). The explicit aim is to reach at a set of partial factors that on average reaches the target level and at the same time minimizes the sum of individually squared deviations. In EN1990:2022, C.4.4.3, the deviations in terms of the reliability index have been chosen. A discussion on other options like the deviation in terms of failure probabilities can be found in [Baravalle, 2017]. The idea to avoid structures with very large reliability (uneconomic) and too small reliability (unsafe) was already the leading thought behind the transfer from “load factor design” to “load and resistance factor design”. One may also find the idea reflected in the requirement $0,16 < \sigma_E/\sigma_R < 7.6$ in C.7(3) of EN1990:2002 intended to set limits to the use of the DVM split alpha values 0,7 and 0,8.

The summarised basis for the optimization method for the mainstream Eurocode is:

- a target reliability $\beta = 3,8$ for a reference period of 50 year.
- a chosen portfolio of applications and a corresponding set of limit state functions
- a corresponding set of weight factors
- probabilistic models for actions and structural properties.

The weight factors are intended to represent the portfolio of design equations for which the partial factors are supposed to be applied and over this domain to achieve minimum deviations from the target reliability. Of course, it is difficult to find these weight factors in an objective way and so some sensitivity study to the choice is relevant. In B.2.4 results are shown for uniform weights and some alternatives.

For a good understanding of the optimization method, it is necessary to realize that when *all* partial factors are kept as “free to optimize parameters”, one should be aware of the fact that the outcome of the Code Optimization is mathematically undetermined. An infinite large set of factors may be obtained, all leading however to the same minimum deviation with the same or similar structural dimensions.. An illustrative example is the case when we have only one load and one resistance term. The reliability requirement is:

$$R_k / \gamma_R > \gamma_F S_k$$

All partial factor choices leading to the same value of the product ($\gamma_R \gamma_F$) are fully equivalent from a mathematical point of view and may all be equally optimal. However, the values like $\gamma_R = 0,5$ and $\gamma_F = 3$ would look very strange to the designer and is not in line with the basic idea behind the partial factor method. So, if all partial factors are free at least one value should be selected to be fixed. This however does not affect the final result in an essential way.

In the current example study reported in B.2.1 this issue is not relevant since the partial factors on the resistance side have been fixed. The values chosen are the ones derived in the various material related Eurocodes, based on the DMV-method and so automatically taking on board the optimizations already carried on the structural level.

Another (related) issue is the fact that the objective function may be relatively flat around the optimum or shows multiple minima, allowing a set of slightly or more widely different solutions, also in terms of structural dimensions. These different solutions are equivalent from the mathematical optimization point of view, but of course may be preferred or not from other considerations (e.g. use of material consumption). It is important to be aware of this possible effect.

As can be seen also from the sensitivity study in Table B.3 completely different partial factors (corresponding to different structural dimensions) come to the same averaged beta values. One should bear in mind here that the optimization is not intended to change the average but to reduce the extreme deviations from the scatter. Information on those aspects can be found in the violin plots in B.2.1, an effect that is of course less pronounced if the “modified values” of 1.75 are assumed for all variable loads, see violin plots in Figure B.1.

Discussion

Having the results of the calculations in this annex in mind, the AHG would like to emphasize the following points:

- The present partial load factors 1.35 for permanent and 1.50 for variable loads in EN1990 may be considered as correct in the sense that they meet on average the target value (see chapter 3.4); also, the use in practice (although over a limited period of time compared to desired return periods) has not shown any obvious shortcomings. Overconservative and possibly uneconomic solutions of course cannot be observed.
- The optimization method in the example calculations in this Annex leads to partial load factors having almost the same average value for the reliability index as the partial factors recommended in EN1990, Annex A., but show less scatter especially in view of the influence of wind and snow actions. In view of the dominating influence of these climatic actions the conclusion is that optimisation of resistance functions for different failure modes of a single material leading to a reduction of the corresponding model uncertainties has nearly no influence at all. A better approach would be to try to reduce the scatter by improved (but also often complex) loading functions. We should have mind, however, that we cannot reduce the natural variability. Maybe a reduction in the load models (pressure coefficients etc) is possible by additional research, but this takes time. We can only profit from that once the results are on the table. Anyhow, it is recommendable to perform coordinated studies on this topic, possibly initiated on the level of TC250 and SC1.
- In this annex the characteristic values for snow and wind have been based on the definition given in chapter 6 of EN1990, that is the value with an annual 2% exceedance probability; if member states use values corresponding to a lower exceedance probability this should be taken into account, either by lowering the partial factor or (preferably) by adapting the characteristic value. One should also be aware of the fact that the example calculations in this annex are based on an average geographical location and that results could be different for other parts of Europe. Some indication of the influence of this effect can be found in Annex B2.5.
- For the calculated example in Annex B.2.1 the result of the DVM and the result of the Code Optimization method are roughly pointing in the same direction. The result is also similar to the result presented in the Background Document to the ENV and implicitly to EN1990:2002 {REF}. The result can be summarised as a drastic increase of the partial factor of the variable loads and some decrease of the partial factors for the permanent loads. It should be noted that as a consequence of the given results (1,20 for the self-weight and 1,94, 2,35 and 1,60 for respectively wind, snow and imposed load). For lighter materials as steel there would be an increase of the dimensions between 25% and 50% if used for building roofs or facades. Of course, there will be gains for other parts of the application field. But for several reasons one should be careful with such a jump in dimensions in a new code: (1) one may question the theoretical basis; (2) there seems to be no urgent need from an unsatisfactory track record and (3) one should also consider possibly far stretching economic consequences
- The statistical models used in this Annex correspond to the present stage of knowledge and is as far as possible in agreement with models mentioned in the Eurocodes or the relevant Backgrounds. The weight factors are to some extent intuitive and may need improvement, and should be reconsidered by national code committees to reflect country specific building traditions.
- Further/other refinements are possible by adopting the EN1990 design formats (8.13) or (8.14); also, partial factors on the resistance side could be fine-tuned (given the partial factors on the loading side) to achieve a more homogeneous reliability.
- Once the basis of a reliability optimization procedure has been accepted it could in future be extended to include explicitly additional issues like risks or be a basis to compare current design procedure with to new developments.
- One should be aware of the fact that some other Eurocodes the NDP values for the partial load factors have already been fixed on 1,35 and 1,50 to arrive at the value of some other parameters. Examples are the documents on fire safety and the value k_{mod} in the timber code. The same warning of course holds for all member states using (8.14) and (8.15) instead of (8.12).
- In the ideal situation, from a scientific point of view, all partial factors should be based on the same principles and methods. As present factors on the resistance side were found using DVM, the use of the load factors found in B.2.1.2 would be the most appropriate.

Finally, it is important to bring to the attention that in 2018 the Coordination Group of CEN TC250 decided that the recommended partial load factors in the EN1990 ENQ-version of the year 2020 should not be changed which has also been confirmed by TC250. The arguments used at that time were that:

refined partial factors have been derived for the new generation of Eurocodes on the resistance side assuming the current values on the loading side. The interdependencies would have needed an extensive new research to achieve the same level of accuracy and detailing without too much of conservatism or reduced safety.

the background of this new method of code optimisation was considered as relatively immature,

the ease of use of the practise was not believed to be served by a change,

a possible disturbance of the market balance,

the fact that the values were NDP's anyway.

Note also that at the time of the development of the ENV and subsequent EN-2002-version, the partial factors for variable loads following from the DVM were also considered as being too high to be codified.

It is now of course up to the member states to deal with the above arguments and look for optimal solutions.

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B.3 Column examples / Euler Column

B.3.1 Introduction

For nonlinear limit state functions the procedure is slightly more complicated than the one shown in sections 3.4 and B.2. However, we can still follow the general procedure as explained in example 2.4 in Chapter 2.6.3, summarized here for the sake of convenience:

1. Choose a portfolio of structures and design situations with corresponding limit state functions
2. Define the corresponding weights w_i
3. Set the target reliability
4. Define the set of statistical properties and characteristic values
5. Define a verification code format
6. Find iteratively the set of partial factors that leads to a minimum of D , being the (weighted) summed squares of the deviations from the target.

In order to find the β -value for each individual design situation we first have to design the structural dimensions using the characteristic values and the chosen set of partial factors; next a probabilistic analysis (FORM or Monte Carlo) is used to find the corresponding probability of failure or reliability index. If the minimum value of D is too large to whatever standard, there may be a need to split up the portfolio in a set of smaller ones.

In the beam example in section 3.4 it was possible to divide resistance and load by the total load effect $(1/8)(g_m + q_m) L^2$ leading to a generalized limit state function (3.7) and a generalized design equation (3.8). The advantage was that the length L was not important any more as a design parameter and only the ratio of the characteristic values over the mean was important. In the general non-linear case this is not possible. In a column analysis the CoV of the "reduced resistance" r will depend on the slenderness. As a consequence, we

have to consider several individual values of the slenderness as separate parts of the portfolio, and give them a weight in correspondence to market share or importance.

However, the next paragraphs demonstrate the analysis for just a single slenderness ratio, respectively for steel and concrete.

B.3.2 Steel column

In this section the reliability level for a steel column designed according to the Eurocode is analysed. For the analysis the following LSF and Design Equation have been chosen:

$$\text{Limit state function: } g(X) = \theta_R N_{cr} - \theta_E N = \theta_R N_{cr} - \theta_E [G + \theta_Q Q]$$

$$\text{Design equation: } N_{cr,k} / \gamma_R = \gamma_G G_k + \gamma_Q Q_k$$

The loading parameters have the same CoV's and partial factors as in the bending case (see chapter 3.4 of the main text) and are presented in Table B.5.

In EN 1993 the critical load is given by:

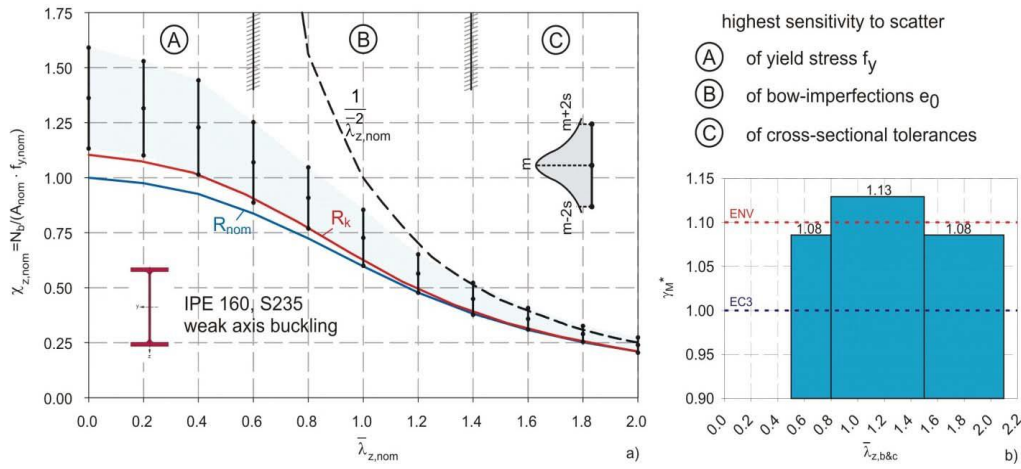
$N_{cr} = \chi A f_{y,nom}$ with χ a function of the slenderness $\bar{\lambda} = \sqrt{(A f_{y,nom} / F_E)}$ and F_E is the Euler load; the definition of the slenderness is not of importance for this analysis. In this model χ includes the randomness in the yield stress, the modulus of elasticity, the dimensions, the eccentricities and the model. Consider an IPE-160-column with $\bar{\lambda} = 1.0$. The curves $\chi(\bar{\lambda})$ for this column are shown in Figure B.5 (SAFEBRICITILE) for the design value (red line), the mean value and the mean plus/minus 2 standard deviations. For $\bar{\lambda}$ equal to 1,0 the design value of χ is 0,6 leading to a design resistance exactly equal to the design value of the total load $\gamma_G G_k + \gamma_Q Q_k$ (note that $\gamma_R = 1.0$). Note that the SAFEBRICITILE project was carried out on the resistance side only and contains many details not discussed here. Only the results of the probabilistic resistance model are used for combination with the loading models.

All data is presented in Table B.5. No values are presented for the CoV of the cross sectional area and the yield stress as their uncertainty is taken care of by the buckling coefficient χ . Mean and scatter for χ given in Table B.5 are corresponding to the diagram shown in Figure B.5. It follows that for slenderness 1,0 the mean resistance is 0,74 and the 4-sigma-range is about 0,32. This leads to the mean and CoV for χ given in Table B.5. In reality there will be some uncertainty in the value of $\bar{\lambda}$ which has been translated into a model uncertainty θ_R

Table B.5 Overview of variables and numerical input values; last column shows resulting FORM α -values for steel column

X	designation	unit	mean	CoV	Xnom Xk	γ_x	α
A_{nom}	Cross sectional area (IPE 160)	mm ²	2000		2000		
$f_{y,nom}$	Nominal yield stress (S235)	MPa	283		235	1,00	
$\bar{\lambda}$	Slenderness (dimensionless)	-	1	-	1,00		
χ	Buckling coefficient	-	0,74	0,12	0,60		0,63
G	Structural weight	kN	170	0,05	170	1,35	-0,20
Q	Variable load (wind 50a)	kN	31,5	0,14	34	1,50	-0,16
θ_R	Resistance model	-	1,00	0,05	-	1,00	0,38
θ_E	Load effect model	-	1,0	0,10	-		-0,52
θ_Q	Wind force model	-	0,97	0,26	-		-0,29

Figure B.5. Data and α results of reliability c-values for steel column IPE-160 and with $\bar{\lambda} = 1.0$.



Having all data we perform a FORM-analysis. The resulting reliability index is **4.12** (so larger than the target 3.8) and the α -vector is shown in Table B.5, last column.

The resulting reliability index is in line with the result for a steel beam with the same ratio of permanent and variable loads. For the above column the α_Q parameter is around 0,25. From Figure B.2, we find that for $\alpha_Q=0,3$ the reliability index for the beam is around 4.2.

B.3.3 Concrete column

In this section the reliability level for a concrete designed according to the Eurocode is analysed. For the analysis the following LSF and Design Equation have been chosen:

Limit state function: $g(X) = \theta_R N_{cr} - \theta_E N = \theta_R N_{cr} - \theta_E [G + \theta_Q Q]$

Design equation: $N_{cr,k} / \gamma_R = \gamma_G G_k + \gamma_Q Q_k$

The loading parameters have the same CoV's and partial factors as in the bending case.

The external bending moment (load effect) due to the total eccentricity $e = e_0 + u$ (where $e_0 + u$ is the initial eccentricity and u corresponds to the displacement) is given by:

$$M_E = N e \quad (B.2)$$

An upper bound for the eccentricity is the given by (ϵ_{max} being the ultimate strain):

$$e = e_0 + e_{2max} = e_0 + L_k^2 \epsilon_{max} / (\pi^2 h / 2) \quad (B.3)$$

For a square column (area $A = h^2$), the bending moment capacity is given by:

$$M_R = 0,8 \rho A f_y h + 0,5 N h [1 - (N/A f_c)] \quad (B.4)$$

where:

0,8 is to take care of the cover and the steel dimensions.

ρA is the area of steel on one side of the column only

The axial load bearing capacity N_{cr} can be found by putting $M_E = M_R$:

$$N e = 0,8 \rho A f_y h + 0,5 N h [1 - (N/A f_c)] \quad (B.5)$$

Which can be elaborated to:

$$0,5 N^2 h / A f_c + N e - 0,5 N h - 0,8 \rho A f_y h = 0 \quad (B.6)$$

Which is a standard quadratic equation.

The values for the input parameters are given in Table B.6.

Table B.6 Overview of variables, numerical input values and resulting FORM-values concrete column

X		unit	mean	CoV	Xnom Xk	γ_x	α
H	Column width (square)	m	0,4		0,4		
L_k	Buckling length	m	10		10		
ε_{max}	Maximum strain (compression)	-	0,0035		0,0035	1,2	
e_o/L	Imperfection (design value)	-	2/1000		2/1000		
ρ	Reinforcement ratio	-	0,02	-	0,02		
f_y	yield stress reinforcement FE500	MPa	540	0,05	500	1,15	-0,16
F_c	Concrete strength C45	MPa	60	0,17	45	1,50	-0,41
G	Structural weight	kN	1980	0,05	1750	1,35	0,21
Q	Variable load (wind 50a)	kN	396	0,14	350	1,50	0,18
θ_R	Resistance model		1,1	0,10	-		-0,56
θ_E	Load effect model		1,0	0,10	-		0,56
θ_Q	Wind force model		0,97	0,26	-		0,31

Note: The values for f_y and f_c have to be confirmed.

Design calculation 'leads to a design value for the resistance $N_{cd} = 2897$ kN and an axial design load $N_d = 2888$. So, the design is sufficient. The resulting reliability index is 4.16 (above the target 3,8) and the α -vector is shown in Table B.6.

For more information on the background of partial factors for concrete columns the reader is referred to Chateuneuf et al, 2001.

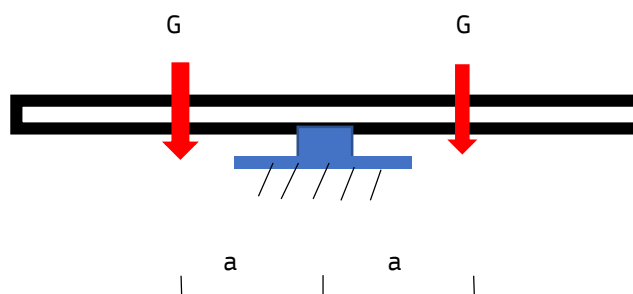
References

Chateuneuf A., Soars R. and Venturin W. S. (2001), 'Partial safety factors for homogeneous reliability of nonlinear reinforced concrete columns', *Structural Safety*, Vol. 23, Issue 2, pp. 137-156

B.4 Partial factors in the case of static equilibrium

A girder (see Figure B.6) is positioned on a block without structural connection, so it is a case of static equilibrium and factors according to case DC3 of EN 1990 Table A.1.8 should be used. In that case we use the load factors 1,35 and 1,15 for the two forces leading to moment $M = 0,2Ga$ and a vertical load of 2,5 G. The resulting eccentricity is 0,08 a. So, if the block in total is wider than 0,16a the structure fulfils the Eurocode requirement.

Figure B.6. Girder on a block, case for EQU example



According to the theory of reliability one would expect load factors in the order of 1,1 and 0,9. This leads to $M=0,2Ga$ and a vertical force equal to $2G$ and an eccentricity of $0,10a$. So a block of $0,16a$ would be considered as unsafe.

Let us consider the results of a reliability analysis based on the limit state function using the minimum dimensions following from the Eurocode factors:

$$g(X) = 0,08 a (\theta_{N1} G_1 + \theta_{N2} G_2) - a (\theta_{M1} G_1 - \theta_{M2} G_2) \quad (B.7)$$

with statistical properties (see section 3.5.3):

$$V(G) = 0,05, V(\theta) = 0,05, \rho(G_1 G_2) = 0,7 \text{ and } \rho(\theta_i \theta_j) = 0,5 \quad (B.8)$$

The values of $\mu(G)$ and a are not relevant and can be taken as unity. Note that the correlations for the chosen four model uncertainties (θ_{N1} , θ_{N2} , θ_{M1} and θ_{M2}) negative values are possible in this specific case. If the girder is positioned a bit out of the centre then G_1 is larger and G_2 is smaller; also the lever on one side is larger and smaller on the other. So maybe uncorrelated values for the model uncertainties are better. On the other hand, the correlation of G_1 and G_2 may be very high (closer to 0,9) as this is one continuous plate (in fact a single source case). An overview of results for different correlations may be found in Table B.7. The key values for $\rho(G_1 G_2) = 0,9$ and $0 < \rho(\theta_i \theta_j) < 0,6$ are highlighted in bold.

Table B.7 Resulting reliability indices for as a function of $\rho(\theta_i \theta_j)$

$\rho(\theta_i \theta_j)$	Block designed according to γ -values 1,35 and 1,15		Block designed according to γ -values 1,1 and 0,9	
	$\rho(G_1 G_2) = 0,7$	$\rho(G_1 G_2) = 0,9$	$\rho(G_1 G_2) = 0,7$	$\rho(G_1 G_2) = 0,9$
-0,4	1,7	1,8	2,2	2,3
-0,2	1,8	2,0	2,3	2,5
0,0	2,0	2,2	2,5	2,7
0,2	2,2	2,4	2,7	3,0
0,4	2,4	2,7	3,0	3,4
0,6	2,7	3,2	3,4	4,0
0,8	3,2	4,1	4,0	5,1

So γ -values “1,35 and 1,15” give rise to β -values in the range 2,2 to 3,20 and γ -values “1,1 and 0,9” to β is 2,7 to 4,0. So from this perspective the values 0,9 and 1,1 are to be preferred, as they are closer to the EN1990 target value 3.8.,

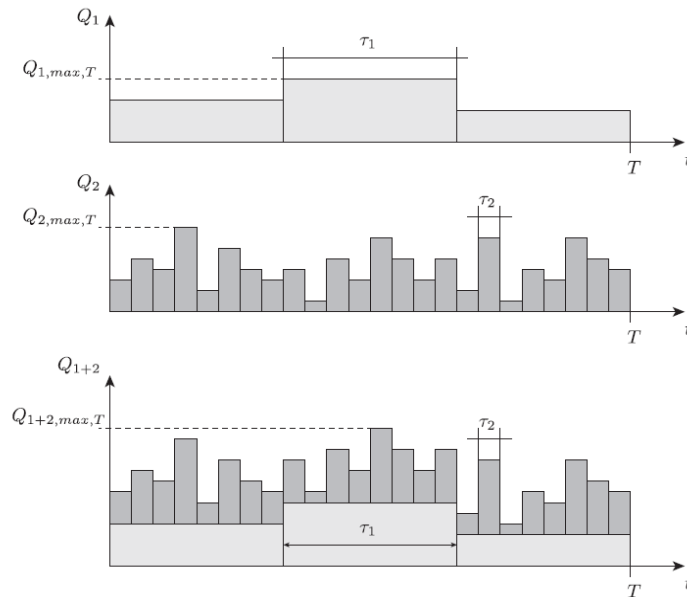
Note finally that the single source distinction is not used in the Eurocode for the limit state of static equilibrium.

B.5 Load combinations

B.5.1 Load combination probabilistic analysis

The combination of two independent variable loads Q_1 and Q_2 is considered. The extremes of variable loads per reference period T are represented by their corresponding extreme value distributions $Q_{1,max;T}$ and $Q_{2,max;T}$. The load processes are represented as sequences of rectangular pulses of uniform (and deterministic) length τ_i , such that the reference period T contains $n_i = T/\tau_i$ load pulses, see Figure B.7. Furthermore, are the loads represented such that $\tau_1 > \tau_2$ and that n_2/n_1 is always an integer.

Figure B.7. Loads Q_1 and Q_2 presented as FBC models



The value of T may be specified as 1 year so that $Q_{1,max;T}$ and $Q_{2,max;T}$ represent the annual extreme value distributions or as 50 year, being the design service life according to prEN 1990:2021

The extreme value distribution $Q_{2,max;T}$ can be scaled down to a smaller reference period, e.g. $\tau_1 = T/n_1$ by considering the realisations of Q_2 in time as independent pulses:

$$F_{Q_{2,max;\tau_1}}(q) = F_{Q_{2,max;T}}^{1/n_1}(q). \quad (B.9)$$

Similarly, the distribution of $Q_{1,max;T}$ can be scaled down to $Q_{1;\tau_1}$.

According to the combination procedure as proposed by (Ferry-Borges and Castanheta 1968) the maximum combination $Q_{1+2,max;T}$ can be approximated as:

$$Q_{1+2,max;T} = \max [Q_{1+2,max;\tau_1}] \quad (B.10)$$

$$Q_{1+2,max;T} = \max [Q_{1+2,max;\tau_1}] \quad (B.11)$$

$$Q_{1+2,max;\tau_1} = Q_{1;\tau_1} + Q_{2,max;\tau_1}$$

The distribution for $Q_{1+2,max,T}$ can easily be evaluated in a reliability analysis. In this note we will concentrate however on the treatment in semi-probabilistic design.

B.5.2 Load combination in semi-probabilistic design

B.5.2.1 Introduction

In semi-probabilistic design random variables are represented as Design Values, i.e. for $Q_{i,max,T}$ this means that the design value Q_{id} follows from:

$$F_{Q_{i,max,T}}(Q_{id}) = \Phi(-\alpha_{Q_{i,max,T}} \beta_t) \quad (B.12)$$

Where $\Phi(\cdot)$ is the cumulative standard normal distribution function, $\alpha_{Q_{i,max,T}}$ is the FORM sensitivity factor of the variable $Q_{i,max,T}$ and β_t is the annual or life time target reliability index. The sensitivity factors α may follow from a reliability analysis on a specific design case or taken as standardised values according to ISO2394. It should be noted that the standardised values may lead to usually unknown inaccuracies.

In order to explore the problem first the load combination for two variables constant in time will be treated. The formulas derived are also valid for the combination of tow loads, one constant in time and the other fluctuating.

B.5.2.2 Combination of two independent loads, at least one load constant in time.

For two time independent loads (or one time dependent and one time independent load) the combination rule is as follows:

Combination 1, Q_1 is dominant and Q_2 is accompanying;

Combination 2, Q_2 is dominant and Q_1 is accompanying.

The design values for the dominant and the accompanying or combination load in each combination can be found by solving (using the subscript "dom" and "com" to indicate the design values of the dominant and combination design value for Q respectively):

$$F_Q(Q_{dom}) = \Phi(-\alpha_S \beta_t) \quad (B.13)$$

$$F_Q(Q_{com}) = \Phi(-0,4 \alpha_S \beta_t) \quad (B.14)$$

where $F_Q(\cdot)$ is the probability distribution function for Q if the load is time independent or of the maximum value in the reference period if the load is time variant. According to prEN 1990 we take $\alpha_S = -0,7$ and $\beta_t = 3,8$ for a reference period $T=50$ year and consequence class CC2. The formulas hold for both Q_1 and Q_2 .

The combination factor Ψ_0 now follows from:

$$\Psi_0 = Q_{com} / Q_{dom} \quad (B.15)$$

Table B.8 shows the elaboration for the Ψ_0 values for the general case, the normal and the Gumbel distribution.

Table B.8 Formulas for Ψ_0 time independent case

General	$\frac{F^{-1}\{\Phi(0,28\beta)\}}{F^{-1}\{\Phi(0,70\beta)\}}$
Normal	$\frac{1+0,28\beta V}{1+0,70\beta V}$
Gumbel	$\frac{1+0,78 \{0,58+\ln(-\ln \Phi(0,28\beta))\} V}{1+0,78 \{0,58+\ln(-\ln \Phi(0,70\beta))\} V}$

The target reliability β_t for $T=50$ year is here simply denoted as β and V is the coefficient of variation for the maximum load in the reference period. In EN 1990 the Ψ_0 for the time independent (permanent) load is denoted as ξ .

B.5.2.3 Combination of two in time varying, statistically independent loads

In the case of two time varying loads the maxima of the loads do not need to occur at the same point in time. Let both loads be presented by FBC process as defined in Figure B.7. The relation between the probability distributions for the maxima $Q_{\max}$ of $Q(t)$ in τ and T is given by equation B.9.

The appropriate procedure in this case is to apply the combination rule of B.5.2.2 within the time interval τ where the maxima of the two loads actually will occur on the same point in time. This leads to:

$$\tau = \max(\tau_1, \tau_2) \quad (B.16)$$

For convenience we will use the notation P_T for the probability that the design value Q_d of a load Q is not exceeded in the reference period T :

$$P_T = \Phi(-\alpha_S \beta_t) \quad (B.17)$$

In the interval τ that probability (using $n = T/\tau$) is given by:

$$P_\tau = \{ \Phi(-\alpha_S \beta_t) \}^{1/n} \quad (B.18)$$

Now define:

$$\beta_\tau = \Phi^{-1}(P_\tau) \quad (B.19)$$

Which enable us to define the design values for the dominant and the accompanying loads in terms of the distributions for the period τ :

$$Q_{\text{dom}} = F_{Q_{\max \tau}}^{-1} \{P_{\text{dom}}\} \quad \text{with } P_{\text{dom}} = \Phi\{\beta_\tau\} \quad (B.20)$$

$$Q_{\text{com}} = F_{Q_{\max \tau}}^{-1} \{P_{\text{com}}\} \quad \text{with } P_{\text{com}} = \Phi\{0,4 \beta_\tau\} \quad (B.21)$$

Of course, we may equivalently express Q_d and Q_{com} in terms of the distribution of $Q_{\max T}$:

$$Q_{\text{dom}} = F_{Q_{\max T}}^{-1} \{P_{\text{dom}}^n\} \quad (B.22)$$

$$Q_{\text{com}} = F_{Q_{\max T}}^{-1} \{P_{\text{com}}^n\} \quad (B.23)$$

Elaborating Q_{dom} we arrive (of course) at:

$$Q_{\text{dom}} = F_Q^{-1} \{ \Phi(-\alpha_S \beta_t) \} \quad (B.24)$$

There is no difference with the time independent case treated in B.5.2.2

Elaborating Q_{com} we arrive at:

$$Q_{\text{com}} = F_{Q_{\max T}}^{-1} \{ \{ \Phi(0,4 \beta_\tau) \}^n \} \quad \text{with } \beta_\tau = \Phi^{-1} \{ \{ \Phi(-\alpha_S \beta_t) \}^{1/n} \} \quad (B.25)$$

As a simplification the following approximation is proposed:

$$Q_{\text{com}} = F_{Q_{\max T}}^{-1} \{ \{ \Phi(-0,4 \alpha_S \beta_t) \}^n \} \quad (B.26)$$

This formula is also presented in EN 1990, Annex C. This approximation is close to the classical Turkstra approximation (Turkstra and Mdsen, 1980) for load combinations. Section B.5.3 shows the accuracy of the approximaton for a specific example.

Table B.9 gives an overview of the formulas for this case, including further approximations for the normal and the Gumbel case; the factors 0,7 in front of $\ln N$ have been tuned to get results very close to the exact solution. Note that exact means: given the FBC modelling of the action and given the α -factors 0,7 and 0,28 according to EN 1990 and ISO 2394.

Table B.9 Formulas for Ψ_0 time dependent case

General Exact	$\frac{F^{-1}\{\Phi(0,4\beta_\tau)^n\}}{F^{-1}\{\Phi(0,7\beta)\}}$ $\beta_\tau = \Phi^{-1}[\Phi(0,7\beta)^{1/n}]$
General Approximation	$\frac{F^{-1}\{(\Phi(0,28\beta))^n\}}{F^{-1}\{\Phi(0,70\beta)\}}$
Normal calibrated to "exact" (N<100)	$\frac{1+(0,28\beta-0,7\ln n)\sqrt{v}}{1+0,70\beta\sqrt{v}}$
Gumbel calibrated to "exact"	$\frac{1-0,78\sqrt{v}\{0,58+\ln(-\ln\Phi(0,28\beta))+0,7\ln n\}}{1-0,78\sqrt{v}\{0,58+\ln(-\ln\Phi(0,70\beta))\}}$

B.5.2.4 Loads including a time dependent and a time independent part.

Consider the load combination given by equation B.27 on the interval (0, T):

$$S(t) = S_1 + a S_2 = C_1 Q_1(t) + a C_2 Q_2(t) \quad (B.27)$$

The coefficients C_i represent the time independent random part of the load model as for example load model uncertainties and pressure coefficients for generic building types. The factor a runs from 0 to infinity to represent all possible ratios of combinations of the two loads in the load effect.

The code representation of the design value S_d for combined load effect $S(t)$ is given by:

$$S_d = \max [S_{1d} + a \Psi_{02} S_{2d}; \Psi_{01} S_{1d} + a S_{2d}] \quad (B.28)$$

The design values S_{di} for the individual actions are defined for the reference period T and can be found from:

$$S_{id} = F_{Si}^{-1} \{ \Phi(-\alpha_S \beta_t) \} \quad (B.29a)$$

$$F_{Si}(s) = \int_{Q_{i \max T}} (s/c) f_{Ci}(c) dc \quad (B.29b)$$

The parameters α_S and β_t are defined as before. The task is to find values Ψ_{01} and Ψ_{02} in such a way that for all values of a (or at least on average):

$$P(\max S(t) > S_d) = \Phi(\alpha_S \beta_t) \quad (B.30)$$

One might use here a minimum distance principle comparable to the calibration processes for partial factors. The exact determination this way, however, is quite cumbersome and it may be convenient to have some approximations.

First of all, instead of the integral expression B.29 for the individual design values S_{di} we may use the standard DVM approximation for a function of two random quantities:

$$S_{id} = \max [C_{i \text{ com}} Q_{i \text{ dom}}, C_{i \text{ dom}} Q_{i \text{ com}}] \quad (B.31)$$

where (dropping the index i for clarity):

$$Q_{\text{dom}} = F_{Q_{\text{max}}T}^{-1} \{ \Phi (-\alpha_5 \beta_t) \} \quad (\text{B.32a})$$

$$Q_{\text{com}} = F_{Q_{\text{max}}T}^{-1} \{ \Phi (-0,4 \alpha_5 \beta_t) \} \quad (\text{B.32b})$$

$$C_{\text{dom}} = F_C^{-1} \{ \Phi (-\alpha_5 \beta_t) \} \quad (\text{B.32c})$$

$$C_{\text{com}} = F_C^{-1} \{ \Phi (-0,4 \alpha_5 \beta_t) \} \quad (\text{B.32d})$$

As by definition C is constant in time, the combination value $Q_{i \text{ com}}$ in (B.32d) is calculated using $N=1$.

A simple approximation for the combination value $S_{i \text{ com}} = \Psi_{0i} S_{id}$ may follow if we continue to neglect the time variability in the loads and use again $N=1$:

$$S_{i \text{ com}} = [C_{i \text{ com}} Q_{i \text{ com}}] \quad (\text{B.33})$$

So, in all formulas B.31-B.33 the number N is absent and the time variability of Q_i does not play a role, which makes the approach conservative.

An alternative formulation may be found when the variability (that is the CoV) in Q (for both loads) is dominant compared to the variability in C. In that case we have:

$$S_{id} = \max [C_{i \text{ com}} Q_{i \text{ dom}}, C_{i \text{ dom}} Q_{i \text{ com}}] = C_{i \text{ com}} Q_{i \text{ dom}} \quad (\text{B.34})$$

And the load combination (1) may be written as:

$$S(t) = C_{1 \text{ com}} Q_1(t) + a C_{2 \text{ com}} Q_2(t) \quad (\text{B.35})$$

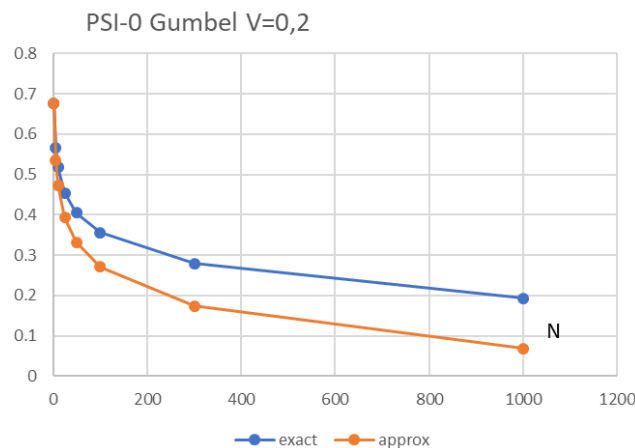
This way the variables C_1 and C_2 are reduced to deterministic quantities and $Q_1(t)$ and $Q_2(t)$ are two random time varying processes. This means that we can use the same Ψ_0 -values as in the standard case without the time independent variables C_1 and C_2 . If the condition is not fulfilled the exact approach B.27-B.30 or the conservative approximation B.31-B.33 has to be used.

Example calculations showing the accuracy (and so the usefulness) of approximations B.33 and B.35 by comparing with the exact approach may be found in section B5.3.2.

B.5.3 Numerical example

As an example consider a load Q having a Gumbel-distribution with mean value 1 and coefficient of variation $V=0,2$. Let the reference period T be 50 years and the target reliability 3.8 according to EN 1990:2021. Figure B.8 shows the value of Ψ_0 as a function of n. The values have been calculated using the exact formula B.25 and the approximation B.26.

Figure B.8. Calculated PSI-0 value for a Gumbel distributed with $V=0,2$



So, for $N=1$ we get the combination factor to be applied if the variable load Q is combined with a permanent load (as for instance in EN1990 (8.13a). Values of N between 50 and 300 seem to be reasonable for combination of for instance wind and snow. Very high values of N may apply to combinations with accidental loads. Note that N should never be chosen higher than the number of τ -intervals present in T for the load Q itself, irrespective of the other load.

So actually, a PSI-0 values should be defined for combination with every other load. The Eurocode neglects however this differentiation, except for the combination with accidental loads where PSI-1 and PSI-2 values can be used.

References

Ferry Borges J. and Castanheta M. (1968), *Structural Safety*, Laboratório Nacional de Engenharia Civil, course 101, Lisbon.

Turkstra C. J., and Madsen H. O. (1980), 'Load combinations in codified structural design', *ASCE Journal of Structural Engineering*, Vol. 106, Issue 12, <https://doi.org/10.1061/JSDEAG.0005599>

B.6 Nonlinear FEM (probabilistic and semi-probabilistic)

B.6.1 Shear wall example

Figure B.9. Shear wall and main data

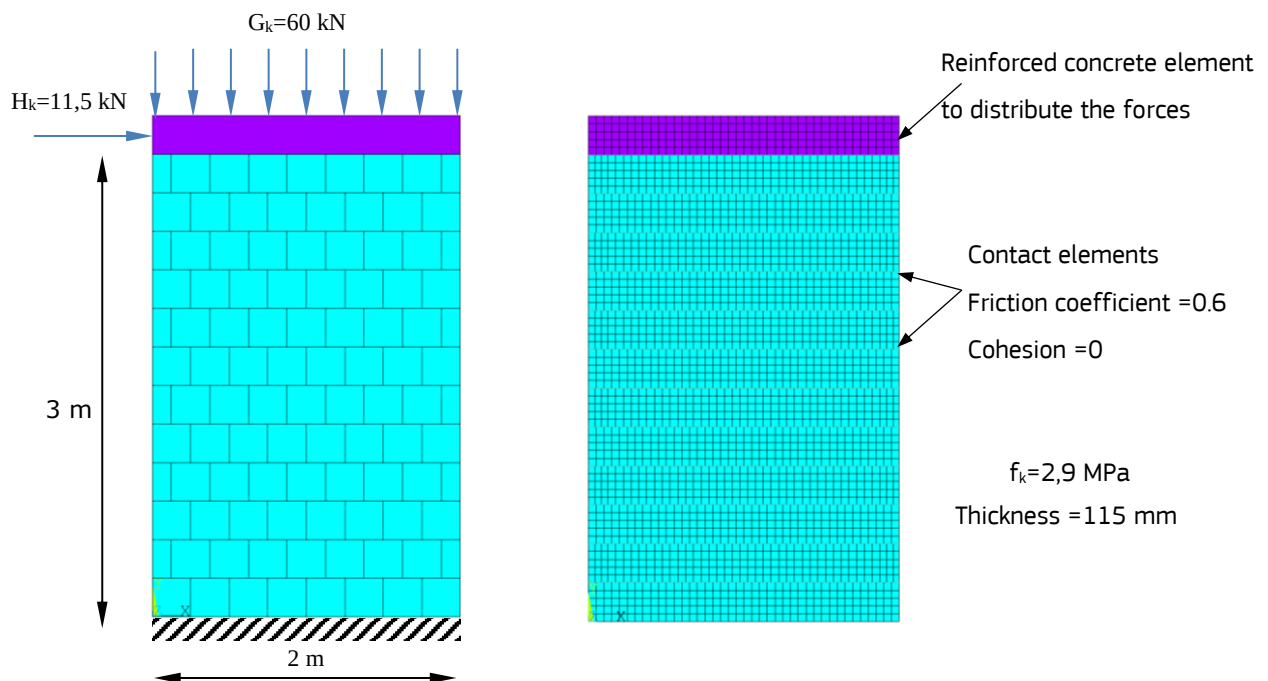


Figure B.9 shows a masonry shear wall loaded by a permanent vertical load and a variable horizontal load. The example illustrates how applying partial safety factors to FEM analysis can change the estimation of safety and the economy. The safety check has been calculated for the following cases:

- Case 1: the vertical action is applied as $\gamma_G G_k$, the horizontal action is applied as $\gamma_Q Q_k$, and the material parameters are applied as f_k/γ_M
- Case 2: the vertical action is applied as $\gamma_G G_k$, the horizontal action is applied as $\gamma_M \gamma_Q Q_k$, (compare 3.8.4.2 in the main report) and the material parameters are applied as f_k

The characteristic values of the input parameters are:

- The characteristic value of horizontal action (unfavorable) $H_k = 11,5 \text{ kN}$

- The characteristic value of vertical action (favorable) $G_k = 60 \text{ kN}$
- The characteristic compression strength $f_k = 2,9 \text{ MPa}$
- Partial factors $\gamma_M = \gamma_Q = 1,5; \gamma_G = 1,0$ (favorable)
- Corresponding other strength parameters (friction and shear strength)

In the FEM analysis the vertical load was applied first, and the horizontal load was raised in steps from 0 up to the failure load. The failure load has been recorded for each case from the finite element analysis. This result was compared to a hand calculation using a simplified rectangular vertical stress distribution at the right end corner and a separate checking of the normal and shear stresses at the support. A summary of the result is given in Table B.10.

Table B.10 Summary of the calculation results

		Case 1	Case 2
Application of partial safety factors for:			
Vertical action (favorable)		G_k	G_k
Horizontal action (unfavorable)		$\gamma_Q Q_k$	$\gamma_M \gamma_Q Q_k$
Material parameters		f_k/γ_M	f_k
Input parameters in FEM			
Vertical action (favorable)	[kN]	60,0	60,0
Horizontal action (unfavorable)	[kN]	17,25	25,88
Compressive strength masonry	[MPa]	1,93	2,90
Calculated Horizontal resistance using			
Hand calculation V_{Rd}	[kN]	17,30	18,20
FEM calculation V_{Rd}	[kN]	16,14	17,55
Unity Check			
Hand calculation		0,997	1,422
FEM calculation		1,07	1,474
Permissible characteristic horizontal action			
Hand calculation	[kN]	11,53	8,09
FEM calculation	[kN]	10,76	7,80
		Reference case	uneconomic

The unity check for case 1 in the finite element analysis was 1,07. In this case, the partial factors were applied in accordance with EN1990. In cases 2, where the unfactored characteristic values were used as input for the material parameters and for compensation the partial factor of the material was applied additionally to the horizontal action, the design was safe but very uneconomic.

Consequently, it is advised to use the design values of the action and design values of material as input parameters for the finite element calculation to avoid uneconomic results.

B.6.2 Example numerical analysis using characteristic or mean values for resistance

As explained in section 3.8 and demonstrated in B.6.1 it is to be preferred to use design values for resistance parameters in FEM (both options are allowed by EN1990). However, sometimes (e.g. in seismic engineering) it is a custom to use characteristic or even mean values. Arguments are that (in particular) mean values corresponds better to average test results and that having design values all over the structure is too pessimistic. However, the starting point is that the reliability level using FEM should not deviate from standard analysis, and so a compensation is required. This annex discusses a simple example in more detail.

In the most common situation we have the reliability requirement:

$$R_k / \gamma_R > \gamma_G G_k + \gamma_Q Q_k \quad (B.36)$$

If we neglect the partial factor on the resistance side we may compensate by adding a factor γ_R on the loading side:

$$R_k > \gamma_R \gamma_G G_k + \gamma_R \gamma_Q Q_k \quad (B.37)$$

This also holds when G acts as a favourable action (taking $\gamma_G = 1,0$):

$$R_k / \gamma_R > - G_k + \gamma_Q Q_k \quad (B.38)$$

Note that G_k itself here is a positive value. This leads to:

$$R_k > \gamma_R (- G_k + \gamma_Q Q_k) \quad (B.39)$$

So, in this case, we may even enlarge the load G_k even when it acts favourably. However, in a more complex LSF this may be unconservative. Consider the limit state where friction forces are involved:

$$R_k / \gamma_R + G_k (\mu_k / \gamma_R) > \gamma_Q Q_k \quad (B.40)$$

Where μ is a friction coefficient. Then we should have:

$$R_k + G_k \mu_k > \gamma_R \gamma_Q Q_k \quad (B.41)$$

A factor γ_R on G should be on the unsafe side in this case. To be always on the safe side one should not increase favourable acting loads in these cases.

Final note: even that correction may not be enough in all cases consider the slightly different failure criterion $G \tan(\varphi) > Q$ and let the partial factor be applied on the friction angle φ . Let the mean value of φ be given by 45° and $\gamma_M = 1,5$. In that case the design is okay if:

$$Q_d < G_d \tan(\varphi_d) = G_d \tan(\varphi_m / \gamma_M) = 0,58 G_d \quad (B.42)$$

Now use the mean value for φ and adjust the partial factor for Q. The factor for G is left unchanged equal to 1,0. In that case the design is considered okay if:

$$\gamma_M Q_d < G_d \tan(\varphi_m) \text{ or } Q_d < G_d \tan(45) / 1,5 = 0,67 G_d \quad (B.43)$$

Which makes the approximation unconservative.

B.7 (Semi) Probabilistic fatigue analysis using advanced models

B.7.1 Fatigue modelling and limit states

In this annex three different approaches to the formulation of the fatigue limit state verification for welded connections will be considered, respectively based on:

1. S-N curves in combination with Miner's Damage accumulation rule;
the fatigue crack growth model;
the fatigue crack growth model in combination with fracture resistance.

B.7.1.1 S-N curves

The S-N curve approach for fatigue is based on the standard experimental curves for constant amplitude loading, relating the number of stress cycles to failure N to the stress the range $\Delta\sigma$. For practical applications in steel structures subject to variable amplitude (VA) loading, a bi-linear log-log S-N-model is used having parameters K_1 , K_2 , m_1 and m_2 . The two linear parts are considered fully correlated, with an intersection at either $N_D = 5 \cdot 10^6$ or $N_D = 10^7$ cycles, depending on the type of detail (see prEN 1993-1-9):

$$\log(N) = \max [\log(K_1) - m_1 \log(\Delta\sigma), \log(K_2) - m_2 \log(\Delta\sigma)] \quad (B.44)$$

where

$$\log(K_2) = \frac{1}{m_1} \log(K_1) + \left(\frac{1}{m_2} - \frac{1}{m_1} \right) \log(N_D) \quad (B.45)$$

and "log" is the 10-th base logarithm. In some models a third horizontal line is added (cut off value) which may simplify calculations but the physical meaning is questionable in variable amplitude loading. The value of m_1 is usually taken as a fixed prior at a certain reasonable value (e.g. $m_1 = 3$ or 5) where the other values (K_1 and N_D) are determined to get the best agreement with test results. The value of m_2 is again a prior with, usually, a value of $m_2 = 2m_1 - 1$.

To account for variable amplitude (VA) loading, the Palmgren-Miner damage accumulation rule (hereafter called "Miner sum") is often used which leads to the following limit state function for a given period of time t (design service life):

$$g(t) = D_{cr} - D_n \quad (B.46)$$

where D_{cr} is the Miner sum at failure and D_n is the damage based on (3.1) due to n applied load cycles present in the period t .

$$D_n = \sum_{i=1}^n \frac{1}{N_i} \quad (B.47)$$

To evaluate D_n we need the stress ranges $\Delta\sigma$ corresponding to n , usually presented as a stress range histogram or spectrum and a total number of cycles n for the design life time (or other period) under consideration. The histogram or spectrum may be derived on the basis of load simulations in combination with rain flow counting or reservoir counting methods. In some cases the load standard presents an equivalent constant amplitude value.

Due to the fact that according to the damage accumulation of Palmgren-Miner rule (not in reality!) the order of the loads is not relevant, we may evaluate B.2.4 by an ordering in magnitude. This means that we may divide the spectrum into a number of stress range intervals, calculate the damage D_i for each stress range interval and multiply with the number of cycles in the interval; D_n follows by adding the contributions of all stress range intervals.

Additionally the model contains global and local model factors for the determination of the load effect $\Delta\sigma$:

$$\Delta\sigma = \theta_{lm} \theta_{glob} \Delta\sigma_{model} \quad (B.48)$$

$$\Delta\sigma = \theta_{lm} \theta_{glob} [(K_{scf} - 1) \theta_{scf} + 1] \Delta\sigma_{model} \quad (B.49)$$

if based on the nominal stress concept respectively on the hot-spot or effective notch stress concept, where $\Delta\sigma_{model}$ is the nominal stress, K_{scf} is the stress concentration factor, θ_{lm} is the uncertainty factor with respect to the representation of loads through a model, θ_{glob} is the uncertainty factor with respect to the (approximations and simplifications) of the structural model, and θ_{scf} is the uncertainty factor with respect to the stress concentration.

Fatigue may be caused by more than one fluctuating action. In that case the distribution (spectrum) should be based on the combined history. When the mean stress level needs to be considered in the damage rule, the mean values may be based on the on EN 1990 (8.17):

$$\Sigma F_d = \Sigma G_{k,i} + \Sigma \psi_{2,j} Q_{k,j} + (P_k) + \gamma_{FF} Q_{fat} \quad (B.50)$$

With $j > 1$ and Q_{fat} is the characteristic is the relevant fatigue action, and $Q_{k,j}$ refer to the variable actions excluding the action type(s) to which Q_{fat} belongs. Depending on the sensitivity of the model with respect to the mean value of the stress cycle it may be necessary to consider the mean value of fluctuation load per cycle and not the life time quasi static value. In equation B.50 it is implicitly assumed that uncertainties and sensitivities with respect to the first three terms of the right-hand side of the equation are sufficiently low so that no partial factors are required.

B.7.1.2 Crack Growth model, (partly) crack through criterion

For welded joints initially present micro-cracks at the surface tend to coalesce to form a single, dominant fatigue crack of roughly semi-elliptical shape. According to the pioneer work of Paris & Erdogan (1963) the crack growth description is given by:

$$\frac{da}{dN} = A(\Delta K)^p \quad (B.51)$$

Where:

a is the crack depth;

da/dN is the instantaneous crack propagation rate (or crack extension per cycle number);

ΔK is the stress intensity factor range at the crack tip;

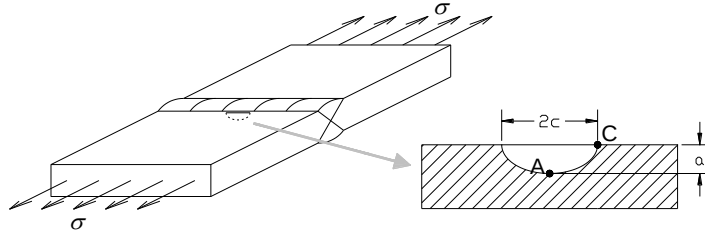
A and p are material constants.

The stress intensity factor K for a given applied stress σ is given as:

$$K = \sigma Y_a \sqrt{\pi a} \quad (B.52)$$

The stress σ is the applied, nominal stress in the vicinity of the crack location, see Figure B.10. The factor Y_a depends on the crack size normalised with respect to other relevant local dimensions (e.g. plate thickness), the geometry of the weld (if any) and the nature of stress distribution (membrane or out-of-plane bending). As in case of the S-N model, rainflow or reservoir counting methods can be applied to determine the ranges ΔK from the history of K . More complicated than for the S-N model, the crack size a increases in time equation B.51 and, through equation B.52, this means that K and ΔK are determined by combining equations B.51 and B.52. In most practical cases, it is sufficient to apply the counting procedure to the stress history and the ΔK histogram is subsequently determined by multiplication with $Y_a \sqrt{\pi a}$.

Figure B.10. A semi-elliptical crack in a steel plate at/near the weld toe



For practical cases, Equation B.51 is often too simplistic and a more accurate model that accounts for the near threshold condition is used, such as:

$$\frac{da}{dN} = \begin{cases} 0 & \text{if } \Delta K \leq \Delta K_{th} \\ \min(A_1 [\Delta K]^{p_1}, A_2 [\Delta K]^{p_2}) & \text{if } \Delta K > \Delta K_{th} \end{cases} \quad (B.53)$$

where A_1 , A_2 , p_1 and p_2 are material constants and the threshold of the stress intensity factor, ΔK_{th} , can be considered as a material constant (BS 7910:2013 + A1:2015) or as a material-specific function of the crack size (McEvily et al, 2003) (Chapetti, 2003) (El Haddad et al, 1979) (Zerbst et al, 2015) (Leonetti et al, 2021).

The limit state function can be formulated as

$$g(t) = a_{cr} - a(t) \quad (B.54)$$

where a_{cr} is a limiting crack depth (for example 75% of the plate thickness) and $a(t)$ is the crack depth after a service exposure of time t . At $t = 0$ the crack size has an initial value a_0 .

The calculation of the crack size at time t is not trivial as the stress is a random process. Assuming for $\Delta\sigma$ a sufficiently mixing (ergodic) process we may use the expectation of the stress range to find the expectation of da/dN :

$$E\left(\frac{da}{dN}\right) = A_1 E[\Delta\sigma^{p_1}]_{\Delta\sigma_0}^{\Delta\sigma_{tr}} (\theta_{glob} \theta_{sif} Y_a \sqrt{na})^{p_1} + A_2 E[\Delta\sigma^{p_2}]_{\Delta\sigma_{tr}}^{\Delta\sigma_{max}} (\theta_{glob} \theta_{sif} Y_a \sqrt{na})^{p_2} \quad (B.55a)$$

$$E\left(\frac{da}{dN}\right) = A_1 E[\Delta\sigma^{p_1}]_{\Delta\sigma_0}^{\Delta\sigma_{tr}} (\theta_{glob} \theta_{sif} [(K_{scf}-1)\theta_{scf}+1] Y_a \sqrt{na})^{p_1} + \dots \quad (B.57b)$$

for a nominal and a hot-spot stress based evaluation, respectively.

Flaws or fatigue cracks in civil engineering structures often manifest as surface cracks, which are usually approximated as semi-elliptical (Figure B.11). In such a case two values of K and ΔK are considered, namely, one for the deepest point of the crack (A), for which the previous equations apply, and one for the surface point of the crack (C), for which similar equations apply where Y_a is replaced by Y_c . A similar differential equation as equation B.51 applies for the crack growth at the surface point. As both Y_a and Y_c are functions on the crack depth, a , and the semi crack with, c , the two differential equations are coupled and should be solved simultaneously.

B.7.1.3 Crack Growth model, fracture based failure

Failure occurs when:

- a compression stress value arrives that causes buckling of the section weakened by the fatigue crack;
- a tensile stress value arrives that causes instable crack growth, caused by an interaction of (brittle) fracture and plastic failure of the remaining cross section to be exceeded. The plastic and the brittle fracture failure modes are a function of the crack size (a and c).

The compression failure mode can be evaluated with the finite element method. For tension, a failure assessment diagram can be used, which gives L_r – a measure of the proximity to plastic collapse – on the abscissa and K_r – the fracture ratio – on the ordinate.

$$K_r = \frac{K + K^s}{K_{mat,T}} + \rho \quad (B.56)$$

$$L_r = \frac{\sigma_{net}}{\sigma_{yT}} \quad (B.57)$$

Where $K_{mat,T}$ is the fracture toughness at temperature T , σ_{yT} is the yield stress at temperature T , σ_{net} is stress in the remaining cross section and ρ is an interaction term which depends on L_r , K and K^s , the latter factor depending on the residual stresses. The residual stress profile depends on the joint geometry and restraint conditions. Reference is made to (BS 7910:2013 + A1:2015, 2012) for approximate equations of all these factors.

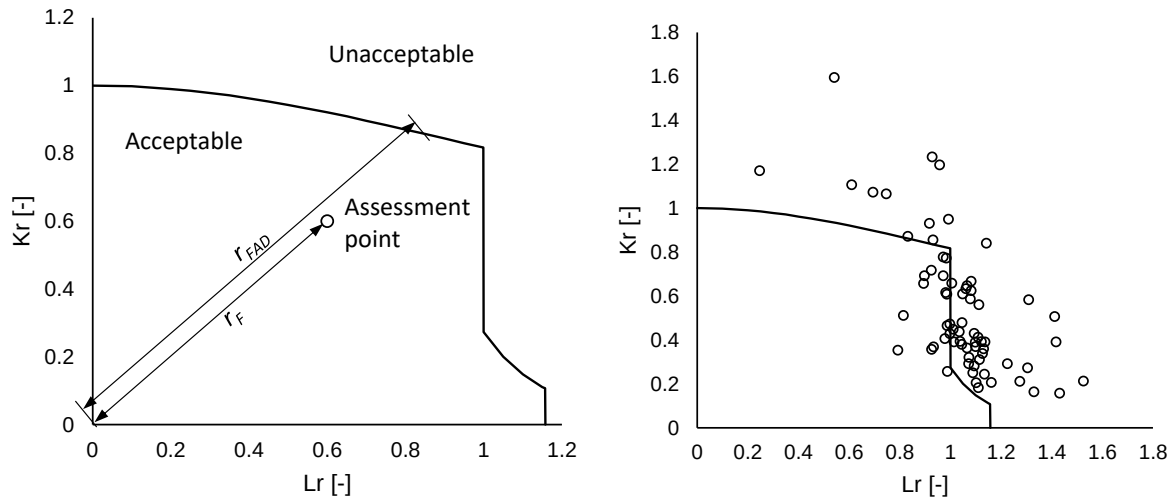
A flaw or crack is acceptable if the assessment point (K_r , L_r) is within the bounds of the (stress-strain relation dependent) curve in the diagram, for which reference is made to (BS 7910:2013 + A1:2015, 2012).

The limit state function of the tension failure mode can be formulated as:

$$g(t) = r_{FAD} - r_F + \theta_{FAD} - 1 \quad (B.58)$$

where r_F is the polar coordinate of the assessment point, i.e. $r_F = \sqrt{K_r^2 + L_r^2}$, r_{FAD} is the polar coordinate that gives the point on the curve in the diagram for the same polar angle (see Figure B.11a), and θ_{FAD} is a model factor evaluated through experiments. Several options are available to evaluate $K_{mat,T}$ and θ_{FAD} is depending on it. See Figure B.11b for the case that $K_{mat,T}$ is evaluated from three KJC tests, from which the average is considered and this average is subsequently corrected for the crack tip constraint through the triaxial stress, using procedures in (BS 7910:2013 + A1:2015, 2012).

Figure B.11. Failure assessment diagram: (a) with definitions of r_F and r_{FAD} ; (b) with data from large-scale wide plate and tubular joint tests (Maljaars et al, 2019)



To evaluate the failure probability on the basis of (B.58) a time variant reliability method like the out-crossing approach is needed, e.g. (Maljaars et al, 2012).

B.7.2 Verification

Verification can be performed using a probabilistic and a semi-probabilistic method. In the case of semi-probabilistic analysis one should derive partial factors on the basis of calibration to probabilistic methods. So, in both cases, one needs to state a reliability requirement.

For structures in reliability class 2, the European standard EN 1990 recommends a target reliability index for the entire fatigue life (actually the design lifetime of 50 year) ranging from 1,5 to 3,8, depending on the degree of inspectability, repairability and damage tolerance.

One should be careful here: the correct way to calculate the failure probability for the lifetime is by explicitly including the effect of inspection. This will lead to an increase of the reliability index and the target should still be 3,8. The reduction of the target to 1,5 is intended to compensate for the case where inspection is carried in reality but included in the calculation. It is in fact a strong simplification. One should never combine the two approaches: that is incorporating the positive effect of inspection on the calculated reliability level and taking the reduced 1,5 value as a reliability requirement. Actually, there is even a need for a better specification of the requirements and corresponding repair for this reduction.

B.7.2.1 Probabilistic method using the S-N model

The failure probability for a total period of time from 0 to t is given by:

$$P_f(t) = P(g(t) < 0) \quad (B.59)$$

Where $g(t)$ is given by B.46. The stress distribution or spectrum in the expression for $g(t)$ follows from the application and the structural properties. Data for the random variables for welded steel connections may be taken from a table like Table B.11.

Table B.11 Input for fatigue reliability analysis for welded steel connections using S-N curves

Variable		Units	Distribution	Mean	Standard deviation
$\text{Log}(K_1)$	Parameter	N and mm	normal		a)
m_1	Slope value S-N curve	-	-	b)	-
m_2	Slope value S-N curve			$2m_1 - 1$	-
D_{cr}	Miner's sum at failure	-	Lognormal	1,0	0,30 ^{e)}
θ_{lm}	MU load model ^{c)}		Lognormal		d)
θ_{glob}	MU global stress model ^{c)}		Lognormal	1,0	0,10
θ_{scf}	MU stress concentration ^{c)}	-	Lognormal	1,0	0,20
a) Detail dependent. In absence of test data, it is often assumed as 0,2 b) Detail dependent, and usually selected as $m_1 = 3$ or $m_1 = 5$ for most steel welded details. See prEN 1993-1-9, prEN 1993-1-11 and prEN 1999-2 for m_1 of the detail considered. c) MU = model uncertainty (correlated for all cycles). d) Load model dependent. Can be obtained by comparing with measurements. e) Original source: Wirsching (1984)					

In case of a probabilistic calculation there is a choice to evaluate the failure probability for the total design service life or of the last year of the design service life. For the annual evaluation holds:

$$P(\text{failure in year } t) = P(g(t) < 0) - P(g(t-1) < 0) \quad (B.60)$$

(or, alternatively, the conditional failure rate, see chapter 2). This probability should be less than the annual target, which according to Table C.2 in EN 1990 for reliability class 2 corresponds to $\beta = 4,7$. This annual value, however, is an upper bound and a lower value may be more appropriate because of the correlations in resistance and model uncertainties in time.

It should be noted that the probabilistic resistance model proposed here is a simplified representation of reality.

B.7.2.2 Semi-probabilistic verification using the S-N model

According to EN 1990 formula (8.11) the design value of the spectral stress range level $\Delta\sigma_{i,d}$ should be derived from:

$$\Delta\sigma_{i,d} = \gamma_{F,f} \Delta\sigma_{i,m} \quad (B.61)$$

where $\Delta\sigma_{i,m}$ is the (nominal) value of the i -th stress range of the stress range spectrum following from the action and structural model used and $\gamma_{F,f}$ is a partial factor for fatigue actions. In EN 1990 a value $\gamma_{F,f} = 1,0$ is recommended, which would mean that safety margin should be present in the action models, such as the λ -values in the equivalent load models. At present the λ -values need more elaboration.

The relevant fatigue load models may be found in EN 1991 for e.g. wind, traffic and waves. Some of these models contain simplifications which probably have been calibrated for a particular application but have no general validity. It is important to keep this in mind as some constructional details are more sensitive to high stress ranges than others.

On the resistance side the (characteristic) S-N curves in the Eurocodes are based on the 95% survival probability. The design values may be found by dividing by the partial factor γ_{Mf} . According to EN 1990 (8.22):

$$\Delta\sigma_{C,d} = \Delta\sigma_C / \gamma_M \quad (B.62)$$

where $\Delta\sigma_C$ is the characteristic fatigue resistance corresponding to N_C cycles ($N_C = 2 \cdot 10^6$). Assuming that $\Delta\sigma_C$ is based on a large number of tests, the mean value of the 10-base logarithm of variable K_1 in Table B.11, $\mu(\log(K_1))$ can be obtained as:

$$\mu(\log(K_1)) = \log(N_C) + 1,645 \cdot s(\log(K_1)) + m_1 \log(\Delta\sigma_C) \quad (B.63)$$

where $s(\log(K_1))$ is the standard deviation of the 10-base logarithm of variable K_1 . Since the two branches of the S-N curve are considered to be fully correlated, $\log(K_2)$ can subsequently be obtained from equation B.45.

The partial factor for fatigue resistance should account for the consequence of fatigue failure and the ease of inspection and repair of fatigue-sensitive members. The partial factors should be calibrated to the same reliability levels as for the probabilistic approach. As mentioned, the probabilistic resistance model of the previous section is a simplified representation of reality. Adopting such a model generally yields heavier designs or shorter fatigue lives than the semi-probabilistic verification with the partial factors given in EN 1990 and EN 1993. This is because these partial factors are based on a more advanced probabilistic resistance model (see (Maljaars et al., 2021), the connected presentations at WG9 (Maljaars, 2019), (Maljaars, 2020) and related publications and (Leonetti et al, 2020). It may be recommended to perform the probabilistic analysis also on more advance models.

It is noted that the partial factors are not calibrated for wind loads (with fatigue load in prEN1991-1-4 Annex F.8), and for other

EN 1991 also gives models to be used to check infinite life. If adopted, there is a need to carefully select the reliability target and corresponding partial factors.

B.7.2.3 Probabilistic verification using crack growth models

The failure probability for a total period of time from 0 to t is given by:

$$P_f(t) = P(g(t) < 0) \quad (B.64)$$

Where $g(t)$ is given by B.54 or B.58. The stress distribution or spectrum in the expression for $g(t)$ follows from the application and the structural properties. Data for the random variables for welded steel connections may be taken from a table like Table B.12.

Table B.12 Input for fatigue reliability analysis for welded steel connections using fracture mechanics

Variable		Units	Distribution	Mean	St. dev.
A_1	Crack growth constant (in air)	N and mm	Lognormal	$4,8e^{-18}$	$8,16e^{-18}$
A_2	Crack growth constant (in air)	N and mm	Lognormal	$5,86e^{-13}$	$3,52e^{-13}$
m_1	Crack growth exponent (air)	-	-	5,10	-
m_2	Crack growth exponent (air)			2,88	-
ΔK_{th}	Threshold value (in air) ^{a)}	N and mm	Lognormal	140	56
a_i	Initial crack depth (as-welded)	mm	Lognormal	0,15	0,10
a_i / c_i	Initial aspect ratio (as-welded)	mm	Lognormal	0,62	0,25
θ_{FAD}	MU failure assessment diagram ^{c)}		Lognormal	1,09	0,14 ^{b)}
θ_{glob}	MU global stress model ^{c)}		Lognormal	1,0	0,10
θ_{scf}	MU stress concentration ^{c)}	-	Lognormal	1,0	0,20
θ_{sif}	MU stress intensity factor ^{c)}	-	Lognormal	1,0	^{d)}
a) In its most simplistic form, i.e. no consideration of the geometry and crack size b) Values applies if $K_{mat,T}$ is evaluated from three KJC tests, from which the average is considered and this average is subsequently corrected for the crack tip constraint through the triaxial stress, using procedures in (BS 7910:2013 + A1:2015, 2012) c) MU = model uncertainty (correlated for all cycles). d) Depending on the source. Reasonable values: 0,03 in case of very often applied (i.e. lots of experience) parametric equations or 0,07 in case of evaluation through the finite element model					

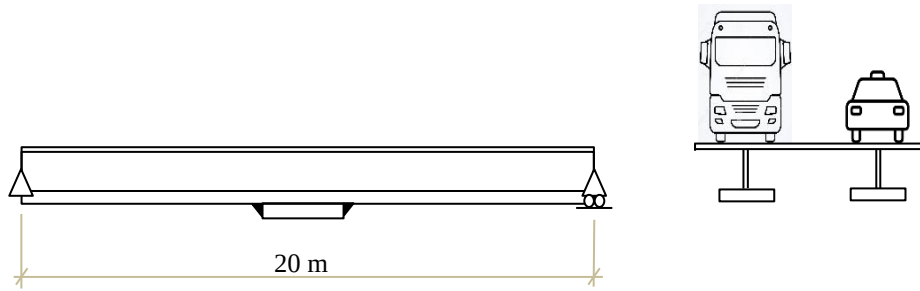
The values for the initial crack sizes are valid for an initial imperfection at a weld to in as-welded state.

The approach using an annual reliability target is considered to be more suitable for the fracture mechanics approach, as this approach is usually adopted to determine the remaining life or the inspection interval.

B.7.3 Worked example

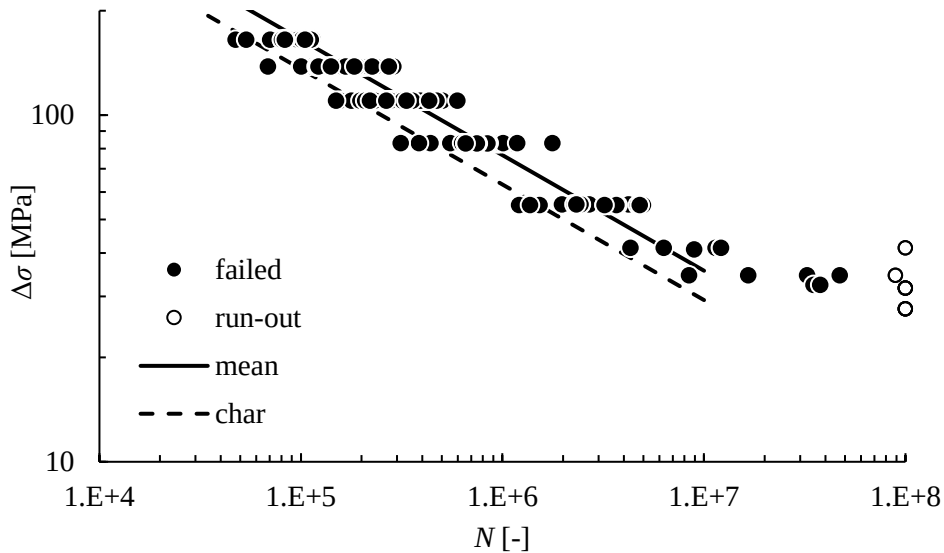
Let us consider a motorway bridge with a single span of 20 m. A cover plate is welded to the bottom flange of the main girder at midspan (Figure B.12). The main girder is loaded by one (slow) lane only. The bridge is designed with the S-N curve of prEN 1993-1-9 and Fatigue Load Model 4 (FLM4) of prEN 1991-2. The S-N curve has a characteristic reference fatigue resistance of $\Delta\sigma_c = 50$ MPa and a knee-point at $N_D = 10^7$ cycles. A 'safe life' design period of 100 years is adopted, and the corresponding values of the partial factors are $\gamma_{Ff} = 1.0$ or 1.1 (depending on the NA) and $\gamma_{Fm} = 1.35$ (large consequences of failure) according to prEN 1990 and prEN 1993-1-9, respectively. Using the damage accumulation rule of Palmgren-Minder, the required elastic section modulus is determined for this bridge such, that the design value of the damage is equal to $D_d = 1.0$.

Figure B.12. Structure considered in the fatigue numerical example



The probabilistic S-N curve is evaluated from constant amplitude tests described in the literature regarding beams with cover plates and with the dimensions corresponding with $\Delta\sigma_c = 50$ MPa as in prEN 1993-1-9, see Figure B.13. Using the value $m_1 = 3$ as a prior, the mean value of $\log(K_1) = 11.65$ and the standard deviation of $\log(K_1) = 0.15$. This gives a characteristic fatigue reference strength at 2 million cycles of $\Delta\sigma_c = 50$ MPa, i.e. exactly the value that prEN 1993-1-9 prescribes. Full correlation is assumed between K_1 and K_2 . The knee-point of the curve is considered as a fixed prior in agreement with prEN 1993-1-9 at $N_d = 10^7$ cycles. The second slope is also considered as a prior as $m_2 = 2m_1 - 1 = 5$. The critical value of the damage accumulation of Palmgren-Miner is lognormal distributed with a mean of 1 and a standard deviation of 0.3.

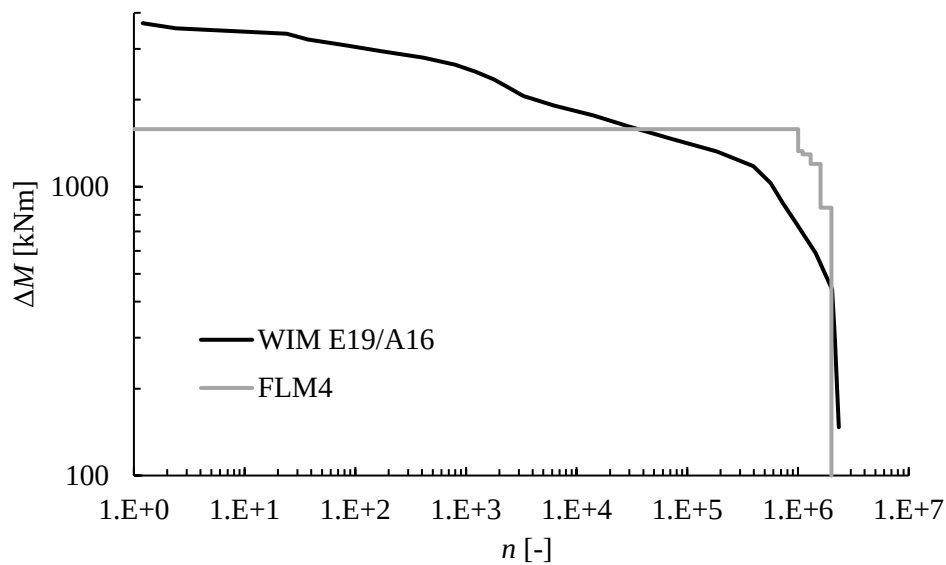
Figure B.13. Typical constant amplitude fatigue test data of cover plate beams



A Weigh-In-Motion (WIM) database recorded during 10 months in 2018 on motorway E19/A16 near Moerdijk (the Netherlands) is considered as the basis of the actual traffic load. The vehicles with their recorded axle loads and distanced between axles are run over the influence line and the load effect is determined. The bending moment ranges ΔM are subsequently determined using the rainflow counting method.

Figure B.14 gives the resulting histogram, adjusted for a full year (i.e. multiplying the recorded number of cycles with 12/10). The figure also gives the histogram of the 5 vehicles that comprise FLM4 and that is used to design the bridge.

Figure B.14. Typical bending moment range histograms of the fatigue example.



Variable θ_{scf} does not apply since the nominal stress method is adopted. The variable θ_{glob} (mean=1, st.dev=0.1) is applied together with variable θ_m (mean=1.1, st.dev.=0.05). The first variable accounts for approximations and simplifications in the derivation of the influence line. The second variable takes into consideration an expected increase in the total traffic load in the coming 100 years, as well as its uncertainty. It should be noted that this is an estimate (experts' opinions). Dynamic amplification due to bridge-vehicle interaction is usually small compared to the above-mentioned uncertainty factors for the relevant (relatively heavy) vehicles (Kalin et al, 2021) and it is hence not considered.

Using the designed elastic section modulus and the probabilistic model as described above, the reliability index is estimated with crude Monte Carlo. The estimate for a 100 year life gives $\beta = 4.1$ or $\beta = 4.6$ for $\gamma_{ff} = 1.0$ or 1.1 respectively but same model. This value should be compared with the target reliability of $\beta = 4.3$ (CC3, life time) for this case. A few notes should be made:

- The reliability varies per type of detail, since the standard deviation of the fatigue tests varies per type of detail and the slope parameter may also vary.
- The reliability also varies per road. Some motorways in Europe have a more aggressive traffic composition, others a less aggressive one as compared to the considered database, (Maljaars, 2020).
- Especially the long-term trend in traffic is highly uncertain. In that respect, the standard deviation used for θ_m may be too optimistic.
- A more refined fatigue resistance model may be adopted as compared to the one given here. In particular, random fatigue limit models are able to consider the uncertainty in the constant amplitude fatigue limit (Pascual et al, 1999) and variations thereof). Models are also available that consider a gradual change of the constant to the variable amplitude S-N curve, depending on the magnitude and number of ranges exceeding the constant amplitude fatigue limit (Leonetti et al, 2020). Adopting these models generally increases the estimated reliability.
- If the hot-spot stress method is used and hence θ_{scf} applies, a lower reliability index or a higher partial factor would follow from the analysis.

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B.8 Failure probability of deteriorated concrete structure

B.8.1 Introduction

The goal of this section is to present the results of the calculations of the annual probability of failure of a deteriorated bridge due to chloride-induced corrosion for a period of 100 years. In the following, the methodology used for the reliability analysis, the structure under investigation, the models of the structural performance and the probabilistic models are described.

B.8.2 Methodology

The annual probability of failure in year i is calculated by:

$$P_{f,i} = P(F_i | S_1 S_2 \dots S_{i-1}) \quad (B.65)$$

where:

F_i denotes the failure event during year i ;

S_1, S_2, S_{i-1} are the survival events during the first year, the second year and year $i-1$.

In the simple case where only one failure mode is considered, the annual probability of failure can be expressed as follows:

$$P_{f,i} = P[g(\mathbf{X}_i) \leq 0 | g(\mathbf{X}_1) > 0 \wedge g(\mathbf{X}_2) > 0 \wedge \dots \wedge g(\mathbf{X}_{i-1}) > 0] \quad (B.66)$$

where:

g is the limit state function;

$\mathbf{X}_i$ is the vector of the uncertain parameters considered in the limit state function.

The vector $\mathbf{X}$ contains both time-invariant and time-variant components. Examples of time-invariant components are the dead weight and the uncertainties of the structural models. Variable loads, the loss of reinforcement area due to corrosion are examples of time-variant uncertain parameters. Due to the presence of these time-variant parameters in the limit state function, vector $\mathbf{X}$ is different from year-to-year. In the following, the annual probability of failure is approximated by using the outcrossing approach:

$$P_{f,i} \approx P[g(\mathbf{X}_i) \leq 0 \cap g(\mathbf{X}_{i-1}) > 0] \quad (\text{B.67})$$

This failure probability is estimated by using the First Order System Reliability Method (Hohenbichler and Rackwitz, 1983):

$$\begin{aligned} P[g(\mathbf{X}_i) \leq 0 \cap g(\mathbf{X}_{i-1}) > 0] &= \\ = P[\alpha_i \mathbf{U} + \beta_i \leq 0 \cap \alpha_{i-1} \mathbf{U} + \beta_{i-1} > 0] &= \\ = P[Z_i \leq -\beta_i \cap Z_{i-1} > \beta_{i-1}] &= \Phi_2(\boldsymbol{\beta}; \boldsymbol{\rho}) \end{aligned} \quad (\text{B.68})$$

where:

$\mathbf{U}$ is the vector of independent random variables with standard normal distribution;

α_i is the vector of the sensitivity factors of the components of vector $\mathbf{X}_i$;

β_i is the reliability index estimated using a FORM reliability calculation considering the limit state function $g(\mathbf{X}_i)$;

Z_i is the standard normal random variable;

$\boldsymbol{\beta}$ is the vector of the reliability indices $[-\beta_i, \beta_{i+1}]$;

$\boldsymbol{\rho}$ is the 2 x 2 correlation matrix, whose off-diagonal elements are:

$$\rho_{12} = \rho_{21} = -\alpha_{i-1} \cdot \alpha_i \quad (\text{B.69})$$

$\Phi_2(\cdot, \cdot)$ is the CDF function of the bi-variate normal distribution.

Only the components of the sensitivity vector α of the time-invariants random variables are considered in the calculation of the correlation matrix.

Description of the case

The structure considered in this section is a simply supported slab bridge Holicky and Sykora, 2009). The slab is 11 m long, 15,6 m wide and 0,6 m thick. The concrete cover is equal to 50 mm. The concrete class is C30 and the steel grade is S500.

The bridge is subjected to the permanent load g and the traffic load q . The characteristic values of the bending moment $E(G_k)$ and $E(Q_k)$ in the midspan section are, per meter of width, equal to 340 kNm/m and 330 kNm/m. The longitudinal reinforcement consists of bars with diameter of 25 mm and the reinforcement area is equal to 4310 mm²/m (per meter of width), leading to the unity check value ($UC = E_d / R_d$) of 1:

$$UC = \frac{E_d}{R_d} = \frac{\gamma_G E(G_k) + \gamma_Q E(Q_k)}{A_s \frac{f_{yk}}{\gamma_s} \left[h - \left(c + \frac{d}{2} \right) - \frac{A_s \frac{f_{yk}}{\gamma_s}}{2\alpha \frac{f_{ck}}{\gamma_c}} \right]} = \quad (\text{B.70})$$

$$= \frac{1,35 \cdot 340 + 1,35 \cdot 330}{10^{-6} \cdot 4310 \cdot \frac{500}{1,15} \cdot \left[600 - \left(50 + \frac{25}{2} \right) - \frac{4310 \cdot \frac{500}{1,15}}{2 \cdot 10^3 \cdot 0,85 \cdot \frac{30}{1,5}} \right]} = \frac{904}{904} = 1$$

where:

$\gamma_s (=1,15)$	is the partial factor for steel reinforcement;
$\gamma_c (=1,5)$	is the partial factor for concrete compressive strength;
$\gamma_G (=1,35)$	is the partial factor for permanent action;
$\gamma_Q (=1,35)$	partial factor for traffic load;
$\alpha (=0,85)$	is the factor accounting for the long-term effects on the concrete compressive strength.

B.8.3 Limit state modelling

Only the limit state of flexural failure is considered. It is assumed that the resistance of the structure decreases in time due to corrosion of the reinforcement caused by the use of de-icing salts. The limit state function is expressed as:

$$g(\mathbf{X},t) = \theta_{MR} M_R(t) - E_G - \theta_{EQ} E_Q \quad (B.71)$$

where:

- θ_{MR} is the model uncertainty of the resisting bending moment;
- $M_R(t)$ is the resisting bending moment at time t ;
- E_G and E_Q are the bending moments induced in the midspan section by the permanent and traffic loads;
- θ_{EQ} is the model uncertainty of the traffic load-induced bending moment.

The resisting bending moment $M_R(t)$ at time t is evaluated as follows

$$M_R(t) = A_s g(t) f_y \left[h - \left(c + \frac{d}{2} \right) - \frac{A_s f_y}{2 \alpha f_c} \right] \quad (B.72)$$

The degradation function $g(t)$ is defined as:

$$g(t) = \begin{cases} 1 & \text{if } t \leq T_{init} \\ \left[1 - \frac{2 \cdot 0,0116 \cdot 1,2 \cdot i_{corr0} (t - T_{init})^{0,71}}{d_0} \right]^2 & \text{if } t > T_{init} \end{cases} \quad (B.73)$$

where:

- i_{corr0} is the initial corrosion rate in $\mu A/cm^2$;
- T_{init} is time to initiation of corrosion in years;
- d_0 is the initial diameter of the bars.

B.8.4 Probabilistic model

The probabilistic model of the parameters involved in the limit state functions (Equations (B.71), (B.72) and (B.73)) are listed in Table B.13.

Table B.13 Input for case under consideration

Variable	Description	Units	Distribution	Mean value	CoV	Note
h	Cross-section height	m	Normal	0,60	0,017	Time-invariant
c	Concrete cover	mm	Gamma	60	0,17	Time-invariant
A_s	Initial reinforcement area	mm ²	Normal	4310	0,03	Time-invariant
f_y	Yield stress S500	N/mm ²	Lognormal	560	0,054	Time-invariant
f_c	Concrete compressive strength C30	N/mm ²	Lognormal	37,5	0,13	Time-invariant
E(g)	Permanent load	kNm/m	Normal	340	0,10	Time-invariant
E(q)	Traffic load (1-year max)	kNm/m	Gumbel	290,4	0,02	Time-variant
Δ_{EQ}	Mod. unc. bending moment (load effect)	-	Lognormal	1,0	0,15	Time-invariant
Δ_{MR}	Mod. unc. bending moment (resistance)	-	Normal	1,08	0,1	Time-invariant
i_{corr0}	initial corrosion rate	$\mu\text{A}/\text{cm}^2$	Normal	2,7	0,63	Time-invariant
T_{init}	Time to initiation of corrosion	y	Exponential	7,4	1,0	Time-invariant

B.8.5 Results

The annual probability of failure and the annual reliability index are presented in Figure B.15 and Figure B.16 for a period of 100 years.

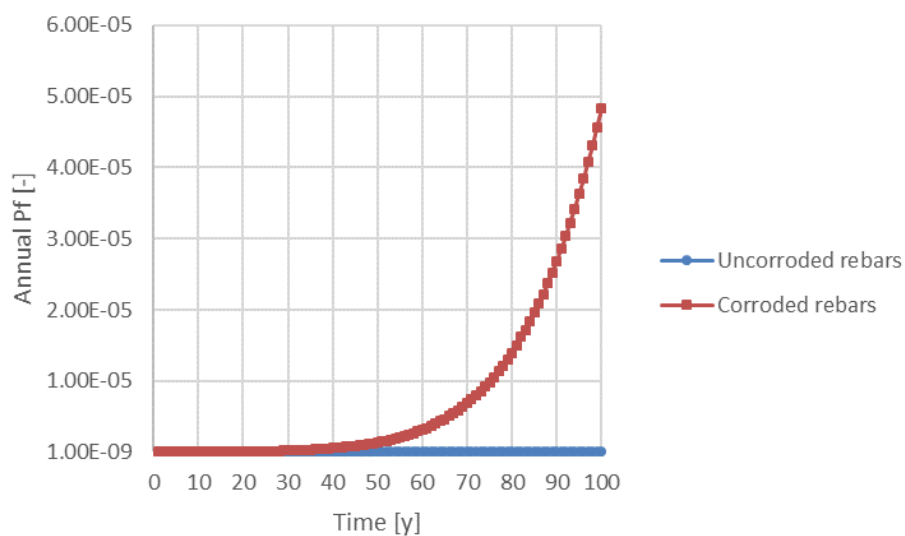
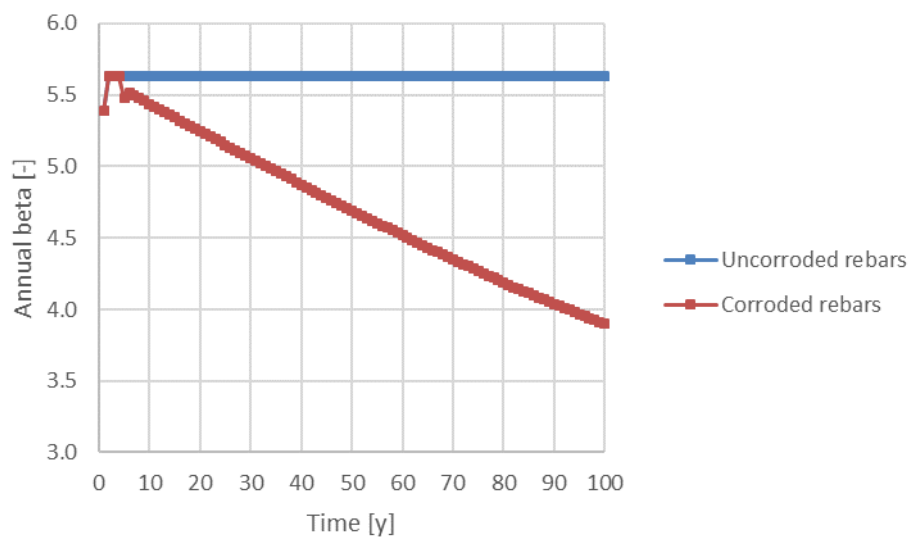
Figure B.15. Annual probability of failure

Figure B.16. Annual reliability index



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- Hohenbichler, M., and Rackwitz, R. (1983), 'First-order concepts in system reliability', *Structural Safety*; Vol. 1, pp. 177-188
- Holicky, M., Sykora, M. (2009), 'Assessment of deteriorating reinforced concrete road bridge', in: *Proceedings of the ESREL2009 conference*

B.9 Example risk based approach: Observational method in geotechnics

B.9.1 Decision-theoretical aspects of the observational method

As discussed in section 3.9 of the main text an inherent issue in geotechnical engineering is the considerable lack of knowledge (epistemic uncertainty) of the ground properties at the outset of a new project. Although geotechnical investigations can be used to increase the knowledge about the ground, the investigations may at times turn out too difficult or expensive, to determine the ground properties with adequate precision. In such cases, the observational method offers a design framework to increase the knowledge about the ground properties and how they affect the structural behaviour. The observational method is defined in EN 1997-1, clause 4.7.

The observational method can best be demonstrated by considering the decision tree as shown in Figure B.17. In the general case there exists a number of design alternatives $E = \{e_1, e_2, \dots\}$ as branches extending from the leftmost node, but here only two alternatives are shown for illustrative purposes, the observational method and a conventional design alternative. If the observational method is applied, structural behaviour is monitored during construction. The outcome Z determines whether or not to implement one of the prepared contingency measures $A = \{a_1, a_2, \dots\}$, which would modify the design from the initial variant into another prepared design variant. In other words, the decision to modify the design into another variant is not free, but dictated by passing monitoring thresholds that have been established in the monitoring plan. In the last branching Θ , it is revealed whether limit state violation occurs with probability p_F .

Each scenario is associated with some cost, the base cost $C(e_{OM})$ including development of all design variants and implementation of one of them at the outset, as well as the cost to prepare and execute the monitoring; the cost $C(a)$ to change into another design variant; and the cost $C(F)$ for violated limit state. The conventional design only has two costs: the base cost $C(e_{CON})$ for the implementation of the design, and $C(F)$ for a violated limit state. The optimal design alternative is the one with the smallest expected cost, conditionally that it satisfies the target failure probability, $p_{F,T}$. This can be determined straightforwardly with a pre-posterior analysis (see e.g. the textbook by Benjamin and Cornell, 1970). An example is presented in section B10.3 (summarised from Spross and Johansson, 2017).

B.9.2 Reliability-based monitoring thresholds

An important safety aspect of the observational method is how monitoring thresholds are determined, as they ensure sufficient safety margin to the limit state. On the other hand, the thresholds cannot be set too conservatively, as they then will trigger expensive contingency measures that are not really warranted. Applying reliability-based design principles, Spross and Johansson (2017) showed how reliability-based thresholds could be established when applying the observational method. Based on the target failure probability, $p_{F,T}$, a threshold x_{alarm} for a monitored parameter X_1 can be determined by solving the following equality:

$$P(G(\mathbf{X}) < 0 \mid X_1 < x_{alarm}) = p_{F,T} \quad (B.74)$$

where $G(\mathbf{X}) = 0$ is the considered limit state with associated random variables in the vector $\mathbf{X}$. Equation B.74 is derived from the definition of conditional probability. It can be interpreted such that the probability of limit state violation is acceptable, conditionally on the measurement readings falling below the threshold. Equation B.74 assumes a time-invariant analysis; Spross and Gasch (2019) discussed the implications of having to account explicitly also for an unavoidable, triggered lead time between threshold violation and an unacceptable structural behaviour.

Figure B.17. Observational method compared against a conventional design in a decision tree (Spross and Johansson, 2017), CC-BY, <http://creativecommons.org/licenses/by/4.0/>, minor changes)

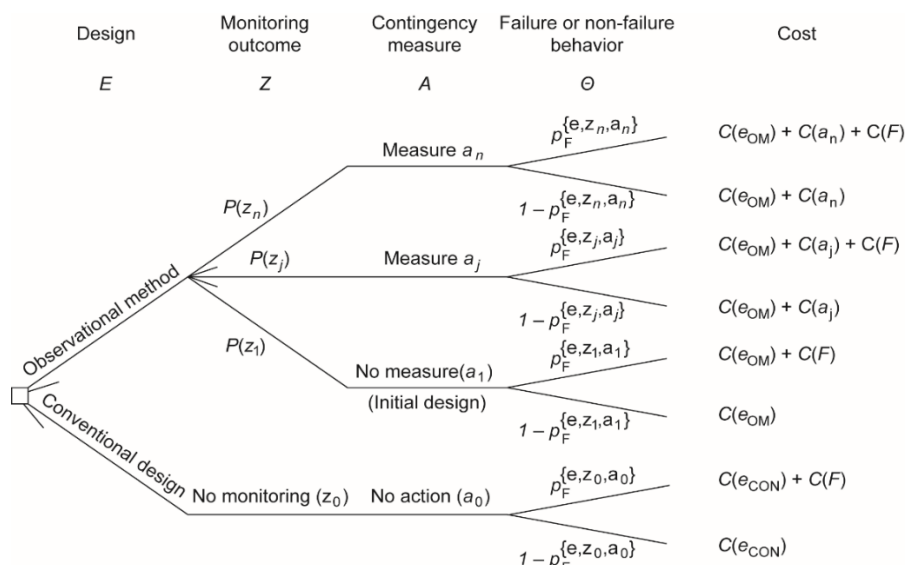


Figure B.18. Section of the analysed rib pillar. Should rock anchors be installed immediately, or is it more favourable to first observe the pillar deformation during excavation (making the pillar thinner and thinner) and then install rock anchors only if measurements show that they are needed.

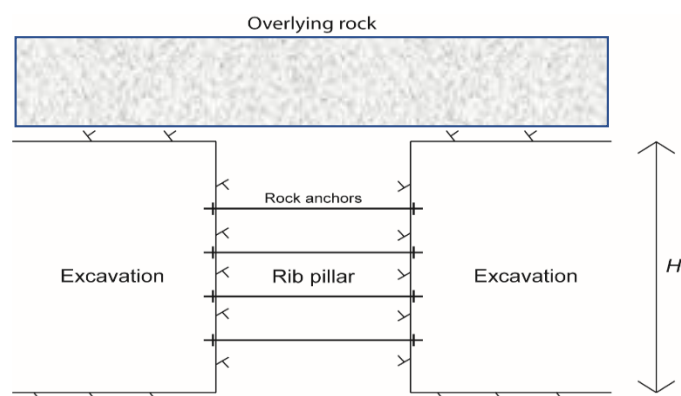
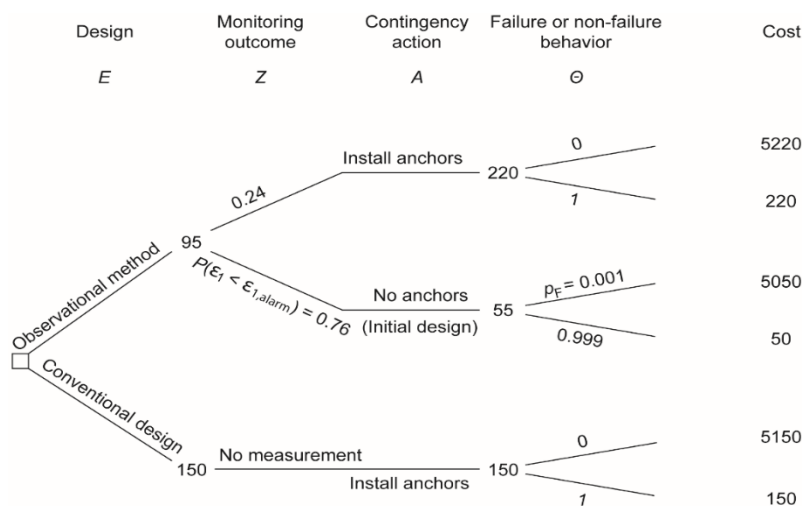


Figure B.19. For the analysed rib pillar, the observational method is found more favourable, because the expected cost of that design alternative (95 MU) is less than that of the conventional design (150 MU) (Spross and Johansson, 2017), CC-BY, <http://creativecommons.org/licenses/by/4.0/>, minor changes)



Application example: vertical deformation of a rock pillar

A very long rib pillar is to be excavated in rock (Figure B.18). The limit state concerns its vertical strain ϵ_1 , which must not exceed a maximum strain $\epsilon_{1,max}$, i.e. we have that $G(\mathbf{X}) = \epsilon_{1,max} - \epsilon_1$, where $\mathbf{X}$ collects a number of underlying rock mass properties as random variables (the reader is referred to Spross and Johansson (2017) for the complete geomechanical model). In this case $p_{F,T}$ is set equal to 0,001. Performing a reliability-based analysis (e.g. using Monte Carlo simulation) it is found that for an unsupported pillar, the probability of limit state violation at the end of the excavation process is $p_F = 0,0046$, which exceeds $p_{F,T}$. However, if four horizontal rock anchors are added in the model to provide confinement, p_F becomes negligible.

The decision maker now stands before two alternatives:

1. to go directly with the more expensive design using the rock anchors (i.e. a conventional design), or
2. to apply the observational method to reduce the uncertainty about the rock mass properties by monitoring the deformation of the unreinforced pillar during the excavation process.

The excavation is assumed to be carried out stepwise, making the pillar thinner and thinner with each step. Measurements are made after each excavation step, providing a number of measurements from which a trendline is derived to predict the final deformation, including the scatter.

If the actual ground conditions turn out to be favourable, rock anchors will not be needed after all. The alternatives are illustrated as a decision tree in Figure B.19 together with their corresponding costs. The cost components are given by (MU = monetary unit):

anchor system 150 MU is applied directly or 170 MU if applied later,

advanced monitoring system: 20 MU

cost increase due to more careful excavation using the observational method: 30 MU

failure 5000 MU

The threshold for the vertical strain without anchors can be found from application of Equation. (B8.1), in this case $P(G(\mathbf{X}) < 0 | \epsilon_1 \leq \epsilon_{1,alarm}) = p_{F,T}$ leading to $\epsilon_{1,alarm} = 0,33 \cdot 10^{-3}$ (corresponding to 2,7 mm deformation). This allows evaluation of the probability of the unsupported design variant being successful: $P(\epsilon_1 \leq \epsilon_{1,alarm}) = 76\%$. Hence, threshold violation that triggers the rock anchor installation occurs with 24% probability.

A so-called pre-posterior analysis shows that the expected cost of applying the observational method is 95 MU, while the expected cost of the conventional design is 150 MU. The analysis is performed by weighing the costs with respect to their probability of occurrence in the decision tree. Based on the result, a risk-neutral decision maker should choose the observational method. Note however that if the observations trigger the contingency measure to install rock anchors, the final cost becomes larger (220 MU) than that of the conventional design alternative. A risk-avoiding decision maker may therefore consider the conventional design better, as it comes with a known cost.

Note that in this case the cheapest solution is “to do nothing” and accept the high probability of failure leading to a cost expectation of $0,0046 \cdot 5000 \text{ MU} = 23 \text{ MU}$. Whether this is legally allowed may depend on the type of limit state (SLS or ULS) and the relevant clauses in the National Annex, as $p_F = 0,0046$ exceeds $p_{F,T}$.

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