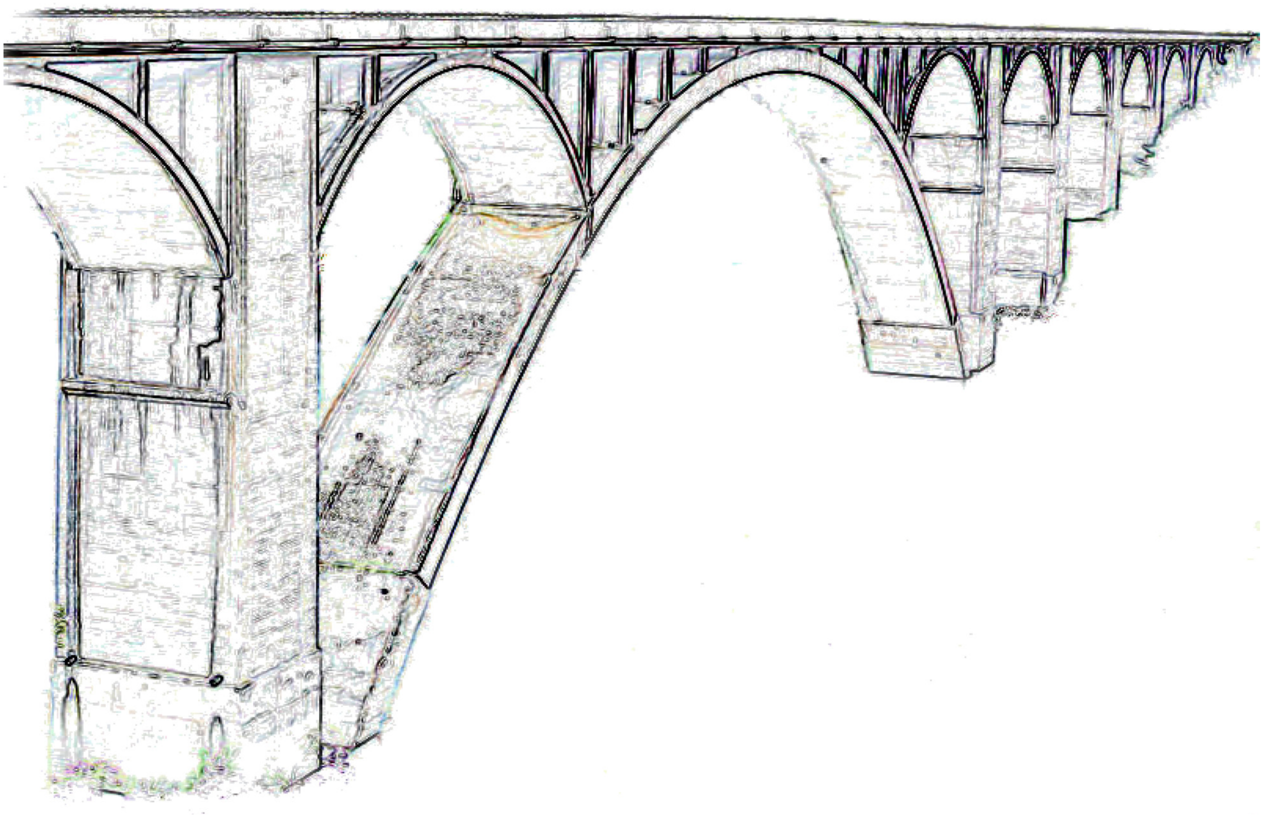
HANDBOOK 1

BASIS OF STRUCTURAL DESIGN



Guide to Interpretative Documents for Essential Requirements, to EN 1990 and to application and use of Eurocodes

LEONARDO DA VINCI PILOT PROJECT CZ/02/B/F/PP-134007

Leonardo da Vinci Pilot Project CZ/02/B/F/PP-134007

DEVELOPMENT OF SKILLS FACILITATING IMPLEMENTATION OF EUROCODES

HANDBOOK 1

BASIS OF STRUCTURAL DESIGN

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Garston, Watford, UK, 4.2004

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BASIS OF STRUCTURAL DESIGN

FOREWORD

The Leonardo da Vinci Pilot Project CZ/02/B/F/PP-134007, "Development of Skills Facilitating Implementation of Structural Eurocodes" addresses the urgent need to implement the new system of European documents related to design of construction works and products. These documents, called the Eurocodes, are systematically based on the recently developed Council Directive 89/106/EEC "The Construction Products Directive" and its Interpretative Documents ID1 and ID2. Implementation of Eurocodes for each CEN Member is a challenging and sometimes difficult task as each country has its own long-term tradition in design and construction.

The output from the project should enable an effective implementation and application of the new methods for designing and verification of buildings and civil engineering works in all the project partner countries (CZ, DE, ES, IT, NL, SI, UK) and in other countries implementing the Eurocodes. The need to explain and effectively use the latest principles specified in European standards is apparent from concerns expressed by the profession, various enterprises, undertakings and public national authorities involved in the construction industry and also from universities and colleges. There is an urgent need for training materials, manuals and software products and other tools to help implementation.

This Handbook 1 is one of 5 handbooks intended to provide required manuals and software products and other tools for training, education and effective implementation of Eurocodes. The five handbooks are as follows:

Handbook 1: Basis of structural design
Handbook 2: Basis of Structural Reliability and Risk Engineering
Handbook 3: Design of Buildings
Handbook 4: Design of Bridges
Handbook 5: Design of Buildings under Fire Situation

The objective is for the Handbooks to address the following intents in further harmonisation of the European construction industry

- structural reliability improvement and harmonisation of the process of design;
- development of the single market for products and for construction services;
- provide new opportunities for the trained target groups in the labour market.

This Handbook 1 is focused European Commission legislative instruments on construction and on Basis of Structural Design related to head Eurocodes EN 1990. The following topics are treated in particular:

- system of European Codes and Standards for construction
- the Construction Products Directive and Essential Requirements for products
- Interpretative Document 1 - Mechanical resistance and stability
- Interpretative Document 2 - Fire safety
- limit states and methods of partial factors
- classification of actions
- resistance and geometric data
- load combinations according to EN 1990

A wide range of potential users of the Handbooks and other training materials includes practising engineers, designers, technicians, officers of public authorities, young people - high school and university students. The target groups come from all territorial regions of the partner countries. However, the dissemination of the project results is foreseen to be spread into the other Members of CEN.

Garston, Watford, UK, 4.2004

CHAPTER I: SYSTEM OF EUROPEAN CODES AND STANDARDS FOR CONSTRUCTION WORK

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Summary

The background to the Eurocode programme is given and the potential benefits of adoption to the profession stated. The Eurocode programme, their format and development to EN are discussed. Details are given on the national format for the Eurocodes, and the contents of National Annexes. Eurocode packages, and the transitional and the co-existence of Eurocodes with National Provisions, are discussed. Future actions regarding maintenance are discussed.

1 INTRODUCTION

The complete suite of the CEN Structural Eurocodes, which presently exists in ENV (European Pre-standard) form, will be converted to full EN (European Standard) by 2006. The first package of Eurocodes relating to the design of common buildings will be converted earlier, possibly by 2005. The package relating to the design of bridges should be converted by 2005. Following a period of co-existence between the Eurocodes and the present National Codes, the National Codes will cease to be maintained; this period is expected to be between 3-6 years. The European Commission (EC), in close co-operation with representatives of Member States (the Eurocode National Correspondents (ENC)) has prepared a document "Application and Use of the Eurocodes" [1].

2 BACKGROUND TO THE EUROCODE PROGRAMME

2.1 The objectives of the Eurocodes and their status

In 1975, the Commission of the European Community decided on an action programme in the field of construction based on article 95 of the Treaty of Rome. The objective of the program was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme the Commission took the initiative to establish a set of harmonised technical rules for the structural design of construction works, with the following European Commission objective:

"The Eurocodes to establish a set of common technical rules for the design of buildings and civil engineering works which will ultimately replace the differing rules in the various Member States".

For fifteen years, the Commission, with the help of a Steering Committee containing representatives of Member States, conducted the development of the Eurocode programme, which led to the publication of a first generation set of European codes in the 1980's.

In 1989 the Special Agreement between CEN and the European Commission (BC/CEN/03/89) specified that the Eurocodes are intended to serve as reference documents to be recognised by authorities of the Member States for the following purposes:

a) as a means of compliance of building and civil engineering works with the Essential Requirements as set out in Council Directive 89/106/EEC (The Construction Products Directive), particularly Essential Requirement No 1 - *Mechanical resistance and stability* and Essential Requirement No 2 - *Safety in case of fire.* The use of EN Eurocodes in technical specifications for products is described in the Commissions Guidance paper L, "Application and Use of Eurocodes" [1],

b) as a basis for specifying contracts for the execution of construction works and related engineering services in the area of public works. This relates to the Council Procurement Directive 204/18/CE:

- For Works, which covers procurement by public authorities of buildings and civil engineering works, with a current threshold of about 5m Euros.
- For Services, which covers procurement of services by public authorities, with current thresholds for Government Departments of 130k Euros and others, including local authorities of 200k Euros.

c) as a framework for drawing up harmonised technical specifications for construction products.

In addition the Eurocodes are foreseen to:

- improve the functioning of the single market for products and engineering services, by removing obstacles arising from different nationally codified practices for the assessment of structural reliability
- improve the competitiveness of the European construction industry, and the professionals and industries connected to it, in Countries outside the European Union.

2.2 Potential Benefits of the use of Eurocodes

The intended benefits of the Eurocodes are:

a) To provide a common understanding regarding the design of structure between owners, operators and users, designers, contractors and manufacturers of construction products;

b) To provide common design criteria and methods to fulfil the specified requirements for mechanical resistance, stability and resistance to fire, including aspects of durability and economy;

c) To facilitate the marketing and use of structural components and kits in Member States;

d) Facilitate the marketing and use of materials and constituent products the properties of which enter into design calculations, in Member States;

e) Be a common basis for research and development;

f) To allow the preparation of common design aids and software;

g) Benefit the European civil engineering enterprises, contractors, designers and product manufacturers in their world-wide activities, and increasing their competitiveness.

2.3 The Eurocodes and National Regulations/Public Authority Requirements

There is a clear and vital distinction between design codes and National Regulations/ Public Authority Requirements. Harmonisation of national requirements is outside the scope of Eurocode development. It is the objective however that the Eurocodes should be recognised in National Regulations as one of the routes for meeting compliance. In accordance with normal rules following the introduction of European Standards, Eurocodes will be called up in public procurement specifications, and to be used for the design of products for the purpose of obtaining a CE (Conformité Européen) mark.

3 EUROCODE PROGRAMME

3.1 Programme

The structural Eurocodes as shown in Table 1, each generally consisting of a number of parts, some of which relate to bridges, will be released in EN form between 2002 and 2005.

All exist at present as ENVs.

Table 1. The Structural Eurocodes

EN Number	The Structural Eurocodes
EN 1990	Eurocode: Basis of Structural Design
EN 1991	Eurocode 1: Actions on structures
EN 1992	Eurocode 2: Design of concrete structures
EN 1993	Eurocode 3: Design of steel structures
EN 1994	Eurocode 4: Design of composite steel and concrete structures
EN 1995	Eurocode 5: Design of timber structures
EN 1996	Eurocode 6: Design of masonry structures
EN 1997	Eurocode 7: Geotechnical design
EN 1998	Eurocode 8: Design of structures for earthquake resistance
EN 1999	Eurocode 9: Design of aluminium structures

Each of the ten Eurocodes listed in this paper are made up of separate parts, which cover the technical aspects of the structural and fire design of buildings and civil engineering structures. The Eurocodes are a harmonised set of documents that have to be used together. Figure 1 shows the structures and links of the Eurocodes. In Annex C of Appendix A the various parts are listed.

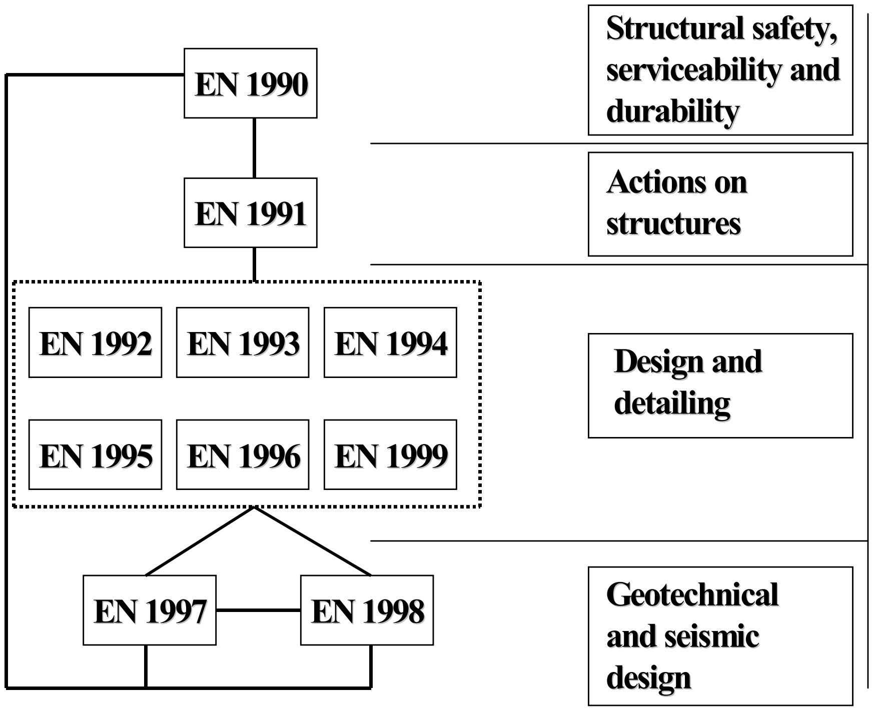


Figure 1. Links between the Eurocodes.

3.2 Format of the Eurocodes

The format of the Eurocodes is different from National Codes in that all its clauses are designated *Principles or Application Rules.*

The *Principles* are those fundamental bases of structural performance, which must be achieved.

Application Rules are recommended methods of achieving those principles.

It is permissible to use alternative design rules different from the Application Rules given in the Eurocodes for works, provided it is shown that the alternative design rules accord with the relevant Principles and are at least equivalent with regard to the structural safety, serviceability and durability which would be expected when using the Eurocodes.

To clarify the use of alternative design rules, the Eurocodes further state that *if an alternative design rule is substituted for an application rule, the resulting design cannot be claimed to be wholly in accordance with the relevant Eurocode although the design will remain in accordance with the Principles of the Eurocode. When the Eurocode is used in respect of a property listed in an Annex Z of a product standard or an ETAG (European Technical Approval Guideline), the use of alternative design rules may not be acceptable for CE marking.*

3.3 Using ENV Eurocodes nationally

Currently the ENV Eurocodes may be used for design purposes, in conjunction with the National Application Document (NAD) applicable to the Member State where the designed structures are to be located. NADs provide essential information for example on:

- safety and other requirements of National Regulations including national values for those values given in a box shown thus [ ]

- use of national supporting standards, if for example a European Product Standard is not yet ready.

3.4 Development from ENV to EN

The conversion of the Eurocodes from the ENV stage to EN is happening now, and six Eurocode Parts had reached EN stage by October 2003. CEN/TC250, the responsible CEN committee has asked its subcommittees to consider in the conversion process only:

- national comment on ENVs
- feedback from users on the ENVs
- co-ordination conditions
- format and editorial consistency.

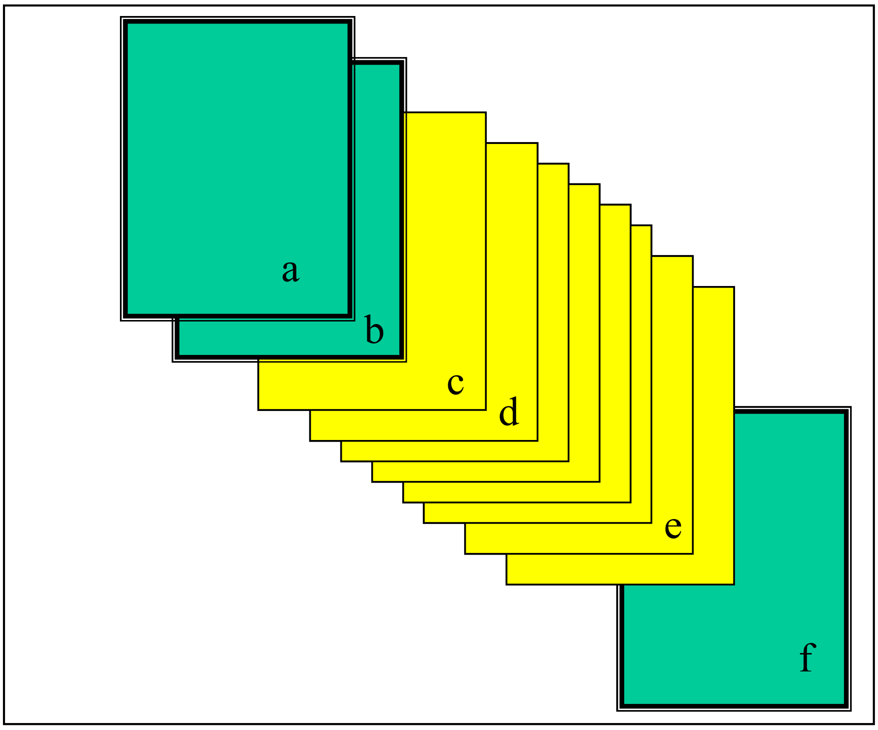
Unlike the ENV Eurocodes, the EN Eurocodes will not have NADs and Box Values, but National Annexes.

3.5 Using EN Eurocode at a national level

In the final EN Eurocodes, National Annexes will replace National Application Documents and box values.

It is the responsibility of each National Standards Body (e.g. British Standards Institute (BSI) in the UK) to implement Eurocodes as National Standards.

The National Standard implementing EN 1990, and the National Standards implementing each Eurocode part, will comprise, without any alterations, the full text of the Eurocode and its annexes as published by the CEN. This may be preceded by a national title page and national foreword, and may be followed by a National Annex (see Figure 2).



Key

a: National Title Page
b: National Foreword
c: EN Title page
d: EN Text
e : EN Annex(es)
f : National Annex

Figure 2. National implementation of Eurocodes.

3.6 Rules and Contents of National Annexes for Eurocodes

The European Commission recognises the responsibility of regulatory authorities (e.g. the Building Regulations Division of the Office of the Deputy Prime Minister in the UK) or national competent authorities (e.g. the Highways Agency or Railway Safety in the UK) in each EU Member State. It has safeguarded their right to determine values related to safety matters at national level through a National Annex.

The National Annex may only contain information on those parameters, which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned.

3.6.1 Nationally Determined Parameters

Possible differences in geographical or climatic conditions (e.g. wind or snow maps) or in ways of life, as well as different levels of protection that may prevail at national, regional or local level, can be taken into account by choices left open about values, classes or alternative methods that are identified in the EN Eurocodes to be determined nationally.

The values, classes or methods to be chosen or determined at national level, called 'Nationally Determined Parameters', will allow the EU Member States to choose the level of safety, including aspects of durability and economy applicable to works in their territory. They include:

- values and/or classes where alternatives are given in the Eurocodes

- values to be used where only a symbol is given in the Eurocodes
- country-specific data (geographical, climatic, etc), e.g. snow maps
- procedures to be used where alternative procedures are given in the Eurocodes

3.6.2 National Annexes

The National Standards bodies should publish the parameters in a National Annex on behalf of and with the agreement of the national competent authorities.

A National Annex is not required if the EN Eurocode Part is not relevant for the Member State (e.g. seismic design for some countries).

The Annex may also contain

- decisions on the application of informative annexes
- references to non-contradictory complementary information to assist the user in applying the Eurocode.

A National Annex cannot change or modify the content of the EN - Eurocode text in any way other than where it indicates that national choices may be made by means of Nationally Determined Parameters.

3.6.3 Example 1

In EN 1990, all safety factors are given as symbols, with recommended values for the symbols given in notes. The National Annex may either adopt the recommended values or give alternative values.

3.6.4 Example 2

EN 1990 gives three alternative proposals in two instances:

a) the expressions combining action effects
b) soil/structure interaction

The National Annex will make a choice of one of the alternatives.

3.6.5 Annexes not transferable

Each EU Member State will have a different National Annex – the Annex used must be the one applicable to where the building or civil engineering work is being constructed.

For example, a UK designer will have to use the appropriate Eurocode with the UK National Annex when designing a building in the UK. The same designer, designing a building in Italy, will have to use the Eurocode with the Italian National Annex.

It is probable that National Annexes will be sold separately to the Eurocodes, so that a UK engineer designing a structure in France need only purchase the French National Annex.

3.7 Packages of EN Eurocode Parts

To facilitate the adoption of the Eurocodes their various parts will be grouped into packages, see Annex C of Appendix A, which generally relate to different types of structures and materials. The Date of Withdrawal (DOW) of conflicting National Standards and/or conflicting National Provisions will take place, at the end of the agreed co-existence period (see 3.8). This co-existence period may only begin when all parts in a package are available.

When a National Standard or Provision has a wider scope than the conflicting Eurocode package, withdrawal concerns only the part of the Scope covered by the Package.

No Codes from EN 1990 or the EN 1991, EN 1997 or EN 1998 series form a package in themselves; those Parts are placed in each of the Packages as material independent.

3.8 Adoption of the Eurocodes nationally and period of co-existence of EN Eurocodes with National Rules

3.8.1 Requirements for adoption

Before an EN Eurocode Part can be accepted/adopted to the Member States national legislation/regulation, the following requirements must be complied with:

a) The Eurocode part must be fit for implementation.
b) The calculations, executed on the basis of the Eurocode Parts together with the Nationally Determined Parameters, shall provide an acceptable level of safety.
c) The use of Eurocode Part(s), with Nationally Determined Parameters does not lead to structures that cost significantly more, over their design life, than those designed to National Standards or provisions, unless changes in safety have been made and agreed.

3.8.2 Transitional arrangements

The term "transitional arrangements" refers to the time period during which existing National Provisions and Eurocodes are both available for use. (See Figure 3)

3.8.3 Periods in the Transitional Arrangements

The transitional arrangements period, which commences from the time the final draft of EN Eurocode is produced by the Project Team, includes five sub-periods as follows.

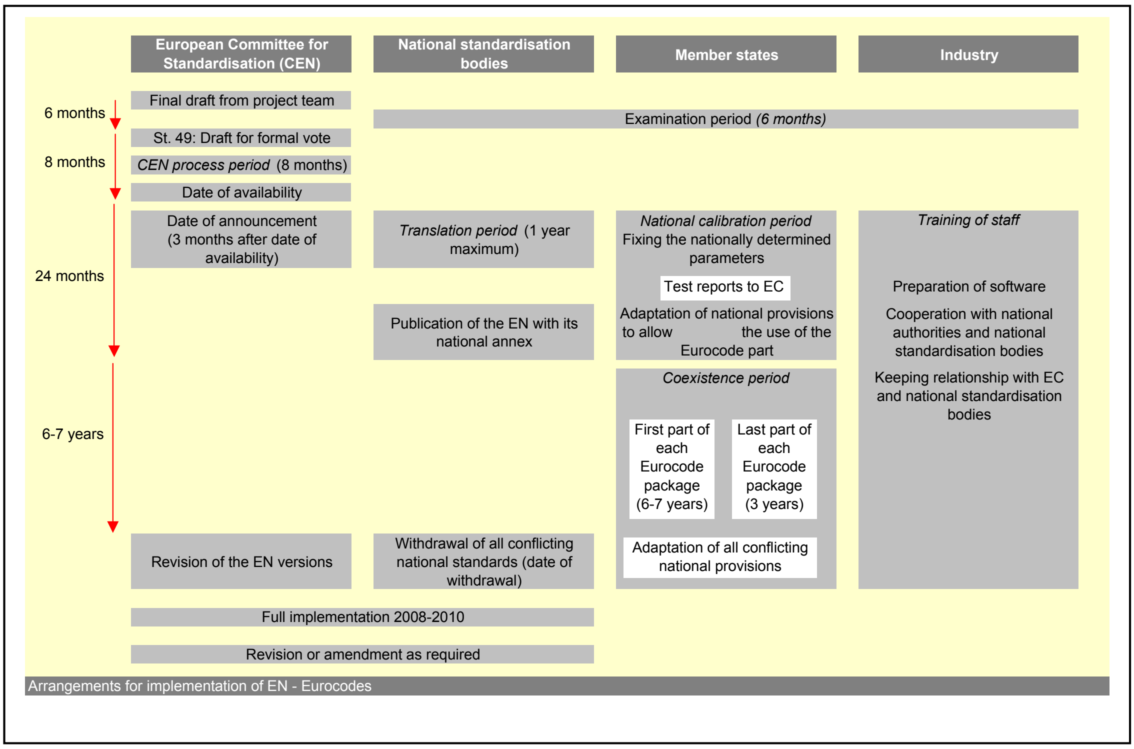


Figure 3. Arrangements for national implementation of Eurocodes.

a) Two periods before the date of availability (DAV)

i) *The examination period*, to enable sub-committees and national delegations to approve the document that should be available in English, French and German, for formal vote (six months).

ii) *A CEN process period,* which allows CEN to perform the operation for a formal vote, ratification, and making available the Eurocode at the DAV (eight months).

b) Three periods after the date of availability (DAV)

i) *A translation period* to enable the translation of the Eurocode Part into the national languages, other than English, French and German (maximum one year).

ii) *A national calibration period*, for the establishment of the Nationally Determined Parameters and the National Annex (maximum two years).

iii) *Coexistence period* - during which the Eurocode Part coexists with national codes, standards or provisions. At the end of this period, all conflicting National Standards shall be withdrawn and all National Provisions conflicting with the scope of the Eurocode shall be adapted. However, withdrawal of the relevant national code will only occur when the last Eurocode Part of a package ends its co-existent period. Nevertheless, contradicting regulations will be changed to allow legitimate use of earlier parts.

4 MAINTENANCE OF THE EN EUROCODES

Under the CEN rules all ENs will have a five-year review. The primary objective for the first review will be to reduce the number of Nationally Determined Parameters for the Eurocodes. This represents a strong wish from the EC. In the case of Eurocodes, the responsible TC will be CEN/TC250.

5 COMMISSION RECOMMENDATION ON EUROCODES

The European Commission have produced a strong recommendation (Brussels, 11 December 2003) to Member States regarding the implementation of the Eurocodes [2]. The recommendations are summarised below:

i) EU Member States are asked to adopt the Eurocodes for designing construction works, checking the mechanical resistance of components, and checking the stability of structures. In the case of construction works designed using the calculation methods described in the Eurocodes, there will be a presumption of conformity with
- essential requirement No.1 'Mechanical resistance and stability',
- such aspects of essential requirement No 4 'Safety in use' as relate to mechanical resistance and stability, and
- with part of essential requirement No 2 'Safety in case of fire,

as referred to in Annex 1 to Directive 89/106/EEC (the Construction Products Directive).

ii) Member States are asked to select NDPs (see 3.3.1) usable in their territory.

iii) For clauses where NDPs have been identified in the Eurocodes, Member States are urged to use the recommended values provided by the Eurocodes. Divergence from the recommended values should only be, where geographical, geological or climatic conditions or specific levels of protection make that necessary. The Commission ask that all NDPs in force in a territory are notified to them within two years of the date on which the Eurocodes become available.

iv) Acting in co-ordination, the Commission has requested Member States to compare the NDPs implemented by each Member State, and assess their impact on the technical differences for works or parts of works, and, accordingly, to change their NDPs in order to reduce divergence from the recommended values provided by the Eurocodes.

v) Member States need to refer to the Eurocodes in their national provisions on structural construction products.

vi) Member States are asked to undertake research, in collaboration, to integrate into the Eurocodes of the latest developments in scientific and technological knowledge, thus ensuring an ongoing increased level of protection of buildings and civil works.

vii) Member States are urged to promote instruction in the use of the Eurocodes, especially in Universities and as part of continuous professional development courses for the profession.

6 REFERENCES

[1] *Guidance Paper L (concerning the Construction Products Directive - 89/106/EEC) - Application and Use of Eurocodes:* European Commission, Enterprise Directorate-General, 2004.
[2] *EN 1990: Eurocode: Basis of Structural Design* – CEN, 2002.
[3] *Commission Recommendation of 11 December 2003 on the implementation and use of Eurocodes for Construction Works and Construction Products*. Office of the European Union pp L 332/62 - 63

CHAPTER II: THE CONSTRUCTION PRODUCTS DIRECTIVE AND ESSENTIAL REQUIREMENTS FOR PRODUCTS

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Summary

After 25 years in the making, the suite of 10 European structural design standards – generally known as the Eurocodes – is at last becoming a reality. A total of over 57 individual parts have been circulating in working draft form for several years but six have finally been converted to full EN (EuroNorm) status in October 2003. All other parts should be converted in a series of packages by 2005 and, following a few years of coexistence, will replace all existing structural design standards in every CEN Member State by the end of the decade. This paper provides an introduction to the Eurocodes and their links to European legislation (e.g. the CPD), and describes the potential benefits they offer to civil engineers and sets out the process and timetable for implementation.

1 INTRODUCTION

1.1 Background documents

The complete suite of structural Eurocodes being prepared by the Comite Europeen de Normalisation (CEN), the European committee for standardisation, will be converted to full EuroNorm (EN) status by 2005 (see Table 1in Chapter I). The head Eurocode, EN1990, and the first Part of Eurocode 1 were converted in October 2001. Four further parts of EN 1991 have since been converted. All others are currently in 'pre-standard' form designated by ENV (meaning EuroNorm Vornorm). The first package of Eurocodes, relating to design of common buildings, will be converted earlier – possibly by 2004/5 and the package relating to bridge design should follow in 2004/5.

Following a period of coexistence between Eurocodes and each European country's current national design codes, the national codes will cease to be maintained. This period is yet to be negotiated but is expected to be between three and six years.

2 BACKGROUND TO EUROCODES AND THEIR STATUS

2.1 Background Documents

By way of guidance the European Commission, in close cooperation with representatives from each Member State (the Eurocode national correspondents) is preparing a document Guidance Paper L *Application and use of the Eurocodes* [1], the latest version of which is in Annex A to this handbook.

3 CE MARKING UNDER THE CONSTRUCTION PRODUCTS DIRECTIVE

3.1 General

This section explains the implications of CE marking under the CPD for manufacturers, specifiers, certification and test bodies and regulatory / enforcement authorities.

3.2 The framework of the CPD

The CPD aims to break down technical barriers to trade in construction products between Member States in the European Economic Area (EEA). To achieve this the CPD provides for the following four main elements:

- a system of harmonised technical specifications
- an agreed system of attestation of conformity for each product family
- a framework of notified bodies
- the CE marking of products

Note that the Directive does not aim to harmonise regulations. Member States and public and private sector procurers are free to set their own requirements on the performance of works and therefore products. What the CPD harmonises are the methods of test, the methods of declaration of product performance values, and the method of conformity assessment. Choice of required values for the chosen intended uses, is left to the regulators in each Member State.

4 A SYSTEM OF HARMONISED TECHNICAL SPECIFICATIONS

4.1 General

Technical specifications are harmonised European product standards (hENs) produced by CEN/CENELEC or European Technical Approvals (ETAs) produced by the European Organisation for Technical Approvals (EOTA).

The purpose of the technical specification for a product is to cover all the performance characteristics required by regulations in any Member State. In this way manufacturers can be sure that the methods of test and methods of declaration of results will be the same for any Member State, (although the values chosen by regulators may be different from one Member State to another).

The preferred route under the CPD is for harmonised standards to be written wherever possible. But if standards cannot be produced or foreseen within a reasonable period of time, or if product deviates substantially from a standard, than an ETA may be written.

ETAs may be written according to Guidelines (i.e. ETAGs) if several manufacturers of a particular product in several countries express an interest. If few manufacturers in only one or two countries express an interest, then ETAs may be issued without guidelines. These are called ‘Article 9.2 ETAs’. ETAs have a validity period of 5 years.

4.2 Annex ZA

European product standards often address characteristics which are not regulated in any Member State, but which have been included for commercial reasons. Because of this, all harmonised product standards under the CPD include an Informative Annex (termed Annex ZA) the first part of which (ZA.1) lists the regulated requirements and the clauses in the standard in which they are addressed. Some of these clauses may in turn refer to separate supporting standards which as test standards.

In this way Annex ZA.1 in the harmonised standard becomes a checklist for CE marking from which the manufacturer can see all the possible requirements of his product and how they can be met.

The parts of the standard which are not required by regulations are termed the voluntary or non-harmonised parts of the standard. These are not included in Annex ZA.1.

For an ETAG, its Chapter 4 services the function of Annex ZA.1 in a harmonised standard.

5 AN AGREED SYSTEM OF ATTESTATION AND CONFORMITY FOR EACH PRODUCT FAMILY

5.1 General

The attestation system is the term applied to the degree of involvement of third parties in assessing the conformity of the product according to the relevant technical specification(s). At present a significant barrier to trade arises from the different attestation levels required by Member States for the same product. Hence these requirements are also ‘harmonised’ under the Directive. For each product family the attestation system has been decided collectively by the Member States and the Commission on the basis of the implications for health and safety of the product, and on the particular nature and production process for the product itself.

5.2 Attestation systems and tasks

Six systems of attestation are used under the CPD as follows:

System	1+	Product conformity certification with audit testing
	1	Product conformity without audit testing
	2+	Factory production control (fpc) certification with continuous surveillance
	2	Factory production control (fpc) certification without surveillance
	3	Initial type testing

4 Manufacturer's tasks only

The tasks for the manufacturer and for the attestation body are summarised in Table 1.

Note that for all systems, including the least onerous (system 4), the manufacturer is required to have a fully recorded fpc system. The criteria for this should be included in the technical specification.

The attestation procedures for a product are set out in the relevant technical specification. For standards these appear in Annex ZA.2 for ETAGs in Chapter 8.

Table 1. Tasks for the manufacturer and for the attestation body.

Attestation tasks under the CPD						
Conformity attestation Commission numbering system	1+	1	2+	2	3	4
Tasks for the manufacturer						
Factory production control	✓	✓	✓	✓	✓	✓
Further testing of samples taken at factory according to prescribed test plan	✓	✓	✓			
Initial type testing			✓	✓		✓
Tasks for the notified body						
Initial type testing	✓	✓			✓	
Certification of FPC	✓	✓	✓	✓		
Surveillance of FPC	✓	✓	✓			
Audit testing of samples	✓					
✓ = task required						

5.3 Manufacturer's declaration of conformity and technical file

Once a manufacturer has had all the appropriate attestation tasks carried out for his product he is required to complete a 'Declaration of conformity' which is kept with his technical file concerning the product. This may by supported by a certificate of product conformity, fpc certificate, test laboratory reports or certificates, and/or own test results, depending on the attestation system required.

An outline of the manufacturer's declaration of conformity and for the certificate of product conformity (if relevant), is included usually in Annex ZA.2 of the product standard or Chapter 8 of the ETAG.

6 A FRAMEWORK OF NOTIFIED BODIES

6.1 Attestation bodies

Notified attestation bodies are the product conformity certification bodies, fpc certification bodies, inspection bodies (in some countries) and test laboratories who are competent to carry out the attestation tasks described in the previous section. Such bodies are first approved by their respective Member States to carry out certain designated tasks, and then notified to the Commission and other Member States. Hence they are variously call ‘approved bodies’, ‘designated bodies’, or ‘notified bodies’ or sometimes ‘Article 18 bodies’ after the relevant clause in the Directive. They will be referred to as ‘notified bodies’ in the remainder of this paper.

Once a harmonised technical specification (eg. CEN material product standard) is available for his product, a manufacturer will require to use a notified body to obtain CE mark for his product. He will be able to approach any notified body in the European Economic Area for assessment according to the appropriate attestation procedure.

6.2 ETA approved bodies

These are organisations designated by their respective Member States as competent to assess products and on this basis to issue European Technical Approvals. Just as for notified bodies described in Section 6.1, ETA approved bodies are notified to the Commission and other Member States.

Note that the process of issuing the ETA in the first instance is a separate process from the subsequent attestation procedures (if any). Hence, once an ETA has been issued for a product, the manufacturer is free to choose another body to carry out the attestation procedures.

It has become accepted terminology to refer to the bodies described in this section as ‘ETA approved bodies’, as distinct from ‘notified bodies’, which applies to attestation bodies as described in Section 6.1 above.

7 CE MARKING OF PRODUCTS

7.1 General

CE marking is a ‘passport’ enabling a product to be legally placed on the market in any Member State. However, as explained below, this does not necessarily mean that the product will be suitable for all end uses in all Member States.

7.2 Quality marks

CE marking is not a quality mark. It simply shows that the product addresses the regulatory requirements. Hence, quality marks are allowed to appear alongside the CE marking, provided their purpose cannot be confused.

8 COMPLIANCE WITH THE STRUCTURAL EUROCODES AND THE CONSTRUCTION PRODUCTS DIRECTIVE

Compliance with the structural Eurocodes will satisfy the Essential Requirement of the CPD in respect to 'Mechanical Resistance and Stability', and part of 'Safety in case of Fire'.

8.1 The essential requirements

The essential requirements apply to construction works, not to construction products as such, but they will influence the technical characteristics of these products.

Thus, construction products must be suitable for construction works which, as a whole and in their separate parts, are fit for their intended use, account being taken of economy, and which satisfy the essential requirements where the works are subject to regulations containing such requirements.

The essential requirements relate to:

- mechanical resistance and stability
- safety in case of fire
- hygiene, health and environment
- safety in use
- protection again noise
- energy economy and heat retention

These requirements must, subject to normal maintenance, be satisfied for an economically reasonable working life.

The essential requirements may give rise to the establishment of classes of a construction product corresponding to different performance levels, to take account of possible differences in geographical or climatic conditions or in ways of life as well as different levels of protection that may prevail at national, regional or local level. EU member states may require performance levels to be observed in their territory only within the classification thus established.

The structural Eurocodes relate to the essential requirement for 'Mechanical resistance and Stability', which states:

"The construction works must be designed and built in such a way that the loadings that are liable to act on it during its construction and use will not lead to any of the following:

a) collapse of the whole or part of the work;
b) major deformations to an inadmissible degree;
c) damage to other parts of the works or to fittings or installed equipment as a result of major deformation of the load bearing construction;
d) damage by an event to an extent disproportionate to the original cause."

The structural Eurocodes also relate to part of the essential requirements for safety in case of fire. The essential requirements are given concrete (i.e. quantitative) form in interpretative documents, which will create the necessary links between those requirements products (Fig.1) Interpretative Documents for each of the six essential requirements are available. ID 1 refers to 'Mechanical Resistance and Stability'. See also Chapter 2.

8.2 Methods of satisfying the Essential Requirements

In practice a product is fit for the intended use when it permits the works in which it is incorporated to satisfy the applicable essential requirements; a product is presumed to be fit for its intended use if it bears the EC marking (Fig 2), which declares the conformity of the product to technical specifications (Fig.1). These specifications comprise:

- Harmonized standards (Article 7 of the CPD) established by the CEN on the basis of mandates

- European technical approval (Article 8 of the CPD). Favourable technical assessment of the fitness for use of a product for an intended use, based on the fulfilment of the essential requirements for construction works for which the product is used.

Article 4 of the CPD allows referencing to National Standards but only where harmonized European specifications do not exist.

Some products for which CE marking is required *(see CE marking under the Construction Products Directive)* will involve structural design in arriving at the properties of the products to be claimed as meeting the technical specifications. A whole section of Guidance Paper L deals with the use of EN Eurocodes in technical specifications for structural products. A distinction is drawn between the cases where properties are obtained by:

(a) testing, and
(b) calculation.

For both methods it is likely that classes will need to be set up to allow for the inevitable differences in Nationally Determined Parameters from country to country.

When the properties are to be derived by testing, the technical specifications need to take into account the assumptions in design according to the EN Eurocode, particularly with regard to characteristic values and, after allowing for countries to set their own safety levels, design values.

When the properties are to be obtained from calculations according to EN Eurocodes, three methods are foreseen:

- **Method 1:** Indication of geometrical data of the component and of properties of the materials and constituent products used.

- **Method 2:** Determination of properties by means of the EN Eurocode (with the results expressed as characteristic values or design values)

- **Method 3:** Reference to design documents of the works or client's order.

Method 1: For this, the information on the geometrical data and properties of materials used, enable the structural component to be designed, using an EN Eurocode, for verifying its adequacy in works.

Method 2: This is the prime method that uses the EN Eurocodes to determine the mechanical resistance and resistance to fire of a structural product. When a relevant Eurocode is available, with NDPs for those countries in which the product is to be sold, the design must be based on the Code and NDPs in the National Annex to the Code. When a Eurocode is not available, then the technical specifications are permitted to include their own method of design, which, nevertheless, is required to be approved by CEN. (In practice, this route is unlikely to be practicable).
The level of safety is set by countries through their NDPs through the National Annexes of the Eurocodes; this means that design values for a structural component will vary from country to country. Characteristic values should not involve the use of the safety factors in a set of NDPs, but more parameters are likely to be varied from country to country, through NDPs, than just the safety factors. The only way in which these variations can be overcome is by using classes, each one covering a unique set of NDPs.

Method 3: For a structural product produced in accordance with a design, provided on behalf of the client, the manufacturer needs only to make reference to the design documents for the works.

REFERENCES

[1] Guidance Paper L (concerning the Construction Products Directive – 89/106/EEC) – Application and Use of Eurocodes. Director General Enterprises – European Commission.

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CHAPTER III: INTERPRETATIVE DOCUMENT 1– MECHANICAL RESISTANCE AND STABILITY

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Summary

The structural Eurocodes are primarily related to Essential Requirement 1 - Mechanical Resistance and Stability. The purpose and scope of the IRs are discussed and defined. The classes and levels of performance required and the basic terms are defined. The verification procedures are given. The ETA is described with timber given as an example. Where relevant EN 1990 is cross-referenced.

1 INTRODUCTION

The Structural Eurocodes, as already explained in Chapter I, are recognised by Member States of the European Economic Area (EEA) to serve as

- a framework for drawing up harmonised technical specifications for construction products; and
- a means of demonstrating compliance of building and civil engineering works with National requirements for regulated works and with **the essential requirements of the Construction Products Directive (CPD).** Council Directive 89/106/EEC: and
- a basis for specifying contracts for construction works and related engineering services.

Note: Chapter II gives the six Essential Requirements, as listed in CPD. The Eurocodes are relevant to essential requirements 1 and 2:

No.1 Mechanical resistance and stability

No.2 Safety in case of fire

This Chapter of the Handbook discusses Essential Requirement 1 - Mechanical Resistance and Stability.

2 INTERPRETATIVE DOCUMENTS TO THE EVENTUAL REQUIREMENTS

2.1 Purpose of the Interpretative Documents

Article 3 of the Construction Products Directive stipulates that the purpose of the Interpretative Documents is to give concrete form to the Essential Requirements for the creation of the necessary links between the Essential Requirements set out in Annex I to the Directive (see 2.3) and the mandates for the preparation of harmonized standards and guidelines for European technical approvals or the recognition of other technical specifications within the meaning of the Directive.

2.2 Purpose and scope of ER1 : Mechanical Resistance and Stability

The Interpretative Document for Mechanical Resistance and Stability deals with the aspects of the works where 'Mechanical resistance and stability' may be concerned. It identifies products or product families and characteristics relating to their satisfactory performance.

For each intended use of the product, mandates indicate appropriate further detail specifying which of those characteristics shall be dealt with in the harmonized specifications, using a step-by-step procedure with CEN/Cenelec/EOTA, which will allow the product characteristics to be modified or complemented, if necessary.

2.3 Definition of ER1 : Mechanical Resistance and Stability

Annex I to the Directive gives the following definition of the Essential Requirement which is applicable when and where the works are subject to regulations containing such a requirement:

'The construction works must be designed and built in such a way that the loadings that are liable to act on it during its construction and use will not lead to any of the following:

(a) collapse of the whole or part of the works;

(b) major deformations to an inadmissible degree;

(c) damage to other parts of the works or to fittings or installed equipment as a result of major deformation of the load-bearing construction;

(d) damage by an event to an extent disproportionate to the original cause '

These requirements above form the basic principles for the Eurocodes and are comprehensively described in Clause 2.1 of EN 1990 [1]

2.4 Effects of Essential Requirements on Member States

In accordance with the Council Resolution (New Approach) the Essential Requirement for Mechanical Resistance and Stability is intended not to reduce the existing and justified levels of protection for works in the Member States.

3 LEVELS OR CLASSES FOR ESSENTIAL REQUIREMENTS AND FOR RELATED PRODUCT PERFORMANCES

Where differences specified in the CPD are identified and justified in conformity with Community law, classes for Essential Requirements and for related product performances may be necessary. The purpose of such classes is to achieve the free circulation and free use of construction products. The Nationally Determined Parameter (NDPs) in the Eurocode system are a prime example and different levels.

4 MEANING OF THE GENERAL TERMS USED IN THE INTERPRETATIVE DOCUMENTS

4.1 Definitions in EN 1990

Although some of these terms have been defined generally slightly differently in EN 1990 they are re-produced here as they are key to the ID.

4.2 General Terms

4.2.1 Construction Works

‘Construction works’ means everything that is constructed or results from construction operations and is fixed to the ground. This term covers both *buildings* and *civil engineering* works. In the Interpretative Documents ‘construction works’ are also referred to as the ‘works’. Construction works include for example: dwellings; industrial, commercial, office, health, educational, recreational and agricultural buildings; bridges; roads and highways; railways; pipe networks; stadiums; swimming pools; wharfs; platforms; docks; locks; channels; dams; towers; tanks; tunnels; etc.

4.2.2 Construction products

(1) This term refers to products which are produced for incorporation in a permanent manner in the works and placed as such on the market. The terms ‘construction products’ or ‘products’ where used in the Interpretative Documents, include materials, elements and components (single or in a kit) of prefabricated systems or installations that enable the works to meet the Essential Requirements.

(2) Incorporation of a product in a permanent manner in the works means:

- that its removal reduces the performance capabilities of the works; and
- that the dismantling or the replacement of the product are operations which involve construction activities.

4.2.3 Normal Maintenance

(1) Maintenance is a set of preventative and other measures which are applied to the works in order to fulfil all its functions during its working life. These measures include cleaning, servicing, repainting, repairing, replacing parts of the works where needed, etc.

(2) Normal maintenance generally includes inspections and occurs at a time when the costs of the intervention which has to be made are not disproportionate to the value of the part of the works concerned, consequential costs being taken into account.

4.2.4 Intended Use

The intended use of a product refers to the role(s) that the product is intended to play in the fulfilment of the Essential Requirements.

4.2.5 Economically reasonable working life

(1) The working life is the period of time during which the performance of the works will be maintained at a level compatible with the fulfilment of the Essential Requirements.

(2) An economically reasonable working life presumes that all relevant aspects are taken into account, such as:

- costs of design, construction and use;
- costs arising from hindrance of use;
- risks and consequences of failure of the works during its working life and costs of insurance covering these risks;
- planned partial renewal;
- costs of inspections, maintenance, care and repair;
- costs of operation and administration;
- disposal;
- environmental aspects.

4.2.6 Actions

Actions which may affect the compliance of the works with the Essential Requirements are brought about by agents acting on the works or parts of the works. Such agents include mechanical, chemical, biological, thermal and electro-magnetic agents.

4.2.7 Performance

Performance is a quantitative expression (value, grade, class and level) of the behaviour of a works, part of the works or product, for an action to which it is subject or which it generates under the intended service conditions (for the works or parts of works) or intended use conditions (for products).

5 EN 1990 and ER1 : MECHANICAL RESISTANCE AND STABILITY

The requirements, principles and verification procedures in EN 1990 very closely follow and also compliment those for verification to ER 1 given in Sections 6, 7 etc of this Chapter.

6 EXPLANATION OF THE ESSENTIAL REQUIREMENT ‘MECHANICAL RESISTANCE AND STABILITY’

6.1 General

As for the definitions in EN 1990, for the meaning of these terms given in 6.2.1 to 6.25, account was taken of the International Standard ISO 8930 dated 15.12.1987.

6.2 Meanings of terms used in the text of the Essential Requirement, ‘Mechanical Resistance and Stability’:

6.2.1 Load Bearing Construction

Organized assembly of connected parts designed to provide mechanical resistance and stability to the works. The Interpretative Document, ‘load-bearing construction’ is referred to as *‘the structure’*.

6.2.2 Loadings that are liable to act on the works

Actions and other influences which may cause stress, deformations or degradation in the works during their construction and use. In the Interpretative Document, ‘actions and other influences’ are referred to as *‘actions’*.

6.2.3 Collapse

Various forms of failure of the structure as described in section 7.4.1.

6.2.4 Inadmissible deformation

Deformation or cracking of the works or part of the works which invalidates the assumptions made for the determination of the stability, the mechanical resistance or the serviceability of the works or parts of it, or causes significant reduction in the durability of the works.

6.2.5 Damage by an event to an extent disproportionate to the original cause

This means large damage of the works relative to the original cause (by events like explosions, impact, overload or consequence of human errors) which could have been avoided or limited without unacceptable difficulties or costs.

6.3 Other Specific Terms

Other specific terms given in particular in Section 7 are defined or explained where they occur in the text.

7 BASIS FOR VERIFICATION OF THE SATISFACTION OF THE ESSENTIAL REQUIREMENT 'MECHANICAL RESISTANCE AND STABILITY'

7.1 General

(1) This section identifies basic principles prevailing in Member States for the verification of the satisfaction of the Essential Requirement 'Mechanical resistance and stability'. These principles are currently complied with when and where the works are subject to regulations containing this Essential Requirement (e.g. A1 to A3 of the Building Representation for England and Wales [2]). Section 7 provides guidance on how to meet this Essential Requirement by compliance with the technical specifications referred to in CPD.

(2) The essential Requirement for Mechanical Resistance and Stability, as far as applicable, is satisfied with acceptable probability during an economically reasonable working life of the works.

(3) The satisfaction of the Essential Requirement is assured by a number of interrelated measures concerned in particular with:

- the planning and design of the works, the execution of the works and necessary maintenance;
- the properties, performance and use of the construction products.

(4) It is up to the Member States, when and where they feel it necessary, to take measures concerning the supervision of planning, design and execution of the works, and concerning the qualifications of parties and persons involved. Where this supervision and this control of qualifications are directly connected with the characteristics of products, the relevant provisions shall be laid down in the context of the mandate for the preparation of the standards and guidelines for European technical approval related to the products concerned.

Note: Annex B of EN 1990 comprehensively covers this article.

7.2 Actions

(1) Actions and other influences which may cause stress, deformations or degradation in the works during their construction and use. In the Interpretative Document, 'actions and other influences' are referred to as *'actions'*.

(2) When considering the satisfaction of the Essential Requirement, a distinction may be made between the following types of actions:

- Permanent actions: permanent actions due to gravity; actions of soil and water pressure; deformations imposed during construction, etc.
- Variable actions: imposed loads on floors, roofs or other parts of the works (excluding wind and snow); snow and ice loads; wind loads (static and dynamic); water and wave loads; thermal actions, frost; loads in silos and tanks; traffic loads on bridges and pavements; actions induced by cranes; dynamic actions from machinery; construction loads; etc.
- Accidental actions: impact; explosions; seismic actions; actions due to fire; etc.

7.3 Verification of the satisfaction of the Essential Requirements

(1) Verifications prevailing in Member States are based on the limit state concept described in section 7.4 using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. According to the Interpretative Document this implies that models are sufficiently precise to predict the behaviour of the structure and normally take into account the minimum standard of workmanship likely to be achieved, and the reliability of the information on which the design is based, and the assumptions made concerning maintenance.

(2) Testing is permitted where calculation methods are not applicable or appropriate.

(3) Potential damage of the works by an event to an extent disproportionate to the original cause See 2.3(d) may be limited or avoided by an appropriate choice of one or more of the following measures:
- avoiding, eliminating or reducing the hazards to which the structure may be exposed;
- selecting a structural form which has low sensitivity to the hazard considered,
- selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part the structure, or the occurrence of acceptable localised damage;
- avoiding as far as possible structural system that can collapse without warning;
- tying structural members together.

7.4 Methods for verifying mechanical resistance and stability of works

(NB. 7.4 is very similar to EN 1990 Section 3)

(1) Limit states are states beyond which the performance requirements are no longer satisfied. Limit states may relate to persistent situations during the working life of the works or to transient situations during the execution of the works stage of construction and/or assembling or repair), or to unintended uses or accidents. In general, distinction is made between ultimate limit states and serviceability limit states.

(2) Ultimate limit states are those associated with the various forms of structural failure or states close to structural failure, which for practical purposes are also considered as ultimate limit states.

(3) Ultimate limit states which may require consideration include:
- loss of equilibrium of the structure or any part of it, considered as a rigid body;
  Note: EQU in EN 1990

- failure by excessive deformation or settlement, transformation into a mechanism, rupture, or loss of stability of the structure or any part of it including supports and foundations.
  Note: STR or GEO in EN 1990

(4) Serviceability limit states correspond to states beyond which specified criteria for the structure related to its use or function are not longer met.

(5) Serviceability limit states which may require consideration are for example:

- deformation or deflections which cause anxiety or hinder the effective use of the works or cause unacceptable damage to finishes or non-structural elements;
- vibrations which cause discomfort to people or damage to the works or its contents, or which limit its functional effectiveness;
- detrimental cracking

8 TECHNICAL SPECIFICATIONS AND GUIDELINES FOR EUROPEAN TECHNICAL APPROVAL

8.1 General

(1) 'Technical specifications' means those referred to the CPD under 'Guidelines for European Technical approval' of a product or family of products means those referred to in the Directive

(2) The ID for ER1 makes a general distinction between category A and category B standards.

- *Category A*: These are standards, which concern the design and execution of buildings and civil engineering works and their parts, or particular aspects thereof, with a view to the fulfilment of the Essential Requirements as set out in the Directive. Category A standards should be taken into consideration within the scope of the Directive as far as the differences in laws, regulations and administrative provisions of Member States prevent the development of harmonized product standards. (The Structural Eurocodes are *de-facto*, Category A standards).

- *Category B:* These are technical specifications and guidelines for European technical approval which exclusively concern construction products subject to an attestation of conformity and marking according to the Directive. They concern requirements with regard to performance and/or other properties, including durability, of those characteristics that may influence the fulfilment of the Essential Requirements, testing and compliance criteria of a product.
  Category B standards that concern a family of products, or several families of products, are of a different character and are called horizontal (Category Bh) standards.

(3) Very importantly the ID for ER1 state the assumptions made in Category A standards on the one hand and those made in Category B specifications on the other shall be compatible with each other.

(4) It is a requirement that Category B technical approval shall indicate the intended use(s) of the respective products.

8.2 Provisions concerning works or parts of them

8.2.1 Basis for verification

In order to satisfy the Essential Requirement on mechanical resistance and stability, works will be verified using the structural Eurocodes and on the basis of procedures:

(a) complying with the provisions of Section 7 of this paper including the relevant limit states to be considered;

(b) making provisions with respect to the serviceability limit states; the owner of the works may lay down special or additional serviceability requirements depending on the function of the works.

8.2.2 Action

(1) The ranges of values for actions and other influences which need to be considered for the design, execution and use of the works which are currently given in the national regulations are also given in EN 1991 [3]. These also provide the representative values of actions and influences and specify the types of actions and values or classes to be considered for particular types of works, covered in EN 1990 and EN 1991

(2) As far as fatigue design is concerned, national regulations or Category A standards (i.e. the Eurocode) may consider rules for different working lives and rules for return periods. EN 1990 includes a fatigue (FAT) verification.

8.2.3 Partial safety factor format

Design rules in technical specifications and in guidelines for European technical approval may be based on the partial safety factor format using representative values for actions and the properties of materials as are the structural Eurocodes. In such a case account should be taken of the fact that levels of safety and serviceability depend on the quality assurance system (see also EN 1990 2.2, 2.5 and Annex B). The desired levels of safety and serviceability may be established by using probabilistic reliability methods. (Annexes B and C of EN 1990 gives comprehensive guidance).

8.2.4 Simplified rules

See ID for GR1 actions.

Technical specifications and guidelines for European technical approval may include simplified design rules based on the limit state concept, such as:

Case 1 - Verification by calculations:

(a) by simplifying the calculation for ultimate limit states and/or serviceability limit states, or

(b) by considering only serviceability limit states, where the ultimate limit states need not be considered explicitly;

Case 2 - Verification without calculations:

(a) by specifying particular detailing rules, or

(b) for simple works, by specifying particular provisions based on substantial experience.

Although ENV 1991-1 Basis of Design provided simplified rules for Ultimate Limit State verification, these have been removed from EN 1990 as it was not apparent that simplified rules gave higher levels of safety.

8.3 Provisions concerning products

8.3.1 Products and related characteristics which may be relevant to the Essential Requirement

(1) For the purpose of preparing the mandates for Category B standards under development and guidelines for European technical approval, the ID for ER1 gives a list given in the appendix and indicates products or families of products which may be placed on the market and which contribute to the ability of the works as a whole, or certain parts of the works, to satisfy the Essential Requirement. This list of products however, is not exhaustive.

(2) In this list, the characteristics relevant to the Essential Requirement, which need to be taken into account in the preparation of the mandates for European standards and guidelines for European technical approval, are shown against each product or family of products. They are also indicative of the characteristics to be considered in the mandates for those products that are not included in the list.

(3) For the characteristics listed in the appendix the following applies:

i) where mentioned, tolerances on sizes are to be considered in the specifications with reference to the overall design or execution need;

ii) where relevant (e.g. plastics), the range of temperature in which characteristics must be valid has to be expressed;

iii) even in cases where this is not specifically mentioned, a conventional age as well as rate of testing may be specified;

iv) durability (referred to the values of characteristics) is intended to mean the extent to which the values of the characteristics are maintained during the working life under the natural process of change of the characteristics, by excluding the effect of aggressive external actions.

v) the Interpretative Document applies to products where their performance affects the structural integrity of works (as a whole and in their separate parts).

(4) As an example a list for Timber products is given in Appendix A of this paper.

8.3.2 Performance of products and attestation of conformity of products

Guidance Paper L – Application and Use of the Eurocodes describes attaining performance and conformity using the Eurocodes.

9 WORKING LIFE, DURABILITY

9.1 Treatment of working life of construction works in relation to the Essential Requirement

(1) The ID leaves it to the Member States, when and where they feel it necessary, to take measures concerning the working life which can be considered reasonable for each type of works, or for some of them, or for parts of the works, in relation to the satisfaction of the Essential Requirements. Clause 2.3 provides guidance on this matter.

(2) Where provisions concerning the durability of works in relation to the Essential Requirement are connected with the characteristics of products, the mandates for the preparation of the European standards and guidelines for European technical approvals, related to these products, the ID asks that the works also cover durability aspects. EN 1990 covers this in Clause 2.4.

REFERENCES

[1] EN 1990 Eurocode: Basis of Structural Design. European Committee for Standardisation, 2002.

[2] H. Gulvanessian, J - A Calgaro and M. Holický: Designers' Handbook to EN 1990: Eurocode: Basis of Structural Design: Thomas Telford, London, 2002.

Appendix A to Chapter III

PRODUCT	**RELEVANT CHARACTERISTICS**
Solid Structural Timber Timber may be round or sawn, planed or otherwise processed and endjointed (glue) Timber may be untreated or impregnated to increase durability or fire resistance	Bending) Compression) Tension) parallel and perpendicular to grain Shear) Durability *(with respect to the values of the above characteristics and under the following actions)*: biological attack from wood-destroying fungi, insects and marine borers
Glued laminated timber Horizontally or vertically laminated, straight and curved, etc.	As for solid structural timber, and additionally bond integrity - glue-line shear strength - delamination resistance
Other glued timber products	Bond integrity as above Strength and stiffness under prescribed actions
Timber poles for transmission lines	Durability *(with respect to the values of the above characteristics and under the following actions)*: biological attack from wood-destroying fungi and insects

PRODUCT	RELEVANT CHARACTERISTICS
Wood-based boards e.g. plywood, particle board, fibreboard, oriented strand board, cement-bonded board	Dimensional stability under varying moisture conditions Strength and stiffness under difference moisture conditions in: Bending) compression) tension) in different directions; in plane and shear) perpendicular to the plane of the panel Durability *(with respect to the values of the above characteristics and under the following actions)*: biological attack from wood-destroying fungi and insects moisture Bond integrity - glue-line shear strength - effect of shrinkage - interaction with wood (acid damage)
Adhesives *(for in-situ use)* e.g. phenolic, aminoplastic and casein	Bond integrity - delaminating resistance - effect of shrinkage - interaction with wood (acid damage)
Mechanical and dowel-type fasteners e.g. nails, staple, dowels, bolts and screws	Durability *(with respect to the values of the above characteristics)* Tensile strength Bending strength Bending stiffness Joint strength in shear Durability *(with respect to the values of the above characteristics under the following actions)*: corrosion agents
Connector and punched metal plate fasteners e.g. nail plates, toothed plate connectors, split ring, shear plates	Joint strength in shear Stiffness in shear Behaviour under cyclic actions Durability *(with respect to the values of the above characteristics and under the following actions)*: corrosion agents

CHAPTER IV: GENERAL PRINCIPLES OF FIRE SAFETY – ID 2

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Summary

Safety in case of fire is one of the essential requirements imposed on construction products and works by the Council Directive. The concrete form of the essential requirement No 2 concerning fire safety is given in the Interpretative document ID 2 that provides bases for preparation of harmonized standards, technical specifications and guidelines for European technical approvals. General principles and application rules of fire design for construction products and works under fire design situations are given in the Eurocode EN 1991-1-2 and in Parts 1.2 of material oriented Eurocodes 1 to 6 and 9. Two possible methods are provided in EN 1991-1-2: the prescriptive approach using the nominal fire and the performance-based approach using physical and chemical parameters to generate thermal actions. Two Mathcad sheets for better understanding of the text are attached (TempTime.mcd and TpTime.mcd).

1 INTRODUCTION

1.1 Background documents

Requirements concerning safety in case of fire imposed on construction works and products by the Essential Requirement No 2 are given in the Council Directive [1]. Corresponding Interpretative Document ID 2 [2] provides detail explanation and extension of the Essential Requirement No 2. The documents [1,2] together with the Eurocode EN 1990 [3] were used as basic background documents for developing Part 1-2 of EN 1991 [4] which is a fundamental European document describing actions on structures exposed to fire. Additional information on fire actions may be found in the International Standard ISO 834 Part 1 [5], CIB publication [6] and recent State of the Art Report [7].

1.2 General principles

General principles of safety in case of fire are provided in the Council Directive [1] and corresponding Interpretative Document ID 2 [2]. In accordance with this essential requirement No. 2 [1] "the construction works must be designed and built in such a way that in the event of an outbreak of fire:

- the load bearing capacity of the construction works can be assumed for a specific period of time,
- the generation and spread of the fire and smoke within the works are limited,
- the spread of the fire to neighbouring construction works is limited,
- occupants can leave the works or be rescued by other means,
- the safety of rescue teams is taken into consideration."

Note that the above requirements may be mutually overlapping depending on particular conditions of the construction work. The form of the essential requirement No. 2 for preparation of harmonized standards, technical specifications and guidelines for European technical approvals is provided by the relevant Interpretative Document ID 2 [2]. It deals with the aspects of construction works where safety in case of fire may be concerned, identifies products and characteristics relating to their satisfactory performance in case of fire.

2 INTERPRETATIVE DOCUMENT ID 2

2.1 General description

The interpretative document ID 2 [2] Safety in case of fire is an extensive document covering the following main topics:

- explanation of the essential requirement
- basis for verification of the essential requirement
- technical specification and guidelines for European technical approval
- treatment of working life and durability

The Interpretative Document ID 2 contains also an Annex with definitions and terms. It should be mentioned that in accordance with the ID 2 the terms "construction works" denotes everything that is constructed or results from construction operations and is fixed to the ground. The "construction products" include materials, members and components (single or in a kit) or installations that enable the construction works to meet the Essential Requirements specified in [1].

It should be noted that the Interpretative Document ID 2 [2] (50 pages) is more extensive and contains more requirements than the Interpretative Documents ID 1 (18 pages). The follow-up part of Eurocode EN 1991 [4] (Part 1-2 of General Actions – Actions on structures exposed to fire) and relevant parts of other material oriented Eurocodes provide further technical details linked to ID 2. However, these documents are primarily focussed on structural design and do not cover all the possible aspects of the performance of engineering works and construction products exposed to fire that are considered in principles and application rules of ID 2.

This contribution describes the most important requirements of the ID 2 [2] and basic principles and application rules of Eurocode EN 1991-1-2 [4]. Taking into account the scope of Eurocodes, the following text is confined to requirements relevant to design of structures exposed to fire.

2.2 Fire Safety Strategy

Fire safety strategy described in ID 2 [2] is consistent with the five headings of the Essential Requirement as defined in the Council Directive [1] (see also 1.2). The five headings characterise mutually dependent requirements of fire safety that should lead to a reliable system of passive and active provisions for fire protection.

The development and magnitude of fire depends on various factors including the nature of fire, the thermal properties of materials used in the buildings, the air supply, the fire and smoke control systems and efficiency of the used fire protection system. Building contents (e.g. used floor coverings, furniture), which can also influence the rate at which fire and smoke develop is outside the scope of ID 2.

In order to prevent fire growing to an unacceptable size leading to a dangerous spread of smoke within the construction works, fire compartments must be properly designed and built (compartmentation is an important aspect of fire safety). A prerequisite for proper functioning of fire compartments is the overall stability of the main structure. Prevention of the spread of fire between neighbouring (separate) buildings is the next important item. Special attention should be paid to the connecting means as doors, stairs, escalators which need to have similar fire-safety properties. All the provisions for fire protection should be seen in close relationship with the intervention, fire-fighting and rescue operation of fire brigade. Fire safety of occupants can be improved by early detection of fire or by suppression of fire by an appropriate active protection system (e.g. sprinklers).

Important part of overall strategy is to minimize the occurrence of fires (fire protection) and to ensure effective system of fire detection and alarm system. These important parts of the strategy that concerns fire safety management are however outside the scope of the Interpretative document ID 2 [2].

2.3 Engineering approach to Fire Safety

Application of engineering principles to evaluating the required level of fire safety and to designing the necessary safety measures can be used in several ways:

- for determining of fire development and spreading inside or outside works
- for the assessment of fire and mechanical actions
- for evaluating the performance of construction products when exposed to fire
- for the evaluation of fire detection, activation and suppression
- for the evaluation and design of evacuation and rescue provisions

For application of an engineering approach, it is important to define characteristics of products and design procedures on an agreed and harmonized basis.

2.4 Thermal actions

Thermal actions consist of radiation, convection and conduction. The following levels of exposure are identified for thermal actions:

- small ignition source (e.g. match type)
- single burning items (e.g. burning equipment, stored products)
- fully developed fire (e.g. natural fire exposure, standard temperature/time curve)

Thermal actions depend on the type, intensity and duration of exposure and may be characterized by size of flame, level of radiation and level of convective heat transfer (combustion gas temperature and velocity) with presence or not of local flame impingements.

2.5 Fire resistance of structures

Evaluation of the performance of engineering works and construction products exposed to fire is an important part of the engineering approach to fire safety. The key element of this consideration is dependence of the air temperature on time duration of the thermal exposure. The two possibilities are prevailing in Member States: consideration of natural fire scenarios and consideration of conventional fire scenarios shortly described below.

Consideration of natural fire scenarios

Considering natural fire scenarios a calculation of the thermal action caused by fire (parametric temperature/time curve) is based on several parameters describing fire load and fire compartment including:

- the fire load (amount and burning rate)
- air supply to the fire
- geometry and size of the fire compartment
- thermal properties of the fire compartment
- effect of fire suppression installation (e.g. sprinklers)
- fire brigade and rescue team action

Engineering methods applied for analysing of thermal actions under natural fire scenarios may be sophisticated procedures and may require use of software products.

Consideration of conventional fire scenarios

Alternatively to natural fire scenarios a conventional model of the standard temperature/time curve defined in ISO 834 Part 1 [5] can be used

$$T = 345 \log_{10}(8\,t + 1) + 20 \qquad (1)$$

where T denotes the gas temperature in °C, t the duration of the thermal exposure in minutes. The standard temperature/time curve is a conventional model used to represent thermal action. The actual thermal attack associated with a natural fire can be higher or lower than that associated with the standard temperature/time curve.

For a more severe attack (higher rate of temperature rise) a harmonised hydrocarbon curve is used

$$T = 1080\,[(1 - 0{,}325 \exp(-0{,}167\,t) - 0{,}675 \exp(-2{,}5t)] + 20 \qquad (2)$$

In some circumstances when rate temperature increase is slower than that of the standard temperature/time curve (e.g. when fire protection coatings and other protecting systems are used), the smouldering curve may be used

$$T = 154\,t^{0{,}25} + 20 \qquad (3)$$

The standard and hydrocarbon temperature/time curves are included also in Eurocode EN 1991-1-2 [4] as nominal temperature/time curves. In addition to the curves described by equation (1) and (2), Eurocode [4] provides also the external temperature/time curve given as

$$T = 660\,(1 - 0{,}687\,e^{-0{,}32\,t} - 0{,}313\,e^{-3{,}8\,t}) + 20 \qquad (4)$$

The temperature/time curves described by equations (1), (2), (3) and (4) may lead to considerably different results (see attached MATHCAD sheet TempTime.mcd). That is why the consideration of natural fire scenarios is sometimes needed and used for a calculation of the thermal action caused by fire (parametric temperature/time curve) .

Note that for special extreme fire scenarios (e.g. tunnels, power stations) more severe conventional curves than those described by equations (1), (2) and (3) may be applied.

In general, when making a calculation of structural resistance to additional information (e.g. insulation, heat transfer, likelihood of cracks and holes) should be taken into account. Some of these data are difficult to estimate and often must be determined by undertaking the fire resistance tests. In some cases alternative calculation procedures are applied. Detailed guidance is provided in Handbook 5.

2.6 Verification of the satisfaction of the Essential Requirement

Several methods exist for verifying that the Essential Requirement is satisfied on the basis of the harmonised characteristics of the construction products. Three different approaches or their combination may be used in the national regulations:

– statement of a minimum performance requirement for works, which may be expressed in numerical or general way. In this case a link is required between the requirement for the works and the product characteristics,
– statement of a minimum performance of the products, e.g. fire resistance, performance of fire safety installations. In this approach, the statement shall be made by reference to the technical specifications,
– statement of the critical fire environment levels for people in or near construction works may. The harmonized terminology shall be used.

More detailed description of these approaches is provided in Handbook 5.

3 FIRE SAFETY IN EUROCODES

3.1 General

The Essential requirements [1] and Interpretative Document No 2 [2] represent the basic background documents for developing Eurocodes concerned with Fire Safety. In accordance with Section 4 of ID 2 [2], Eurocodes are classified to the top Category A of standards related to safety. The relevant parts of Eurocodes, directly related to the fire safety consist of:

- EN 1991-1-2 General actions – Actions on structures exposed to fire [4]
- Parts 1-2 of material oriented Eurocodes 2 to 6 and 9 that deal with passive fire protection.

The following text of this contribution is particularly linked to EN 1991-1-2 [4] (the Part 1-2 of Eurocode 1), which provides general guidance and actions for the structural design of buildings exposed to fire. In addition to Foreword and Section General, the final version of EN 1991-1-2 [4] contains the following main Sections:

- Section 2 Structural fire design procedure

- Section 3 Actions for temperature analysis (thermal actions)
- Section 4 Actions for structural analysis (mechanical actions)
- Annex A (informative) Parametric temperature-time curves
- Annex B (informative) Thermal actions for external members – simplified calculation method
- Annex C (informative) Localised fires
- Annex D (informative) Advanced fire models
- Annex E (informative) Fire load densities
- Annex F (informative) Equivalent time of fire exposure
- Annex G (informative) Configuration factor

The Parts 1-2 of material oriented Eurocodes 2 to 6 and 9 that deal with passive fire protection of construction works made of different materials represent extension of the basic document EN 1991-1-2 [4].

3.2 Basic approaches in EN 1991-1-2

General application of EN 1991, Part 1-2 is explained in the Foreword to [4]. As indicated in Figure 1 two possible methods are indicated: the prescriptive approach and the performance-based approach.

The prescriptive approach uses nominal fires (standard temperature/time curves) to generate thermal actions.

The performance-based approach, using fire safety engineering, refers to thermal actions based on physical and chemical parameters (parametric temperature/time curves). At the present time it is possible to undertake a procedure for determining adequate performance which incorporates some, if not all, of these parameters, and to demonstrate that the structure, or its members, will give adequate performance in a real building fire.

However where the procedure is based on a nominal fire (standard temperature/time curve), the required periods of fire resistance may be specified in such a way that the features and uncertainties considered by performance-based approach described above are taken into account (though not explicitly).

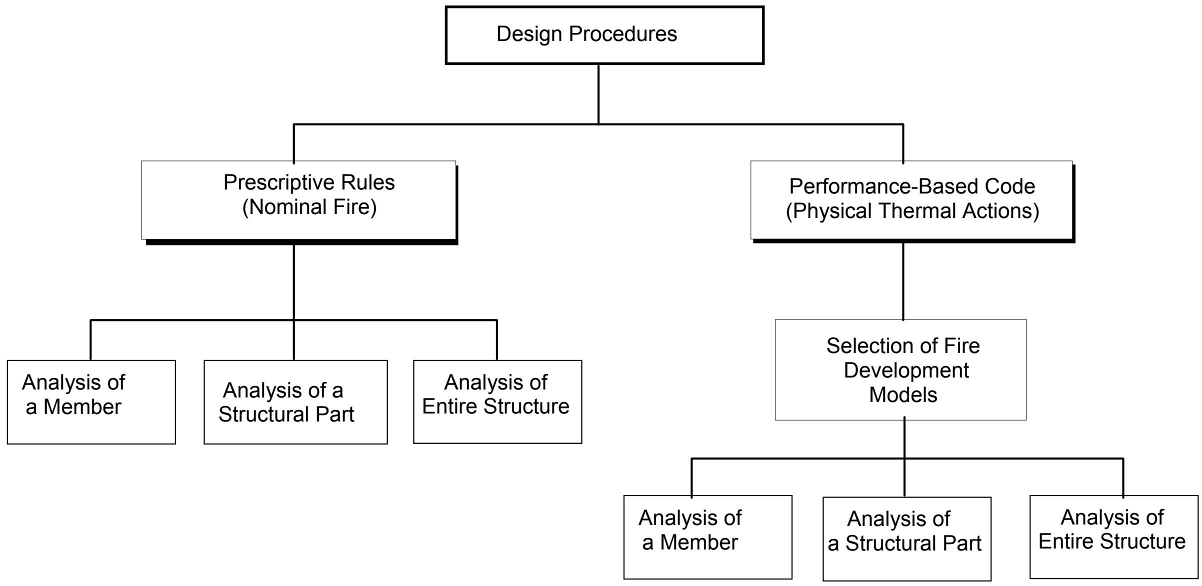

Figure 1. Design procedures.

The prescriptive approach and the performance-based approach indicated in Figure are described and applied in Handbook 5.

4 DESIGN PROCEDURE

4.1 Design fire scenarios

Structural fire design involves thermal actions as well as mechanical actions. As indicated in Figure 1, the thermal actions may be determined using either prescriptive rules (nominal fire) or physical based rules (parametric thermal curves). Actions due to fire are classified as accidental actions and should be combined with mechanical actions using combination rules provided in EN 1990 [3].

In general (see Section 2 of 1991-1-2 [4]) a structural fire design analysis should follow these steps:

- selection of the relevant design fire scenarios;
- determination of the corresponding design fires;
- calculation of temperature evolution within the structural members;
- calculation of the mechanical behaviour of the structure exposed to fire.

Selection of the relevant design fire scenarios and corresponding design fires should be done on the bases of general principles of risk analysis taking into account possible risks due to other accidental actions. The design fire should be usually applied only to one fire compartment. Post-fire situations after the structure cooled down need not to be considered in fire design.

Temperature analysis should take into account position of fire in relation to the structural member and separating walls. Depending on particular conditions the analysis may be based on a nominal temperature curve without cooling phase of full duration of the fire including the cooling phase.

4.2 Mechanical analysis

The analysis of mechanical behaviour of a member should consider the same duration as the temperature analysis. In accordance with Section 2 of Eurocode EN 1991-1-2 [4] three design requirements should be generally verified. The time requirement given by the inequality

$$t_{\mathrm{fi,d}} \geq t_{\mathrm{fi,requ}} \tag{5}$$

where $t_{\mathrm{fi,d}}$ denotes the design value of the fire resistance time and $t_{\mathrm{fi,requ}}$ the required fire resistance time.

Considering bearing capacity of a structural member the following condition is applied

$$R_{\mathrm{fi,d,t}} \geq E_{\mathrm{fi,d,t}} \tag{6}$$

where $R_{\mathrm{fi,d,t}}$ denotes the design value of resistance of the member in the fire situation at time t and $E_{\mathrm{fi,d,t}}$ denotes the design value of the load effect of the relevant actions in the fire situation at time t.

In addition to the design criteria expressed by equation (5) and (6) in some cases also the material temperature should be checked using the condition

$$\Theta_{\mathrm{d}} \leq \Theta_{\mathrm{cr,d}} \tag{7}$$

where Θ_{d} denotes the design value of material temperature and $\Theta_{\mathrm{cr,d}}$ the design value of the critical material temperature.

To verify design conditions (5) to (7) a number of computational tools and software products are available. Some of these tools are described in the Handbook 5 devoted to design of structures made of different materials that are exposed to fire.

Example

Consider an unprotected steel member having designed under the normal temperature for the load effect E_{d} such that $R_{\mathrm{d}} = E_{\mathrm{d}}$ (economic design). Assume that the load effect $E_{\mathrm{fi,d,t}}$ under accidental design situation due to fire is given by a time invariant value ηE_{d}, where η denotes the reduction factor following from accidental load combination (discussed in Handbook 5, an estimated value is $\eta = 0{,}5$). At the time t after the fire outbreak the resistance of the member (entering design condition (6)) may be expressed as $R_{\mathrm{fi,d,t}} = \kappa(t)R_{\mathrm{d}}$, where $\kappa(t)$ is the time variant reduction factor depending on the material temperature $\Theta(t)$ (assumed here to be equal to gas temperature $T(t)$). Then the design value of the fire resistance time $t_{\mathrm{fi,d}}$ entering the design condition (5) may be found from the relationship

$$\kappa(t_{\mathrm{fi,d}}) = \eta \tag{8}$$

The MATHCAD sheet TpResist.mcd attached to this contribution shows an application of equation (8) by numerical examples. It is shown that the fire resistance design time $t_{\mathrm{fi,d}}$ is strongly dependent on the assumed type of the nominal temperature/time curve.

5 Concluding remarks

5.1 Fire safety in Eurocodes

Safety in case of fire is covered by several parts of Eurocodes developed taking into account the Essential Requirement No 2 imposed on construction products and works by the Council Directive and the Interpretative Document ID 2. This Interpretative Document ID 2 (50 pages) is more extensive and contains more requirements than the Interpretative documents ID 1 (18 pages). The Eurocode EN 1991 (Part 1-2 of General Actions – Actions on structures exposed to fire) and relevant parts of other material oriented Eurocodes give further technical details linked to ID 2, however does not cover all the requirements provided in ID 2.

General principles and application rules of fire design of construction works and products are provided in the Eurocode EN 1991-1-2. In EN 1991-1-2 two possible methods are described: the prescriptive approach and the performance-based approach. The prescriptive approach uses nominal fires to generate thermal actions. The performance-based approach, using fire safety engineering, refers to thermal actions based on physical and chemical parameters.

Transformed Parts 1-2 of material oriented Eurocodes 2 to 6 and 9, which are linked to the Part 1-2 of EN 1991, deal with passive fire protection of construction works made of different materials (concrete, steel, composites, timber, masonry and aluminium).

5.2 General remarks

Safety in case of fire is becoming more and more important issue related to construction works and products. Engineering methods applied for verification of fire safety are rapidly developing and becoming extremely sophisticated procedures. A number of new computational techniques and software products are already available. In addition to the deterministic approach, probabilistic methods and methods of risk engineering are often applied for analysing safety of complicated construction works under fire scenarios and hazard situations. However, relevant input data describing actions and material properties are urgently needed.

To increase fire safety of construction works and products, various active and passive measures, including fire suppression installations (e.g. sprinklers), compartmentation and other fire protection measures to restrict development of fire and to facilitate fire brigade and rescue team appropriate actions, may be used. Practical experiences however indicate that these measures may be relatively expensive. If relevant active and passive measures affects the overall economy of the construction works significantly, then the methods of risk analysis and risk assessment may be effectively applied to provide important background materials enabling to make decision concerning the optimum combination of fire protection measures.

REFERENCES

[1] Council Directive 89/106/EEC on the approximation of laws, regulations and administrative provisions of the Memeber States relating to construction products. Official Journal of the European Communities, No I. 40, 1989.

[2] Interpretative Document, Essential Requirement No 2 "Safety in case of fire", Official Journal of the European Communities, No C 62/23, 1994.

[3] EN 1990 Basis of structural design. European Committee for Standardisation, CEN 250, Brussels, 2002.

[4] EN 1991-1-2 Actions on Structures. General Actions - Actions on structures exposed to fire. Basis of design. European Committee for Standardisation, CEN/TC 250/SC1, Brussels 1992.

[5] ISO 834 Part 1, Fire resistance tests - Elements of building construction, 1992.

[6] CIB Report, Publication 166. Action on Structures, Fire, September 1993.

[7] J.B. Schleich et al.: Fire engineering design for steel structures. State of the Art Report. International Iron and Steel Institute, Brussels, 1993.

ATTACHMENT

MATHCAD sheets illustrating the nominal temperature/time curves.

1 TempTime.mcd

The MATHCAD sheet enabling to compare and to determine temperatures given by the nominal temperature/time curves (standard, hydrocarbon, smouldering and external curves).

Definitions of the nominal curves given in ID2 [2] and EN 1991-1-2 [4] considered in the MATHCAD sheet TempTime.mcd.

The standard curve (ID 2 and EN 1991-1-2)

$T1(t) = 345 \log(8t + 1) + 20$

The hydrocarbon curve (ID 2 and EN 1991-1-2)

$T2(t) = 1080\, [(1 - 0{,}325 \exp(-0{,}167\, t) - 0{,}675 \exp(-2{,}5t)] + 20$

The smouldering curve (ID 2)

$T3(t) = 154\, t^{0,25} + 20$

The external curve (EN 1991-1-2)

$T4(t) = 660\, (1 - 0{,}687 \exp(-0{,}32\, t) - 0{,}313 \exp(-3{,}8\, t)) + 20$

Note that the curve T2(t) asymptotically approaches the temperature 1100 ^{o}C, T4(t) approaches the temperature 680 ^{o}C.

2 TpTime.mcd

The MATHCAD sheet enabling to calculate the fire resistance $R_{fi,d,t} = \kappa(t)R_d$ and the design value of the fire resistance time $t_{fi,d}$ assuming the nominal temperature/time curves (standard, hydrocarbon, smouldering and external curves).

It is shown that the fire resistance time $t_{fi,d}$ is strongly dependent on the assumed type of the nominal temperature/time curve.

Attachment 1. TempTime.mcd

ISO Temperature/Time Curves

1 Definition of the nominal temperature/time curves

Temperature T in°C, time t in minutes

The standard curve (ID 2 andEN 1991-1-2)

$T1(t) = 345 \log(8t +1)+20$

The hydrocarbon curve(ID 2 and EN 1991-1-2)

$T2(t) = 1080 [(1 - 0{,}325 \exp(-0{,}167 t) - 0{,}675 \exp(-2{,}5t)] + 20$

The smouldering curve(ID 2)

$T3(t) = 154\, t^{0{,}25} + 20$

The external curve (EN 1991-1-2)

$T4(t) = 660 (1 - 0{,}687 \exp(-0{,}32 t) - 0{,}313 \exp(-3{,}8 t)) + 20$

2 Graphical representation of the temperature/time curves

$T1(t) := 345 \cdot \log(8 \cdot t + 1) + 20$ $\quad T2(t) := 1080 \cdot (1 - 0.325 \cdot \exp(-0.167 \cdot t) - 0.675 \cdot \exp(-2.5 \cdot t)) + 20$

$T3(t) := 154 \cdot t^{0.25} + 20$ $\quad T4(t) := 660 \cdot (1 - 0.687 \cdot \exp(-0.32 \cdot t) - 0.313 \cdot \exp(-3.8 \cdot t)) + 20$

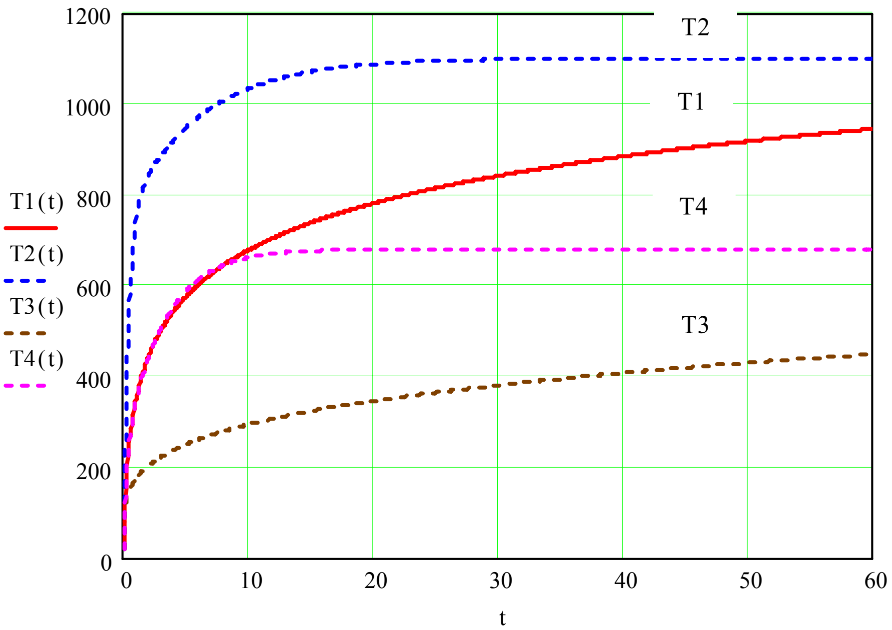

Figure 1. The nominal temperature/time curves (temperature T in°C, time t in minutes)

3 Temperatures determined assuming the nominal curves

Assuming t = 10 minutes $\quad T1(10) = 678.427 \quad T2(10) = 1.034 \times 10^3$

$T3(10) = 293.855 \quad T4(10) = 661.518$

Assuming t = 20 minutes $\quad T1(20) = 781.355 \quad T2(20) = 1.088 \times 10^3$

$T3(20) = 345.67 \quad T4(20) = 679.247$

Note that T2(t) asymptotically approaches the temperature 1080 °C, T4(t) approaches the temperature 680 °C.

Attachment 2. TpTime.mcd

The fire resistance for the nominal temperature/time curves

1 Definition of the nominal temperature/time curves

Temperature T in °C, time t in minutes

The standard ISO curve (ID 2 andEN 1991-1-2)

T1(t) = 345 log(8t +1)+20

The hydrocarbon curve(ID 2 and EN 1991-1-2)

T2(t) = 1080 [(1 - 0,325 exp (-0,167 t) - 0,675 exp (-2,5t)] + 20

The smouldering curve(ID 2)

T3(t) = 154 $t^{0,25}$ + 20

The external curve (EN 1991-1-2)

T4(t) = 660 (1 - 0,687 exp(-0,32 t) - 0,313 exp(-3,8 t)) + 20

2 The temperature/time curves

$t := 0, 0.5..\ 120$

$T1(t) := 345 \cdot \log(8 \cdot t + 1) + 20$ $\quad T2(t) := 1080 \cdot (1 - 0.325 \cdot \exp(-0.167 \cdot t) - 0.675 \cdot \exp(-2.5 \cdot t)) + 20$

$T3(t) := 154 \cdot t^{0.25} + 20$ $\quad T4(t) := 660 \cdot (1 - 0.687 \cdot \exp(-0.32 \cdot t) - 0.313 \cdot \exp(-3.8 \cdot t)) + 20$

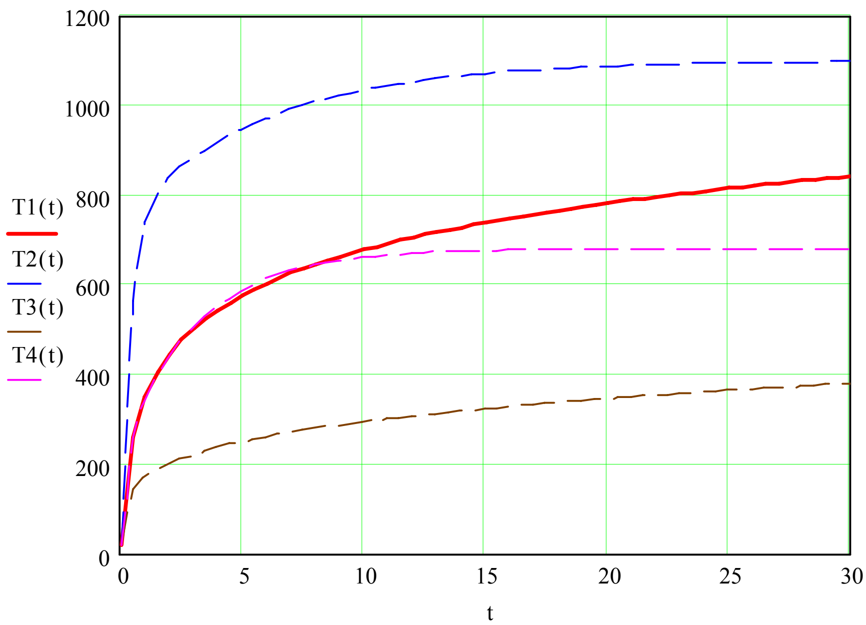

Figure 1. The nominal temperature/time curves (temperature T in °C, time t in minutes)

3 The fire resistance of structural steel

Variation of the fire resistance R(T) = κ(T) Rd with temperature T, an approximation of the data given in ENV 1993-1-2:

$$TT := (100\ \ 200\ \ 300\ \ 400\ \ 500\ \ 600\ \ 700\ \ 800\ \ 900\ \ 1000)^T$$

$$ky := (1\ \ 1\ \ 1\ \ 1\ \ 0.78\ \ 0.47\ \ 0.23\ \ 0.11\ \ 0.06\ \ 0.04)^T$$

Approximation curve for temperature range $T := 0, 10..\ 1000$

$$\kappa(T) := \left(1 + \exp\left(\frac{T - 482}{39}\right)\right)^{-0.26}$$

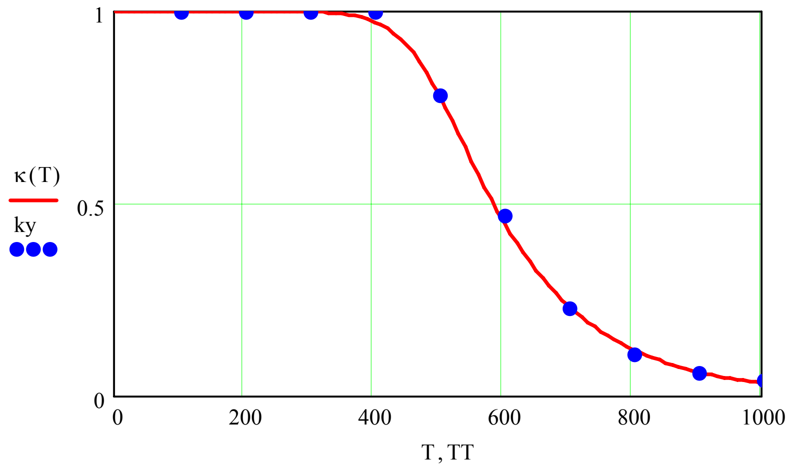

Figure 2. Variation of the reduction factor for the yield point with temperature T in °C according to ENV 1993-1-2 (points ky) and approximation function (full line κ(T)).

Considering four nominal temperature/time curves the reduction factor κ(T(t)) is given as

$$\kappa 1(t) := \left(1 + \exp\left(\frac{T1(t) - 482}{39}\right)\right)^{-0.26} \qquad \kappa 2(t) := \left(1 + \exp\left(\frac{T2(t) - 482}{39}\right)\right)^{-0.26} \qquad \kappa 1(15) = 0.181$$

$$\kappa 3(t) := \left(1 + \exp\left(\frac{T3(t) - 482}{39}\right)\right)^{-0.26} \qquad \kappa 4(t) := \left(1 + \exp\left(\frac{T4(t) - 482}{39}\right)\right)^{-0.26} \qquad \kappa 4(150) = 0.267$$

Reduction factor η of the load effect Ed, Efi,d,t= ηEd, an estimated value $\eta := 0.6$

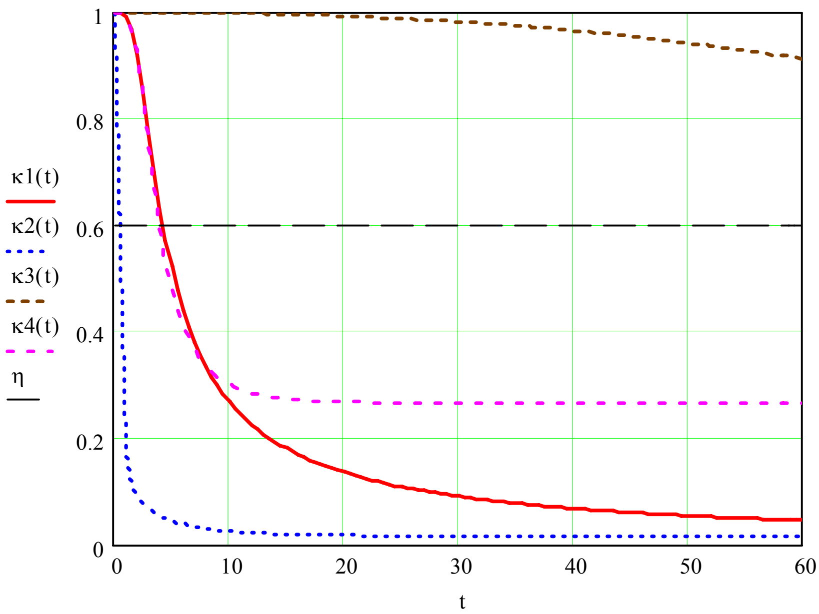

Figure 3. Variation of the fire resistance R(T) = κ(T(t)) Rd with time t (time t in minutes)

The fire resistance time given by intersection of the reduction factor η with the factor κ(T(t)) is strongly dependent on the assumed type of the nominal temperature/time curve. For $\eta = 0.6$ and the standard and external curve the fire resistance time is about 4 minutes. While the smouldering curve is less significant, t he hydrocarbon curve leads to a rapid decrease of the steel strength.

4 Notes to design of a steel member considering the standard ISO temperature/time curve

4.1 Action effect

One permanent load G and one variable load Q is considered only. Reduction of the required design resistance under normal situation (the minimum being γG Gk+ γQ Qk)) to the required design resistance under fire design situation (the minimum being Gk+ ψ Qk)) is given by factor η expressed in terms of the load ratio χ = Qk/(Gk+Qk) and combination factor ψ used for accidental fire situation as

$$\eta(\chi,\psi,\gamma G,\gamma Q) := \frac{1\cdot(1-\chi)+\psi\cdot\chi}{\gamma G\cdot(1-\chi)+\gamma Q\cdot\chi} \qquad \eta(0.4,0.5,1.35,1.5) = 0.567$$

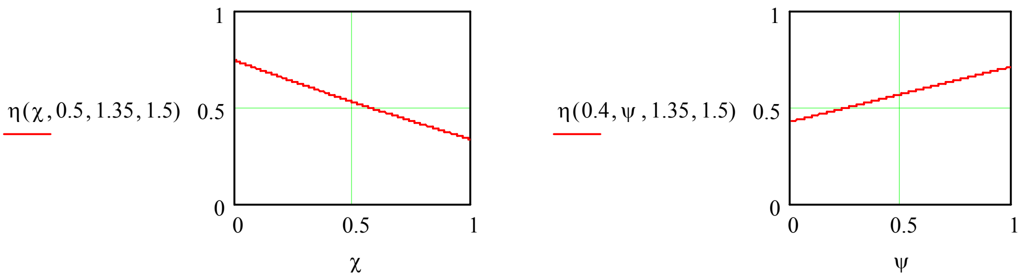

Figure 4. Variation of η with the load ratio χ and combination factor ψ for fire situation.

4.2 Fire resistance

Resistance decrease for the Standard ISO curve is considered:

For a required fire design time tr, for example $tr := 10$ the resistance decrease is $\kappa 1(tr) = 0.269$

The following "if" function shows whether the resistance under normal temperature is sufficient. It delivers either:

"1" indicating that the resistance is sufficient and fire design is not required, or
"0" indicating that the resistance is insufficient and fire design is required.

$$\text{if}(\kappa 1(tr) > \eta(0.4,0.5,1.35,1.5), 1, 0) = 0$$

If "0" is delivered then the normal resistance must be increased at least by the factor ω, given as

$$\omega(tr,\chi,\psi,\gamma G,\gamma Q) := \frac{\eta(\chi,\psi,\gamma G,\gamma Q)}{\kappa 1(tr)}$$

An example: $\chi := 0.4$ $\gamma G := 1.35$ $\gamma Q := 1.5$ Auxiliary range variable ttr for tr $ttr := 0, 1..30$

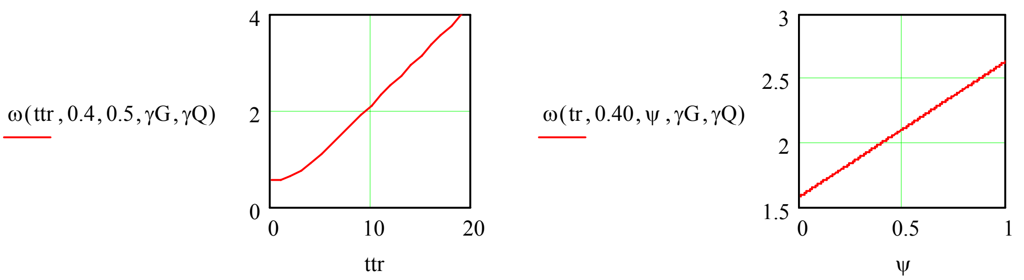

Figure 5. Variation of the required increase ω with the load ratio χ and combination factor ψ.

Thus if tr = 10 minutes and the factor considered in fire design situation $\psi = 0.5$, then the resistance under normal conditions should be increased by the factor

$$\omega(tr,0.4,0.5,\gamma G,\gamma Q) = 2.105$$

Note that the standard ISO temperature/time curve is considered. Similar procedure is however applicable for other nominal or natural temperature/time curves.

Chapter V - LIMIT STATES AND METHOD OF PARTIAL FACTORS

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Summary

This section addresses limit states and the so-called partial factor method. The fundamental requirements for the treatment that follows is the Eurocode - Basis of Structural Design, whose guidelines pertinent to the chapter topic are considered and explained. The first part of the chapter deals with some general principles, amongst which the definition of the limit states (i.e. when a structure reaches conditions where it is no longer able to perform one or more of the functions for which it was designed) and sets forth the fundamental distinction between ultimate limit states and serviceability limit states.

The remainder of the chapter is devoted to detailing the partial factor method, which enables performing an evaluation of structural performance in terms of both ultimate and serviceability limit states. The starting point for such analyses is represented by the characteristic values of the actions and the mechanical properties of the structural materials and members, which are represented via appropriate factors in order to account for the uncertainties associated with the characteristic values themselves, as well as with the geometric data and the process of physical-mathematical modelling.

1 INTRODUCTION

1.1 Background documents

Regulations EN 1990, Eurocode - Basis of Structural Design [1], plays a highly strategic role because it contains the principles and requisites for attaining safety, serviceability and durability, which form the bases of the nine sets of Structural Eurocodes.

The Eurocodes call for evaluating the reliability of a structure (in the widest sense of the term, including civil buildings, bridges, industrial buildings, and so forth) within the conceptual framework of Limit States by application of the partial factor method. Chapter 3 of EN 1990 sets forth the definition of limit state and provides a description of limit-state design principles, while the verification procedures by means of the method of partial factors are addressed in Chapter 6 of the EN 1990. The purpose of this Chapter is to expound upon the two aforementioned chapters of EN 1990 [1].

1.2 General principles

Assessments of the safety of civil engineering works must be conducted by examining all aspects of their behaviour and all the possibilities for failure or poor functioning that can be manifested. Analysing potentially critical situations is performed by identifying so-called "limit states".

A limit state is defined as a condition beyond which a structure (or one or more of its constituent members) is no longer able to satisfy the provisions for which it was designed. A fundamental distinction to be made is that between ultimate limit states and serviceability limit states.

Ultimate limit states are strictly related to bearing ability or, in any event, to the attainment of extreme conditions, and therefore refer to all those situations that can compromise people's safety, safety of the structure and in exceptional circumstances the potential contents (as underscored by the EN 1990, clause 3.3(1)P). More specifically, an ultimate limit state is to be considered to have been reached when:

- a structure has lost its equilibrium, either overall or in any one of its subcomponents, and therefore overturns as a rigid body;
- collapse of the bearing structure (or its constituent members) occurs through formation of mechanisms;
- buckling of structural members (either overall, or at a local level - buckling of compressed parts of straight sections).

Also to be included amongst the ultimate limit states are the loss of connection between structural members, the lack of ductility (essential for proper energy dissipation) in the face of seismic events and, finally, those extreme situations of ultimate failure reached because of fires, explosions or collisions.

Serviceability limit states are instead related to the criteria governing a construction's functionality, the requirements for durability and ordinary usage, user comfort and the structure's appearance (clause 3.4(1)P in EN 1990 [1]). Within the framework of serviceability limit-state verifications, three different aspects to be considered are distinguished: deformations, vibrations and damage (clause 3.4(3) [1]).

Excessive deformation of the structure (or individual members) may jeopardize the integrity of load-bearing members, such as partition walls and window frames) (especially if made of non-ductile materials) and the proper operation of precision equipment and instrumentation.

Vibrations may be induced by wave motion, wind effects or the actions of machinery, and have the effect of limiting functional efficiency and causing discomfort, unease, or even panic on the part of those using the construction.

The first consequence of any damage is economic in nature, because it involves the need to institute restoration operations on the damaged parts (with the direct costs of repair and possible indirect costs due to the temporary suspension of operations). Moreover, damage to the construction, even when localised, may provide a preferential route of attack for destructive chemical substances. The classic example in this regard is the formation of excessively wide cracks in concrete members, which expose the steel reinforcement to the corrosive actions, while at the same time facilitating carbonation phenomena in the concrete itself. The harsher are the prevailing environmental and atmospheric conditions, the more pronounced such effects will be (in this perspective, polluted industrial areas and marine environments represent the worst scenarios). Lastly, physical damage has psychological consequences on users: in fact, damage to a structure's aesthetic aspects - for example, the presence of cracks or rusted structural members - inevitability creates the impression that the work's static structure is in a precarious state and causes a sense of insecurity.

The phenomena of fatigue call for some considerations apart. Manifestations of fatigue are typical not only in machinery, *per se*, but also in bridges (both road and railway) and the gantry runways of industrial warehouses, as well as offshore platforms and wharfs, due to the continuous actions of wave motion. Although fatigue phenomena are correlated to the values of the typically frequent actions under normal operating conditions, they must be considered in relation to ultimate limit states because they cause the collapse of the structure or some of its parts.

According to the partial factor method, a structure is deemed reliable if no limit state considered to be relevant is exceeded when calculation models are applied using appropriate design values for the geometric data, the actions in question and the properties of structural materials and members (clause 3.5(2)P [1]). It therefore becomes necessary to identify design situations and critical *load cases* (3.5(6)P[1]). A *load case e*ncompasses a set of compatible provisions for actions imposed deformations that must be considered simultaneously. *Load cases* must also take into account structural imperfections. These may be evaluated in two distinct ways: via an equivalent geometric imperfection, which yields an initial displacement of the structure from its initial geometry, or in terms of equivalent forces. Figure 1 represents schematically such a situation considering a simple frame.

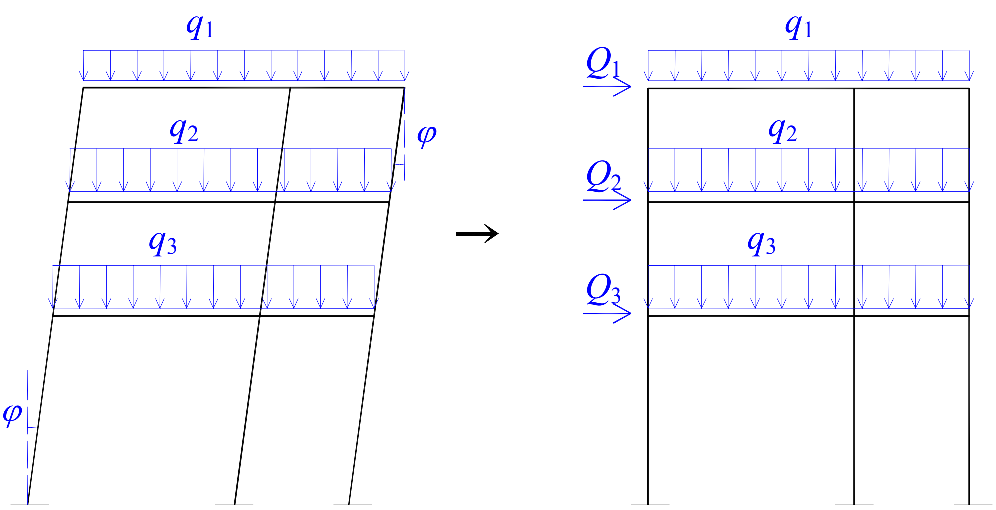


Figure 1. Evaluation of the imperfections of a frame structure.

The requirement given in clause 6.1(2) [1] underscores that "*actions that cannot occur simultaneously, for example due to physical reasons, should not be considered together in combination*". In this regard, however, apart from regulatory guidelines, the designer's judgment and good sense are important: for instance, it would be physically impossible for a snow load to be present on a heated surface, such as the highly thermally conductive metal roof a boiler room.

Thus, to sum up, the following fundamental elements should be identified:

- the various physical and mathematical models adopted for ultimate and serviceability limit-state verification;
- the design values of the quantities involved (actions, mechanical material properties, geometric data), defined by starting with the corresponding characteristic values (or other representative values), in combination with a set of partial factors (γ) and coefficients ψ.

2 THE METHOD'S LIMITATIONS

The requirements specify the applicability range of the method. The rules proposed by EN 1990 are limited to ultimate and serviceability limit-state verifications of structures subjected to static loads. Such static loads include, for instance, the action of the wind or vehicles passing over a bridge, which though they are inherently dynamic, can be referred to static conditions by considering the dynamic effects via suitable amplification factors.

3 DESIGN VALUES

3.1 Design values for actions

In general terms the design value F_d of an action F is expressed by the following relation

$$F_d = \gamma_f \cdot F_{rep} \tag{1}$$

where F_{rep} indicates the representative value of the action, and γ_f is a partial factor for the action, which provides for the possibility that the action's values may in fact present unfavourable variations from the representative values.

F_{rep} is calculated from the characteristic value F_k of the action, via expression

$$F_{rep} = \psi \cdot F_k \tag{2}$$

where ψ is a reduction factor equal or less than 1.

3.2 Design values of actions effects

The effects of an action represent the structural response, in terms of the stress characteristics (bending moment, shear, normal stress and torsion), as well as in terms of deformation (displacements, strains). Such response concerns both the structure as a whole, as well as the members and substructures making it up.

For a specific *load case*, the design values of the action effects, E_d , can be written in general form as

$$E_d = \gamma_{Sd} \cdot E\left\{\gamma_{f,i} \cdot F_{rep,i}; a_d\right\} \quad i \geq 1 \tag{3}$$

The partial factor γ_{Sd} accounts for the uncertainties involved in the modelling, including two distinct aspects: modelling the effects of the actions and, in some cases, modelling the actions themselves (clause 6.3.2(1) [1]). In general, a mathematical model represents a rational, quantitative formulation of a specific natural physical aspect to be analysed. The greater the number of quantities that the model takes into consideration and the more sophisticated and refined is its mathematical processing, the more realistic will be its description of the phenomenon under examination. However, although extremely sophisticated levels can be attained, mathematical representation of reality can never be exact, and discrepancies will always exist between the actual phenomenon and the predictions furnished by the model. Then, on the operational level, there is the practical need for simple, usable models. For this reason, it must always be borne in mind that modelling, by its very nature, involves some uncertainties.

Now, "a_d" indicates the design values of the geometric quantities. The subscript "i" stands to show that the evaluation must be extended to all the actions contributing to produce a given effect. Expression (3), by its very formulation, implies that the effects of the actions are not influenced by the characteristics or mechanical properties of the material, provisions for which are in fact absent from the expression.

Regulation EN 1990 also adopts a simplified form of (3), widely applied in designing the most commonly occurring structures

$$E_{\mathrm{d}} = E\left\{\gamma_{\mathrm{F},i} \cdot F_{\mathrm{rep},i}; a_{\mathrm{d}}\right\} \quad i \geq 1 \tag{4}$$

where coefficient $\gamma_{F,i}$ is:

$$\gamma_{\mathrm{F},i} = \gamma_{\mathrm{Sd}} \cdot \gamma_{\mathrm{f},i} \tag{5}$$

In evaluating a set of actions' effects, EN 1990, make a fundamental distinction between favourable permanent actions and unfavourable permanent ones. The terms "favourable" and "unfavourable" are meant to be interpreted in relation to the specific effect in question, and, particularly, in concomitance with the joint action of other variable loads. Applying the aforesaid distinction necessarily implies resorting to two separate values for the partial factors of the permanent actions: $\gamma_{\mathrm{G,sup}}$ and $\gamma_{\mathrm{G,inf}}$.

Example 1

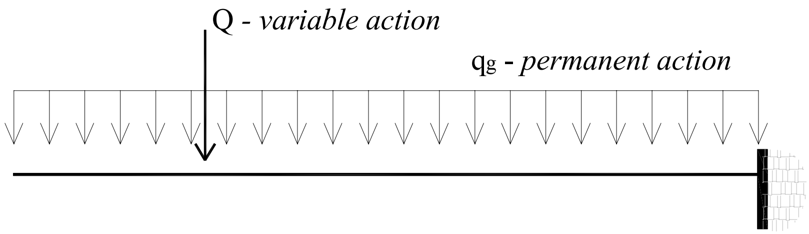


Figure 2. Fixed beam subjected to permanent loads and a variable action

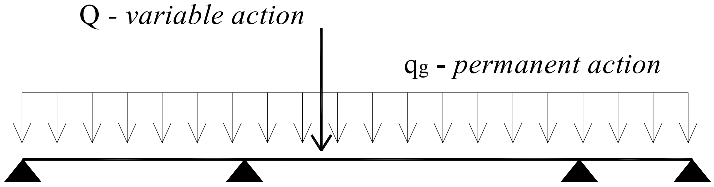


Figure 3. Continuous beam subjected to permanent loads and a variable action

Referring now to the simple situation illustrated in Figure 2, we wish to determine the effects on the cross sections of the cantilevered beam (shear, bending moment and deflection). The permanent load and variable action act in unison, so the permanent action must therefore be multiplied by $\gamma_{G,\mathrm{sup}}$.

Instead, regarding the continuous beam schematically shown in Figure 3, depending upon the characteristics of the sought-after stress and the section in question, the effects of the permanent load and variable action may be either unfavourable or favourable. Thus, the effects of the permanent action must be multiplied by $\gamma_{G,\text{sup}}$, in the first case, and by $\gamma_{G,\text{inf}}$, in the second. When considering unfavourable effects the variable action Q must be placed only on the appropriate span.

Further considerations are necessary when implementing non-linear analyses (i.e., analysis in which the adopted relation between the actions and the effects they produced is not linear). In the event there is a single predominant action F, two distinct cases may present, depending on whether the effects grow more or less than the causative action. If the effect of the action increases more than the action itself, then the partial factor must be applied to the action's characteristic value, as indicated in relation

$$E_{\text{d}} = E\left(\gamma_{\text{F}} \cdot F_{\text{k}}\right) \tag{6}$$

Otherwise, when the action's effect increases less than the action itself, then the partial factor must multiply the effect of the action

$$E_{\text{d}} = \gamma_{\text{F}} \cdot E\left(F_{\text{k}}\right) \tag{7}$$

These are surely simplified rules, but ones which provide a good approximation. With the exception of cable or membrane structures, most real constructions can be considered to fall within the first of the foregoing two cases.

3.3 Design values of structural materials and members properties

Eurocode EN 1990 provides design values for the resistance of materials. Alternatively, such design values can be inferred (as expressly indicated in a note to EN 1990) from empirical relations adopting properties that have been either measured physically or deduced from data on the product's chemical composition, or from prior experience or, finally, by using values furnished in suitable documents of verifiable validity.

Within the framework of limit-state design philosophy and the method of partial factors, the design value X_{d} of a mechanical property of a material or product is inferred from the characteristic value X_{k} (which in general corresponds to a fractal in the statistic distribution assumed by the particular specified property of the material). Such characteristic value is appropriately formulated in order to account for the effects of volume, scale, humidity and temperature (all incorporated into parameter η), as well as the possibility of unfavourable deviations of the values of the product properties from the characteristic ones, including a certain randomness inherent in η, defined the conversion factor in EN 1990 [1] (these last aspects are taken into account via introduction of the partial factor γ_{m}). In conclusion, then

$$X_{\text{d}} = \eta \cdot \frac{X_{\text{k}}}{\gamma_{\text{m}}} \tag{8}$$

3.4 Design values for geometric data

Design values for geometric data are explained in chapter VII, where background is given. Variations in the geometric dimensions of structural members are generally quite small, so much so as to enable considering them negligible. In this regard, one need only consider the extremely narrow tolerances deemed acceptable nowadays for the production of steel members. For this reason, the design values now accepted for geometric data correspond to the nominal values themselves (clause 6.3.4(1) [1]).

$$a_{\mathrm{d}} = a_{\mathrm{nom}} \tag{9}$$

In actual construction practice, however, situations can arise in which possible geometric variations may cause effects whose full significance is not anticipated in the stages of calculation: for instance, improper positioning of a structural member on its supports or imprecision in the application of the load (which can produce dangerous second-order P-Δ effects). In such circumstances necessary caution must be exerted during calculations, and the design values for geometric data can no longer be considered to correspond to the nominal ones (clause 6.3.4(2)P [1]), as provided for in expression

$$a_{\mathrm{d}} = a_{\mathrm{nom}} \pm \Delta a \tag{10}$$

In fact, Δa accounts not only for any possible deviations of the real data from the characteristic or nominal values, but also for the cumulative effect of deviations in various geometric data that may manifest themselves simultaneously. Clearly, the sign to be chosen for Δa must be that which yields the most demanding conditions under the given circumstances.

3.5 Design resistance

The following relation represents an entirely general formulation for the design resistance

$$R_d = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot R\{X_{\mathrm{d},i}; a_{\mathrm{d}}\} = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot R\left\{\eta_i \cdot \frac{X_{\mathrm{k},i}}{\gamma_{\mathrm{m},i}}; a_{\mathrm{d}}\right\} \qquad i \geq 1 \tag{11}$$

(the last equality of which follows from relation (8)).

The design resistance is clearly a function of the design values for materials properties as well as those for the geometric data. The purpose of the partial factor γ_{Rd} is to account for not only the inherent uncertainties in modelling the resistance R_{d}, but also any geometric deviations, if they have not been otherwise expressly modelled. The expression "design resistance " is to be taken in the most general sense possible. In fact, it may involve precise aspects of the material (resistance in terms of stress state and strain components), or the resistance of a straight section (for instance with respect to local buckling phenomena of thin-walled metal profiles), or furthermore, the resistance of an member. Distinct expressions for R_{d} obviously result depending upon the situation and quantities in question and the phenomena involved. However, all such expressions are governed by and conform to general expression (11) and have been set forth in the various specific Eurocodes, which provide for a varying number of parameters on geometric data and materials' resistance and mechanical properties.

A simplified formulation of (11) allows for calculating the design resistance as

$$R_{\mathrm{d}} = R\left\{\eta_i \cdot \frac{X_{\mathrm{k},i}}{\gamma_{\mathrm{M},i}}; a_{\mathrm{d}}\right\} \qquad i \geq 1 \tag{12}$$

In this way, the partial factor γ_{Rd} has been incorporated into the one for material resistance

$$\gamma_{\mathrm{M},i} = \gamma_{\mathrm{Rd}} \cdot \gamma_{\mathrm{m},i} \qquad i \geq 1 \tag{13}$$

If the structure is made up of a single material, such as, for instance, a metal framework, the following simple relation (14) can be used. Here the design resistance is calculated directly from the characteristic resistant values of the material or member, without resorting to explicit determination of the design values for the individual intervening variables.

$$R_d = \frac{R_k}{\gamma_M} \tag{14}$$

The Eurocodes recommend the following partial factors γ_M for the most commonly used materials in structural design by way of example

	γ_M
Structural steel	1,10
Concrete	1,50
Reinforcing steel	1,15
Profiled steel decking	1,10
Shear connectors (in steel-concrete composite structures)	1,25

Note that National Annexes to Eurocodes may recommend different partial factors γ_M.

4 ULTIMATE LIMIT STATES

4.1 Definition of ultimate limit states

Focusing one's attention on a single specific limit state (that is to say, on a critical condition of the structure), in order to verify its structural safety, one must determine whether the effects produced by the actions are smaller than the structure's resistance capacity. As already underscored in the foregoing, one crucial aspect is the set of limit states that are meaningful for the structure under verification. With the aim of standardising the terms of reference and avoiding possible discrepancies in interpretation, ultimate limit states have been divided into four distinct categories (each one assigned a three-letter designation in EN 1990 [1]).

EQU Limit States. These involve the loss of static equilibrium in the considered structure, either as whole rigid body or in any one of its parts. In such situations, the mechanical and resistance properties of the materials are not generally determining factors, while even modest geometric variations in the distribution of actions or their points of application may be crucial. Going beyond such limit conditions generally causes collapse of the structure, and their inclusion amongst the ultimate limit states thus seems obvious. Destabilising actions (unfavourable actions) must be taken into account by adopting higher design values, while assuming lower design values accounts for stabilising actions (which have a favourable effect on the structure's equilibrium). With regard to stabilising effects, only those actions that can reasonably be expected to occur in the structure should be included in the combination (for instance, when considering a specific stage of construction, the effective presence of finishing accessories or other equipment must be accounted for). It is moreover necessary to bear in mind the possibility that non-structural members can be replaced or removed.

In actual design practice, the real situations in which EQU limit states may arise are quite rare and mostly involve specific stages of a structure's execution. By way of example, consider a bridge deck being built with a launching nose and counterweighted on the opposite end (figure 4), or the overturning of a ground support member (Figure 5).

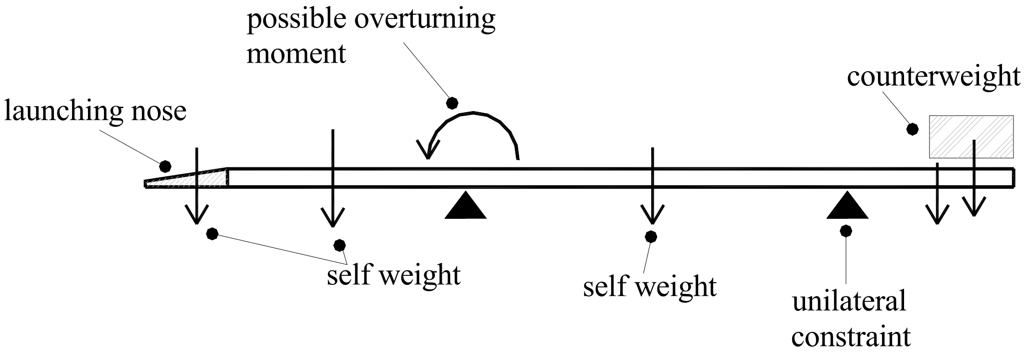


Figure 4. Example of an EQU limit state, bridge construction with launching nose

<u>*STR Limit States.*</u> These concern the failure or excessive deformation of a structure or its constituent members. In such cases, it is the resistance of the material that is the determining factor in verification.

<u>*GEO Limit States.*</u> These involve failure or excessive deformation of the soil. The critical factor in ensuring safety for such limit states is the mechanical characteristics of the ground. In the case of the design structural members (footings, piles, basement walls, etc.) involving geotechnical actions and resistance of the ground, STR and (GEO), for which three separate approaches are recommended in EN 1990, Section 6 should be used together.

<u>*FAT Limit States.*</u> These bear on the failure of the structure or structural members due to fatigue effects. As already highlighted, such limit states are atypical, as they occur not because of the design values for ultimate limit-state actions, but as the consequence of lower values which are however repeated frequently over the structure's operational lifetime. For this reason, the combination of actions to consider for FAT limit states are not furnished in EN 1990 [1], but in Eurocode EN 1991 and Eurocodes 1992 to 1999.

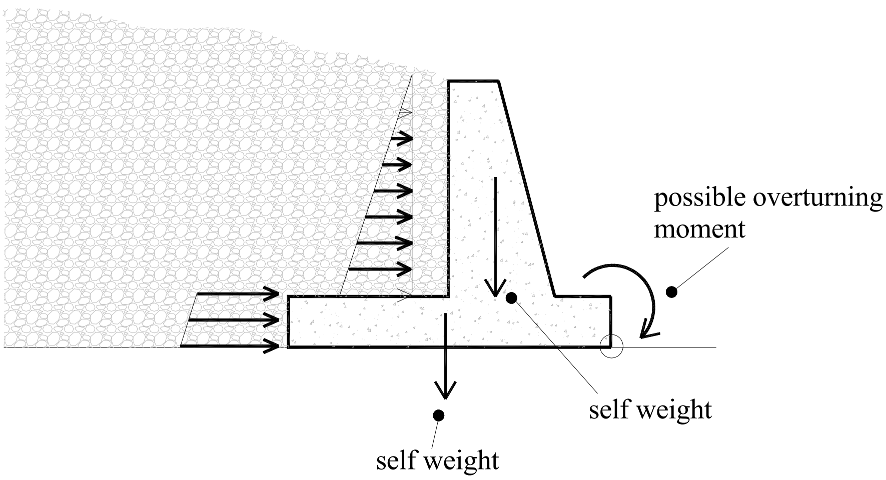


Figure 5. Overturning of a retaining wall

4.2 Verification of resistance and equilibrium at limit states

When considering a limit state due to cracking or excessive deformation (whether it be STR or GEO), structural safety can be guaranteed by satisfying relation

$$E_d \leq R_d \tag{15}$$

where E_d is the design value for the effect of the actions relevant to the limit state in question (for instance, a particular stress characteristic in a structural member), and R_d represents the corresponding resistance capacity. The equals sign (=) in expression (15) defines the limit situation, at which such capacity is exactly equal to the demands consequent to the actions, and no margin of safety exists any longer.

In concrete terms, checking the resistance of a structural member involves the following relation:

$$\left(N_{Sd}, V_{Sd}, M_{Sd}\right) \leq \left(N_{Rd}, V_{Rd}, M_{Rd}\right) \tag{16}$$

where N_{Sd}, V_{Sd}, M_{Sd} represent the values of the characteristics of the stress consequent to the design applied to the member in question, and N_{Rd}, V_{Rd} and M_{Rd} are the corresponding resistance values. In general, relation (16) is a condition regarding the stress characteristics considered separately, but it also includes the effects of possible interactions ((*V,M*); (*M,N*)).

Regarding structural verification, the analysis may concern one member alone, a section or a connection, but it can also pertain to particular aspects involved in the structural behaviour as a whole, as clarified by the following example.

Example: particular criteria for the safety at ultimate limit states

A well-known, fundamental property of frame structures for seismic areas is their capacity to dissipate energy. Dissipative capacity can be expressed fully only if the structure can draw on all its own plastic reserves, a situation that can occur only by avoiding partial collapse mechanisms that call into play only a limited number of resistant members (figure 6). The requirements of a global collapse mechanism impose that cracking form at the beams' ends, while the columns do not undergo plasticisation (except, at worst, in correspondence to the base sections), as per the so-called design principle of "*weak beams - strong columns*". For this reason, guidelines require that suitable relations be satisfied between the design resistant bending moments of sections contributing to joints belonging to a column and those of sections belonging to a beam. In essence, such a safety check involves resistance quantities and not the effects of actions.

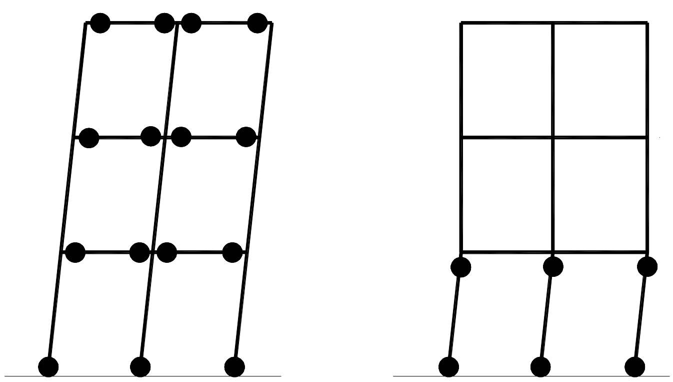


Figure 6. Global and partial collapse mechanisms in a frame structure

5 SERVICEABILITY LIMIT STATES

From a conceptual point of view, evaluating the reliability of a structure does not involve any change with respect to ultimate limit states. What must be verified is that the design values of the actions' effects, E_d, specified via the criterion in question (and calculated on the basis of suitable combinations), remain lower than the corresponding design limit values C_d

$$E_d \leq C_d \tag{16}$$

Conceptually, all such serviceability limit state checks conform to this general relation, though, as previously explained, they may concern aspects that are quite distinct one from the other, such as limitations to deflections, or the opening of cracks in reinforced concrete structures.

The partial factors, γ_M, for the materials properties in the case of serviceability limit states take a value of 1.0, except where otherwise specified (clause 6.5.4(1)).

As a practical example, if an operation check concerns the displacements and deformations of frame structures, (16) will take the form (17):

$$\left(\delta_{Vd}, \delta_{Hd}\right) \leq \left(\delta_{Vmax}, \delta_{Hmax}\right) \tag{17}$$

where δ_{Vd} generically represents the design vertical deflection of beams and floors, and δ_{Hd}, the design horizontal displacement of the frame. The corresponding limit values are indicatively δ_{Vmax} = L/250 (with L the span of the member in question) and δ_{Hmax} = h/300 (where h is the height of the individual storey) or the building's overall height).

In calculating the deflection of a beam, various contributions are distinguished, as highlighted by EN 1990, Annex A1, clause A1.4.3 [1] (Figure 7).

These include:
w_c: the precamber in the structural member without loading;
w_1: the initial deformation under permanent loads;
w_2: the deformation under permanent loads due to the action of long-term effects (creep,…)
w_3: additional degree of deflection caused by variable actions;
w_{tot}: total deflection, calculated as sum of w_1, w_2 and w_3.
w_{max}: the remaining part of the deflection after having discounted the portion due to precamber.

Table 1 gives indicative limiting values for particular serviceability criteria.

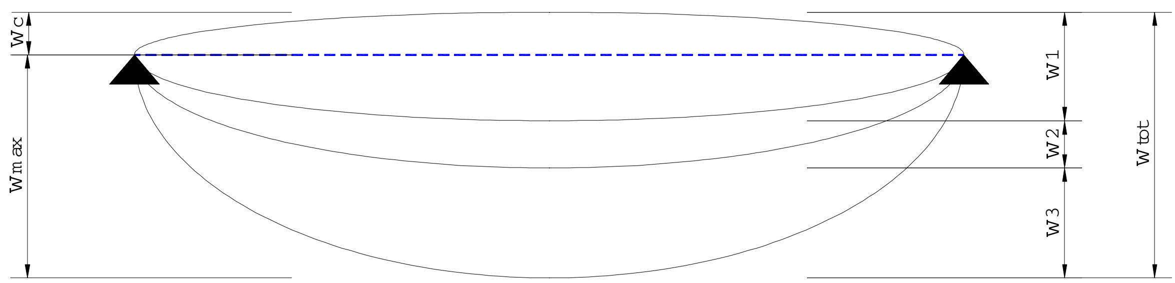


Figure 7. Overall deflection of a beam

Table 1. Indicative limiting values for particular serviceability criteria

	Serviceability Limit States Vertical deflections – See figure 7 (1), (2)		
	Irreversible effects of Actions	Reversible effects of Actions	
Serviceability requirements	Characteristic Combination w_{tot} or w_{max}	Frequent Combination w_{max}	Quasi-permanent Combination w_{max}
Function and damage to non-structural members (e.g. partition walls, claddings, etc) (3) • Brittle • Non-brittle **Function** and damage to structural members	 $\leq L/500$ to $L/360$ $\leq L/300$ to $L/200$ $\leq L/300$ to $L/200$		
To avoid ponding of water. Roof covered with waterproof membrane		$\leq L/250$ (4)	
Comfort of user or functioning of machinery		$\leq L/300$	
Crane gantry girders, deflection due to static wheel loads		$\leq L/600$	
Appearance			$\leq L/250$

(1) See EN 1990, Clause A1.4.3.
(2) The benefits of any pre-camber may be considered if appropriate.
(3) These figures assume that partitions, cladding and finishes have not been specifically detailed to allow for anticipated deflections.
(4) The deflection limit of $L/250$ is appropriate for flat roofs of 2,5% slope or greater. A more restrictive limit would apply for roofs of less than this slope.

Regarding verifications with respect to the natural vibrations of structural members, the calculations must consider all possible sources of vibrations (of which the most common include the synchronized movement of people walking, such as soldiers marching across a bridge, high-intensity traffic in a structure's immediate vicinity, the presence of machinery or the actions of the wind, to name just a few).

Sometimes the purpose of serviceability limit-state analysis is to verify that certain hypotheses assumed during the calculation stages are effectively corroborated by the physical reality and true structural behaviour. For instance, in composite steel-concrete beams, a serviceability check may concern the limitations to the slippage between the interface of the metal profile to the concrete slab: a single high value may be enough to effectively invalidate any verifications conducted on other serviceability limit states under the assumption of perfect adherence between the two materials (for instance, the beam deflection calculated under the assumption that the connection mechanisms are infinitely rigid and completely resistance recovering).

To sum up then, the procedures for limit-state verification by means of the method of partial factors can be illustrated schematically as in the flow chart in Figure 8:

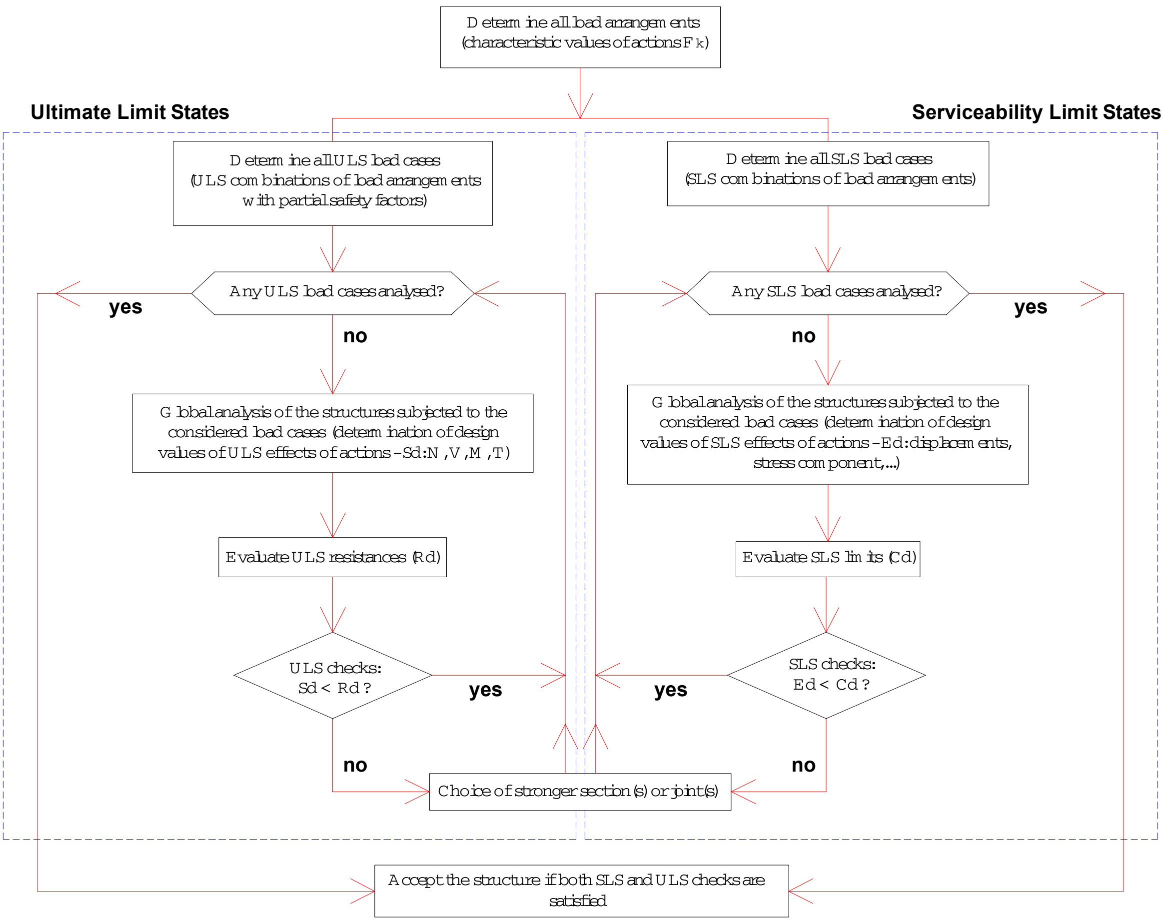


Figure 8. Verification procedure via partial factor method.

6 EXAMPLE: ULS AND SLS IN A FRAME STRUCTURE

To exemplify the foregoing considerations, let us now turn to a concrete situation: a composite steel-concrete frame structure (Figure 9), for which we shall describe the serviceability and ultimate limit states in reference to which the static performance of the structure is to be evaluated without entering into the details of the numerical values assumed by the various partial factors.

The foundations aside, the structure's resistant members include its floors, beams, columns and beam-column joints. The limit states against which the structural members must be safeguarded are given in the following.

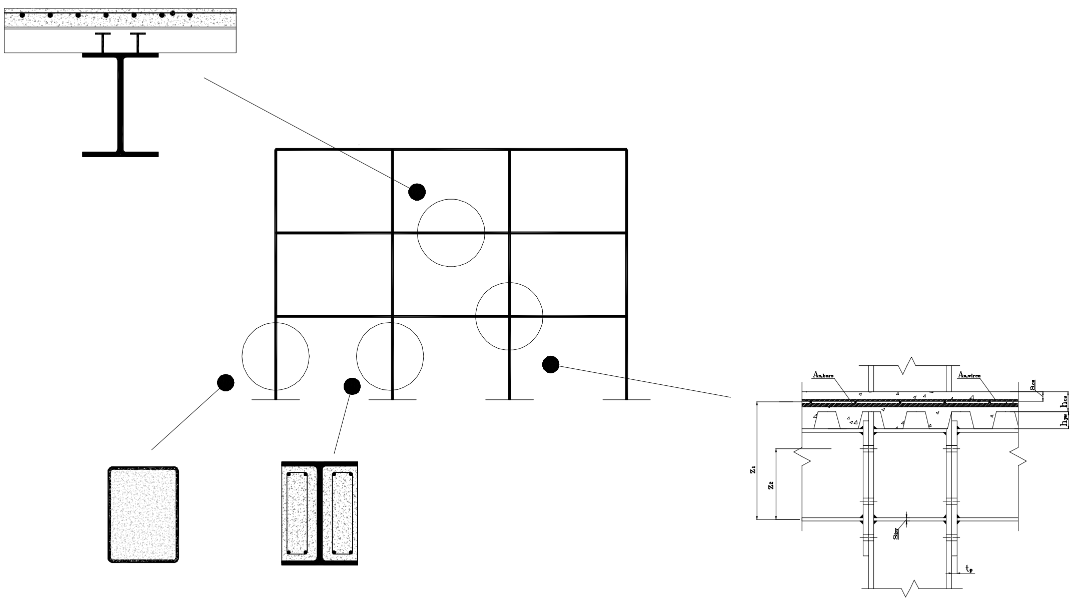
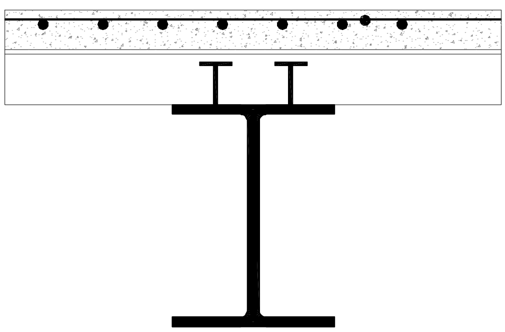


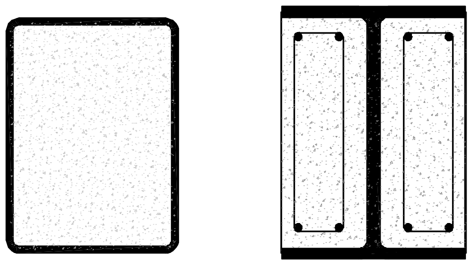
Figure 9. Example frame

The ultimate limit states involve the resistance of the structural members. With regard to the beams and columns, for instance, the following critical situations can be identified.



<u>*Resistance of beam cross section*</u>:

- Hogging and sagging bending moment;
- Resistance to vertical shear;
- Stability of web to vertical shear;
- Interaction between bending and vertical shear;
- Lateral torsional buckling of composite beam;
- Resistance of shear connectors.



Resistance of columns:
- Local buckling of steel members;
- Shear between steel and concrete components
- Resistance of members in compression and uniaxial bending;
- Resistance of members in compression and biaxial bending.

Concerning the composite steel-concrete floor, such a structure must be designed with the following ultimate limit states in mind:

- flexure;
- longitudinal shear;
- vertical shear;
- punching shear.

It should be noted that, as we are dealing with a representative frame structure for civilian dwelling, fatigue phenomena, connected to frequent cyclic actions during the structure's lifetime, are not meaningful.

The serviceability limit states regard deformational and tensional aspects, as well as checks of the vibration frequencies of the floors.

Limitations on displacements:

- Indicating Δ_i as the absolute horizontal displacement in correspondence to any given *i*-th floor level, the relative interstorey drift δ_i, for plane i will be equal to $\delta_i = \Delta_i - \Delta_{i-1}$.

Thus, it must be verified that:

$$\begin{cases} \delta_i \le \dfrac{h_i}{300} \\ \Delta_{\text{roof}} \le \dfrac{h}{500} \end{cases} \tag{18}$$

holds, where h_i represents the height of any given storey, h is the overall height of the frame and Δ_{roof}, the absolute horizontal displacement of the summit.

- The deflection of every single beam and every floor span of the structure must be limited under the most unfavourable load conditions.

Tensional limitations

- In order to avoid significant damage to the concrete of the composite beams as a consequence of frequent serviceability actions, it is necessary to control for material cracking. This is a particularly important aspect when dealing with a chemically hostile environment. Checks are also required for the floors, to which end, guidelines have been furnished as to the minimum amounts of reinforcement to use.

Limitations on frequencies

• The natural vibration frequency of the floors must be above a minimum value, set at 3 Hz. This particular verification is usually omitted for buildings, while it takes on particular importance for bridge decks.

7 COMBINATIONS OF ACTIONS

The effects E_d must be calculated by appropriately combining the respective design actions of the limit states under consideration (serviceability or ultimate). The issues involved have been thoroughly detailed in Chapter VIII. “Load combinations according to EN 1990”.

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[1] EN 1990 – Eurocode - Basis of structural design. European Committee for Standardisation, 04/2002.

CHAPTER VI: CLASSIFICATION OF ACTIONS

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Summary

Principles for specifications of actions are illustrated by practical examples concerning the different classes of actions. In general the characteristic, design and other representative values are defined as fractiles of appropriate theoretical models taking special account of its variation in time.

1 INTRODUCTION
1.1 Background materials

Basic principles and rules concerning actions and effect of actions are given in EN 1990 [1], and in the background material documents [2], in the International Standard ISO 2394 [3] and explained in the handbook [4]. Methods for the obtaining the representative values of the different classes of actions are given in the different parts of EN 1991devoted to actions and effect of actions [5], in the CIB documents Actions on structures [6]. Additional information may be found in the material oriented Eurocodes EN 1992 to EN 1999. Probabilistic characterization of the actions can be found in the Model Code of the JCSS [7]

2 ACTIONS AND EFFECT OF ACTIONS

2.1 Definition of actions

EN 1990 defines actions as: *a) Set of forces (loads) applied to the structure (direct action). b) Set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action)*. The effects of actions (or action effects) are the internal forces, moments, stresses, strains, etc on structural members, or deflections, rotations, etc. caused by the actions on the whole structure.

In general an action is described by a model; in most of the cases it is enough only one scalar to represent its magnitude, which may have several representative values. In some cases and for some verifications, a more complex representation of the magnitudes of some actions may be necessary.

In general the actions acting on a structure may have some statistical correlation between them and with the resistance variables. Most of the times, when they correspond to different sources, the error of considering statistical independence is not too significant and we can consider then as independent. But it is important to take account of the cases where the dependence consideration is relevant. The actions that can be assumed to be statistically independent in time and space of any other action acting on the structure are called *single actions*.

3 CLASSIFICATION OF THE ACTIONS

3.1 General

The actions can be classified according to different criteria. The relevant criteria will be related with the considered situation. Actions are classified by their variation in time as explained in the following section 3.2. Actions are also classified
– by their origin (see 3.3);
– by their spatial variation (see 3.4);
– by their nature and/or the structural response (see 3.5).
In addition environmental influences are described in section 3.6.

3.2 By their variation in time

The most important classification of actions is referred to the time the action is acting compared with the reference period. The actions are classified as:

- *permanent action (G)* that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value; *e.g.* self-weight of structures, fixed equipment and road surfacing, and indirect actions caused by shrinkage and uneven settlements
- *variable action (Q)* for which the variation in magnitude with time is neither negligible nor monotonic, *e.g.* imposed loads on building floors, beams and roofs, wind actions or snow loads
- *accidental action (A)*, usually of short duration, that is unlikely to occur with a significant magnitude on a given structure during the design working life, but its consequences might be catastrophic. *e.g.:* earthquakes, fires, explosions, or impact from vehicles.

The reference period is the time used as a basis for the statistical assessment of the actions and the time varying resistances. That means that the action has a working life of the structure could be split in some reference periods, of the same length or of different (random) length, in which the actions vary in a more or less similar pattern; i.e.: could be adopted independent, identical distribution functions for the action in any of such reference periods. Therefore, the maximum in each period corresponds to a realization of the same distribution function of maxima. See Figure 1.

The adequate reference period depends on the type of action: for climatic actions – snow, wind, etc. - a period of a year is, in general, adequate; i.e.: can be assumed that each annual maximum is independent of the maxima of previous and next year.

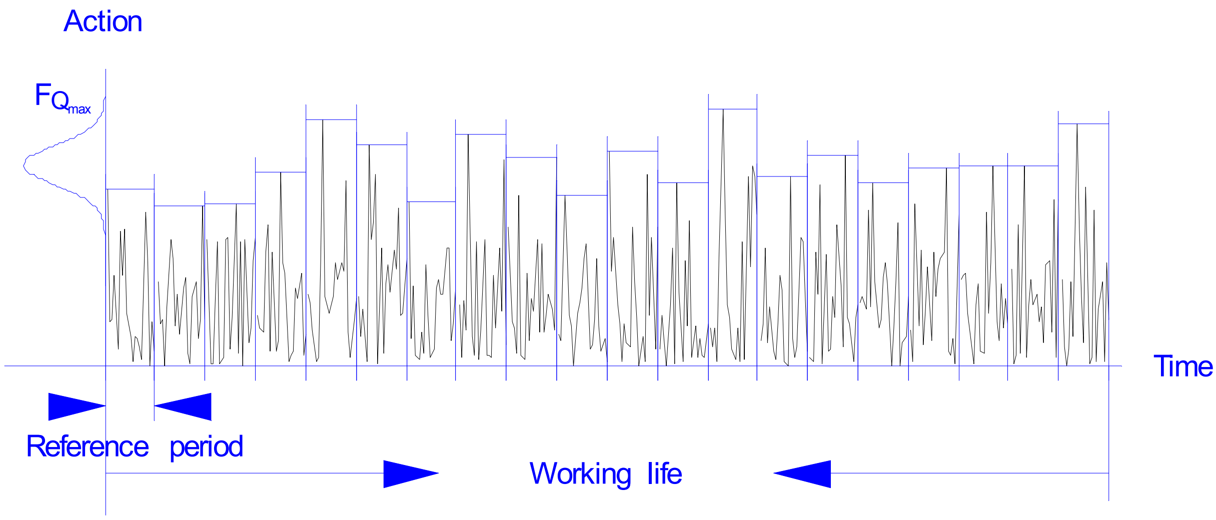


Figure 1 Model and distribution for a variable climatic action, e.g. wind

For other variable actions, like imposed loads, the period corresponding to a change of use or a change of owner may be more adequate. In this case, the action can be represented by a Poisson's process where both the length of the reference period and the values in each period are random. The average rate of change is generally assumed between 5 to 10 years depending of the use of the building. See Figure 2.

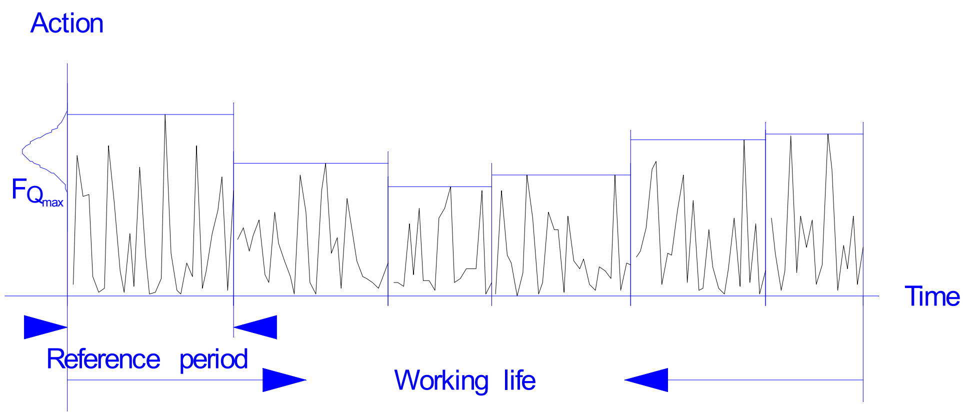


Figure 2. Model and distribution for a variable action with reference periods of random length.

For permanent actions, the reference period is taken generally the whole working life of the structure, and so is stated in EN 1990 for the self-weight of the structure it self. But, the EN1990, for the permanent actions, makes a distinction between the reference period, talking in general, and the working life, for the self-weight of the structure.

3.3 By their origin

Two classes are distinguished: *direct* actions consisting of forces (loads) applied to the structure and *indirect actions* consisting of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes.

3.4 By their variation in space

When an action has a fixed distribution and position over the structure or structural member such that the magnitude and direction of the action are determined unambiguously for the whole structure or structural member then it is considered as a *fixed action.* If the action may have various spatial distributions over the structure then it is a *free action*

3.5 By their nature or structural response

The *static actions* are those that do not cause significant acceleration of the structure or structural members. The d*ynamic actions* cause significant accelerations of the structure or structural members. In most cases for dynamic actions it is enough consider only the static component that may be multiplied by a coefficient to take account of the dynamic effects.

3.6 Environmental influences

The environmental influences may have a physical, chemical or biological character and may deteriorate the material of the structure affecting the safety and serviceability of the structure. For instance: the presence of chlorides or carbon dioxide and humidity what will lead to the corrosion of the reinforcement; fire that deteriorates the resistance of the materials; etc.

These influences can be classifies depending on the time variability and also considered as permanent, variable or accidental. Action and action effects can also be distinguished

3.7 Bounded and unbounded actions

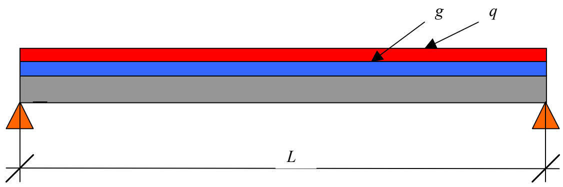
In some cases an upper (or lower) bound of an action can be found and then this bound can be established as representative value. For instance the load due to the water level in a tank.

3.8 Example

In the following example a comparison is made between the different action effects that correspond to the same actions in the case of a simply supported beam and a fully fixed beam.

Note that in the ideal case of a beam submitted to a uniform increase of temperature, ΔT, in all its length and cross-sections it will tend to have an expansion given by $\Delta L = \alpha\ L\ \Delta T$, where α is the coefficient of thermal expansion of the steel in this case. In the isostatic case there is no constrains and therefore the beam will expand without stresses. In the double fixed beam the expansion is constrained by the supports and a uniform compression stresses develop in the beam. The stress in the cross-section is given by $\Delta L/L = \sigma /E$.

a) Simply supported beam: IPE 240 S235



Span $L = 6{,}0$ m
Cross section area: $A = 39{,}12 \cdot 10^{-4}$ m^2
Moment of inertia $I_y = 3\,892 \cdot 10^{-8}$ m^4
Yield stress $f_y = 235$ MPa
Elastic modulus $E = 210\,000$ MPa
Thermal expansion coef.: $\alpha = 12 \cdot 10^{-6}$ / ºC

Actions, characteristic value:
Direct:
Permanent load: $g_k = 7{,}0$ kN/m
Variable load: $q_k = 3{,}0$ kN/m
Indirect:
Uniform temperature increase: $\Delta T = 20$ºC
Settlement at one support: $\delta = 12$mm

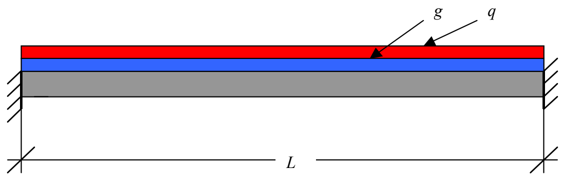
Effects of actions, characteristic value:
Permanent loads:
Mid span moment
$1/8\, g_k L^2 = 31{,}5$ kNm

Variable loads
Mid span moment
$1/8\, q_k L^2 = 13{,}5$ *kNm*

Indirect:
Settlement at one support: $\delta = 12$ mm
No effects

Uniform temperature increase: $\Delta T = 20$ºC
no effects*

b) Double fixed beam IPE 220 S235



Span $L = 6{,}0$ m
Cross section area: $A = 33{,}37 \cdot 10^{-4}$ m^2
Moment of inertia $I_y = 2\,772 \cdot 10^{-8}$ m^4
Yield stress $f_y = 235$ MPa
Elastic modulus $E = 210\,000$ MPa
Thermal expansion coef.: $\alpha = 12 \cdot 10^{-6}$ / ºC

Actions, *characteristic value:*
Direct:
Permanent load: $g_k = 7{,}0$ kN/m
Variable load: $q_k = 3{,}0$ kN/m
Indirect:
Settlement at one support: $\delta = 12$mm
Uniform temperature increase: $\Delta T = 20$ºC

Effects of actions, characteristic value:
Permanent loads:
Mid span moment
$1/24\, g_k L^2 = 10{,}5$ kNm
Moment at supports
$-1/12\, g_k L^2 = -91{,}0$ kNm
Variable loads
Mid span moment
$1/24\, q_k L^2 = 4{,}5$ kNm
Moment at supports
$-1/12\, q_k L^2 = -9{,}0$ kNm
Indirect:
Settlement at one support: $\delta = 12$ mm
Mid span moment 0 kNm
Moment at supports
$\pm\delta\, 6\, EI / L^2 = \pm\, 11{,}64$ kNm

Uniform temperature increase: ΔT
Uniform compression stress*
$\sigma = \alpha\, E\, \Delta T = 50{,}4$ Mpa

4 CHARACTERISTIC VALUES

4.1 General

The characteristic value of an action is its principal representative value and the basis for defining the accompanying values. When there is data enough to fix its value on statistical bases, it is chosen so as to correspond to a prescribed probability of not being exceeded on the unfavourable side during the "reference period" taking into account the design working life of the structure and the duration of the design situation. If not, a nominal value or a value fixed in the project documentation is chosen, provided that consistency is achieved with methods given in EN 1991.

4.2 Permanent actions

The characteristic value of a permanent action shall be assessed as follows:

- if the variability of G during the working life can be considered as small, one single value G_k equal to the mean value may be used;
- if the variability of G cannot be considered as small, two values shall be used: an upper value $G_{k,sup}$ and a lower value $G_{k,inf}$. Where $G_{k,inf}$ is adopted generally as the 5% fractile and $G_{k,sup}$ the 95% fractile of the statistical distribution for G, which may be assumed to be Gaussian.

With these assumptions $G_{k,inf}$ and $G_{k,sup}$ can be obtained from:

$$G_{k,inf} = \mu_G - 1{,}64\ \sigma_G = \mu_G\,(1 - 1{,}64\ V_G) \tag{1a}$$

$$G_{k,sup} = \mu_G + 1{,}64\ \sigma_G = \mu_G\,(1 + 1{,}64\ V_G) \tag{1b}$$

where μ_G is the mean value, σ_G is the standard deviation and V_G the coefficient of variation of the distribution of G. See Figure 3.

The self-weight of the structure may be represented by a single characteristic value and be calculated on the basis of the nominal dimensions and mean unit masses, see EN 1991-1.1.

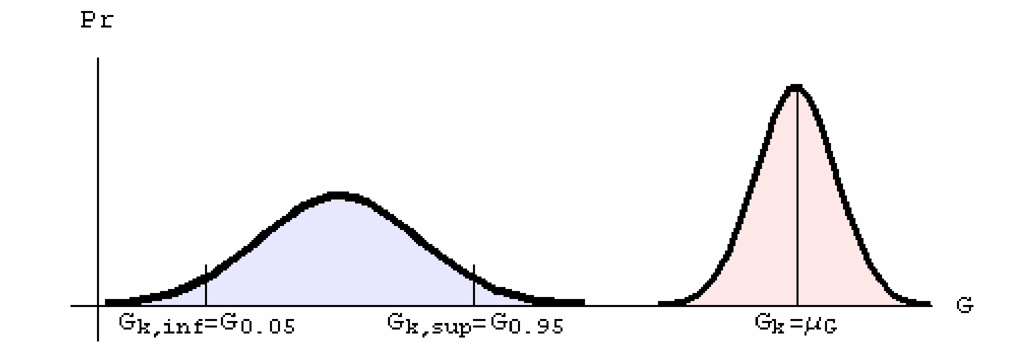


Figure 3. Characteristic values of permanent actions.

In the case that reference periods different from the whole working life were considered, assuming that the time length of the reference periods is exponentially distributed, the number of changes in the working life is Poisson's distributed. In this case the distributions of the maxima and minima for the working life are obtained as:

$$F_{Gmin}(x) = 1\text{-}\exp\,[-\lambda T\ \ F_G(x)] \tag{2a}$$

$$F_{Gmax}(x) = \exp\,[-\lambda T\ (1\text{-}F_G(x))] \tag{2b}$$

where λT represents the expected number of reference periods, (changes of use/owner) in the working life (values between 5 to 10 are usually considered) and $F_G(x)$ the distribution function of the permanent action. It is usually assumed that this distribution function do not change with the time in the different reference periods.

From these expressions, the characteristic values lower and upper, corresponding to values with a 5 and 95 % of not being reached or being over passed, respectively, in function of the fractiles of the distribution of $F_G(x)$, are given in the following table, depending in the mean number of changes:

λT	5	7	10
$G_{k,inf}$	0.010	0.007	0.005
$G_{k,sup}$	0.990	0.993	0.995

Assuming a Normal distribution for F_G (x), the characteristic values are obtained as:

$$F_{k,inf} = \mu_G - k\,\sigma_G = \mu_G\,(\,1 - k\ V_G) \tag{3a}$$

$$F_{k,sup} = \mu_G + k\,\sigma_G = \mu_G\,(\,1 + k\ V_G) \tag{3b}$$

where μ_G and σ_G are the mean value and coefficient of variation of F_G and k is obtained from the standardized Normal distribution values. Examples in function of the mean number of changes in the working life are given in the following table :

λT	5	7	10
k	2,32	2,44	2,57

4.3 Variable actions

For variable actions, the characteristic value (Q_k) shall correspond to either:

- an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period;
- a nominal value, which may be specified in cases where a statistical distribution is not known.

For characteristic value of climatic actions (wind, snow, etc,) a year reference period is generally chosen with a probability of excedence taken as 0,02 during the working life. This is equivalent to a mean return period of 50 years for the time-varying part.

However, in other cases, different values for the reference period may be more appropriate, e.g.: for imposed loads on buildings a reference period corresponding to the mean time between changes of ownership or a general refurbishment might be more appropriate. In these cases the probability of excedence will have to be different in order to obtain a similar return period. Return period of 50-100 years are usually chosen for variable actions.
ion directly.

4.4 Accidental actions

There is a lack of statistical data referred to accidental actions and the design value should be specified on the basis of nominal values for individual projects. Generally, the design value for accidental actions A_d, for a structure with medium consequences of failure can be fixed as such a level that the probability of excedence can be assessed in the order of magnitude of 10^{-4} per year.

There are specific parts of the Eurocodes to deal with the seismic actions (Eurocode 8) and fire (the Part 1-2 of Eurocode 1 and Part 1-2 of the material related Eurocodes (EN1992-1-2 to EN19991-2, except EN1997 and EN 1998)

4.5 Examples

Example 1:
Consider, for instance, a beam of normal weight concrete: from EN 1991-1-1 the mean value of the density can be taken as $\gamma = 24$ kN/m^3. In normal situations the characteristic value of the permanent load due to the self-weight of the beam is obtained multiplying this value by the nominal dimensions of the cross section. That is:

$$g_k = 24 \text{ a b kN/m},$$

where a and b are the dimensions of the cross section in metres.

Consider, now, that, for any circumstance, the structure is very sensitive to this self-weight and, then, is necessary to take account of the inferior and superior characteristic values. In [7] a coefficient of variation of 0,04 is given for the density of concrete. Introducing this value in the formulas (1a) and (1b), with the nominal dimensions of the cross section

$$g_{k,inf} = \mu_G\,(1- 1{,}64\ V_G) = 24 \text{ a b } (1\text{-}1{,}64\ 0{,}04) = 22{,}4 \text{ a b kN/m}$$

$$g_{k,sup} = \mu_G\,(1+ 1{,}64\ V_G) = 24 \text{ a b } (1\text{+}1{,}64\ 0{,}04) = 25{,}6 \text{ a b kN/m}$$

Example 2:

Considering now that the material of the beam is glulam of the type GL 36h. The mean density for this material given in EN 1991-1-1 is 4,4 kN/m^3 and in [7] a coefficient of variation of 0.1. This coefficient is in the limit indicated in EN 1990 for the consideration of a unique (mean) characteristic value or two values. To one or two in this case will depend in the ratio between the self-weight load and other loads. Substituting the mean and coefficient of variation values in (1a) and (2a) we obtain:

$$g_{k,inf} = \mu_G\,(1- 1{,}64\ V_G) = 4.{,}4 \text{ a b } (1\text{-}1{,}64\ 0{,}1) = 3{,}7 \text{ a b kN/m}$$

$$g_{k,sup} = \mu_G\,(1+ 1{,}64\ V_G) = 4{,}4 \text{ a b } (1\text{+}1{,}64\ 0{,}1) = 5{,}1 \text{ a b kN/m}$$

where a and b are the dimensions of the cross section in metres. We can see that now the characteristic values are a 16% inferior or superior to the mean values, while in the case of concrete were only a 7%.

Example 3:

Finally, consider a building where it is foresee that changes or use or owner can modify the permanent load of some non-structural elements or equipments. It is assumed that this part of the permanent loads has a mean value of 0,8 kN/m^2 with a coefficient of variation of 0,15. If a mean number of changes of 7 is adopted, i.e.: changes with a rate of approximately 7 years in the 50 years working life of the structure, from the equation (3b) the following characteristic value is obtained:

$$F_{k,sup} = \mu_G (1+ k\ V_G) = 0{,}8 (1 + 2{,}44\ 0{,}15) = 1{,}09 \text{ kN/m}^2$$

That is a 37% bigger than the mean value.

5 REPRESENTATIVE VALUES

5.1 General

The accompanying value of a variable action (ψQ_k) is the value of a variable action that accompanies the leading action in a combination. The accompanying value of a variable action may be the combination value, the frequent value or the quasi-permanent value.

5.2 The combination value of a variable action ($\psi_0 Q_k$)

Represented as a product of the characteristic value multiplied by the coefficient ψ_0 ($\psi_0 \leq 1$). It is used for the verification of ultimate limit states and irreversible serviceability limit states; is the value chosen - in so far as it can be fixed on statistical bases - so that the probability that the effects caused by the combination will be exceeded is approximately the same as by the characteristic value of an individual action.

5.3 The frequent value of a variable action ($\psi_1 Q_k$)

Represented as a product $\psi_1 Q_k$, used for the verification of ultimate limit states involving accidental actions and for verifications of reversible serviceability limit states; is the value determined –also if it can be fixed on statistical bases - so that either the total time, within the reference period, during which it is exceeded is only a small given part of the reference period, or the frequency of it being exceeded is limited to a given value. For buildings, for example, the frequent value is chosen so that the time it is exceeded is 0,01 of the reference period; for road traffic loads on bridges, the frequent value is assessed on the basis of a return period of one week. It may be expressed as a determined part of the characteristic value by using a factor $\psi_1 \leq 1$.

5.4 The quasi-permanent value of a variable action ($\psi_2 Q_k$)

Represented as a product $\psi_2 Q_k$, used for the verification of ultimate limit states involving accidental actions and for the verification of reversible serviceability limit states. Quasi-permanent values are also used for the calculation of long-term effects; is the value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. It may be expressed as a determined part of the characteristic value by using a factor $\psi_2 \leq 1$.

For loads on building floors, the quasi-permanent value is usually chosen so that the proportion of the time it is exceeded is 0,50 of the reference period. The quasi-permanent value can alternatively be determined as the value averaged on a chosen period of time. In the case of wind actions or road traffic loads, the quasi-permanent value is generally taken as zero.

The recommended values given in EN 1990 for buildings are given in table 1. In the Figure 2 a schematic representation of what means these accompanying values for a variable action along the working life of the structure.

Table 1: Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991- 1.1)			
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	0,7	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category F: traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G: traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H: roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)			
– Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
Note: The ψ values may set by the National annex.			

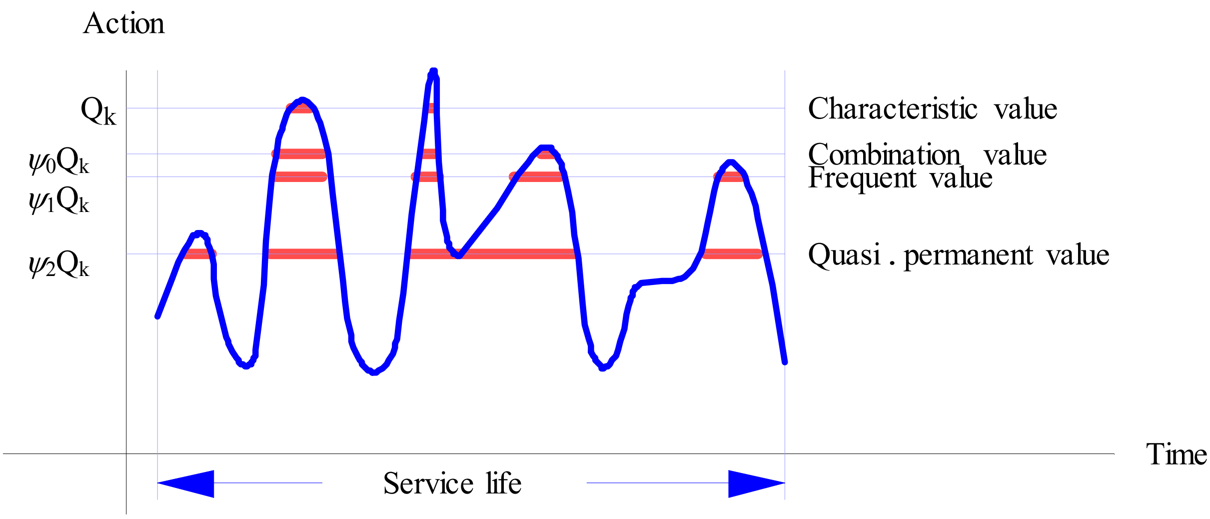


Figure 4. Schematic representation of a variable load and its representative values.

6 REPRESENTATION OF THE DYNAMIC ACTIONS

In the common cases the dynamic actions can be treated as static actions, *i.e.: quasi-static actions,* taking into account the equivalent static action obtained multiplying the magnitude of the static part of the action by an adequate coefficient. In most cases this coefficient is bigger than one, but if the time of application of the dynamic action is short, e.g. impacts from vehicles, this coefficient can be lower than one. The influence of the dynamic actions of the fatigue of the structural material has to be borne in mind.

The dynamic effects of the action are generally taken into account in the characteristic values and fatigue load models given in EN 1991. These effects are considered well implicitly in the characteristic loads or, well explicitly by applying dynamic enhancement factors to characteristic static loads.

When dynamic actions cause significant acceleration of the structure, and the simplification of the quasi-static approach is no longer valid, dynamic analysis of the system should be used to assess the response of the structure. The model shall describe the time variation of the action in such a way to give enough accurate results. The description can be done in the time domain, which is the time history of the action, or in the frequency domain (the last one the Fourier's Transformed of the former one). It is necessary to take into account the mutual influence of loads and structures. For instance, in lightweight structures the presence of loads changes the natural eigen-frequency of the structure. The models of dynamic analysis include:

- a stiffness model, similar to the static one
- a damping model, due to different sources, and
- a inertia model, taking account of the masses of the structural and non-structural elements

7 REPRESENTATION OF THE FATIGUE ACTIONS

When the actions may cause fatigue of the structural material it shall be verified that the reliability with respect to fatigue is sufficient. The models for fatigue actions are strongly dependent on the type of structural material and should be those that have been established in the relevant parts of EN 1991 from evaluation of structural responses to fluctuations of loads performed for common structures (e.g. for simple span and multi-span bridges, tall slender structures for wind, etc.).

In many cases the models are based on empirically known relations between the stresses and the number of cycles to failure (S-N curves) or in mechanic of fracture considerations.

8 REPRESENTATION OF GEOTECHNICAL ACTIONS

Geotechnical actions shall be assessed in accordance with EN 1997-1.

9 REPRESENTATION OF THE ENVIRONMENTAL INFLUENCES

The effects of environmental influences should be taken into account, and where possible, be described quantitatively in the same way as for actions. The effects of the environmental influences are strongly dependent on the type of structural material.

When a model of structural deterioration related to the *in situ* environmental conditions can be established it is possible to define a limit state according with it. In this case the environmental influences are treated exactly as actions. This model could be deterministic with the uncertainties introduced via some selected random parameters or taking account of the model uncertainty.

Unfortunately, up to now there are no accepted models for most of these influences, due to the lack of data, which essentially depend very sharply on the location of the site. Up to now, most of the cases we have to deal with the environmental influences only with empirical deemed to satisfy rules.

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CHAPTER VII: RESISTANCE AND GEOMETRIC DATA

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Summary

Principles for specifications of material properties and geometric data are illustrated by practical examples concerning concrete, steel and other structural materials and products. In general the characteristic and design values are defined as fractiles of appropriate theoretical models. The most important material properties and tolerance criteria are included in attached Appendix A (Material properties) and Appendix B (Tolerances for the overall imperfections).

1 INTRODUCTION

1.1 Background materials

Basic principles and rules concerning structural resistance and geometric data are given in EN 1990 [1], in the International Standard ISO 2394 [2], background materials [3] and Handbook [4]. Additional information may be found in the material oriented Eurocodes EN 1992 to EN 1999 and in working materials of JCSS [5] devoted to material properties and geometric data.

1.2 General principles

The reliability principles of Eurocodes are based on a general principle that all the basic variables are considered as random quantities having appropriate type of probability distribution. Characteristics of material properties and geometric data are defined as fractiles of appropriate distributions. Guidance concerning models for material properties and geometric data is provided in JCSS documents [3,5]. MATHCAD sheets that supplement described computational procedures can be effectively used in practical applications of computational procedures for determination of characteristic and design values.

2 CHARACTERISTIC VALUES

2.1 General

Properties of materials and soils constitute an important group of basic variables that may significantly affect the structural reliability. In design calculations the properties of materials (including soil and rock) or products are represented by characteristic values, which correspond to the prescribed probability of not being infringed (exceeded in an unfavourable sense). When a material property X is an extremely significant variable, both the lower and upper characteristic values $X_{\mathrm{k,inf}}$ and $X_{\mathrm{k,sup}}$ should be taken into account (see Figure 1).

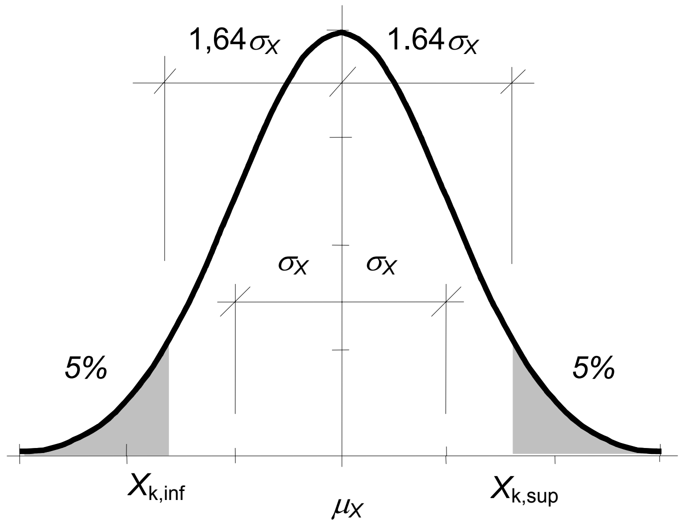

Figure 1. Lower $X_{k,inf}$ and upper $X_{k,sup}$ characteristic values of a material property X

In most cases the lower value $X_{k,inf}$ of material or product property is unfavourable. Then the 5% (lower) fractile is usually considered as the characteristic value. There are, however, cases when an upper estimate of strength is required (e.g. for the tensile strength of concrete when the effect of indirect actions is analysed). In these cases the use of the upper characteristic value of the strength $X_{k,sup}$ is needed. When the upper value is unfavourable, then the 95% (upper) fractile is usually considered as the characteristic value. General information about the strength and stiffness parameters is presented in Appendix A to this Chapter.

2.2 Determination of the characteristic values

A material property shall normally be determined from standardised tests performed under specified conditions. It is sometimes necessary to apply a conversion factor to convert the test results into values which can be assumed to represent the behaviour of the structure or the ground. These factors and other details of standardised tests are given in EN 1992 to 1999. For traditional materials, e.g. steel and concrete, previous experience and extensive tests are available and appropriate conversion factors are well established and presented in EN 1992 and 1993 (see also Annex D of this Background Document). Properties of new materials should be obtained from an extensive testing program, including tests on complete structures, revealing the relevant properties and appropriate conversion factors. New materials should be introduced only if comprehensive information about their properties supported by experimental evidence is available.

Assuming that the theoretical model for random behaviour of a material property is known or sufficient data are available to determine such a model, basic operational rules to determine specified fractiles are described below in this Chapter. If only limited test data are available, then statistical uncertainty due to limited number of data should be taken into account and the above mentioned operational rules should be substituted by more complicated statistical techniques (see for example Annex D of EN 1990 [1]).

According to EN 1990 [1] whenever there is a lack of information about the statistical distribution of the property a nominal value may be used in the design. In the case of insignificant sensitivity to the variability of the property a mean value may be considered as the characteristic value. Relevant values of material properties and their definitions are available in ENs 1992 to 1999.

Note that stiffness parameters are normally defined as the mean value. This can be explained in the following way. Very often, stiffness parameters are used in interaction models (e.g. ground-structure interaction) or in finite element models associating several materials. The stiffness properties of materials cannot be altered by partial factors because the results of the calculation would be distorted. This is the reason for which it is recommended not to alter stiffness parameters. Nevertheless, in some cases (e.g. for the calculation of piles subject to horizontal forces at the top), it may be necessary to take into account a lower and an upper value of these parameters, generally assessed from engineering judgment.

In general, when the lower or upper characteristic value is derived from tests, the available data should be carefully examined for cases where the material (e.g. timber and steel components) is classified using a grading system with a number of classes in order to avoid the possibility that the manufacturer may have included the specimens failed for the upper grade into the lower grade, thus distorting the statistical characteristics (including the mean, standard deviation and fractiles) of the lower grade, see Figure 2. Obviously such a "mixture" of two grades may significantly affect both the lower and upper characteristic value.

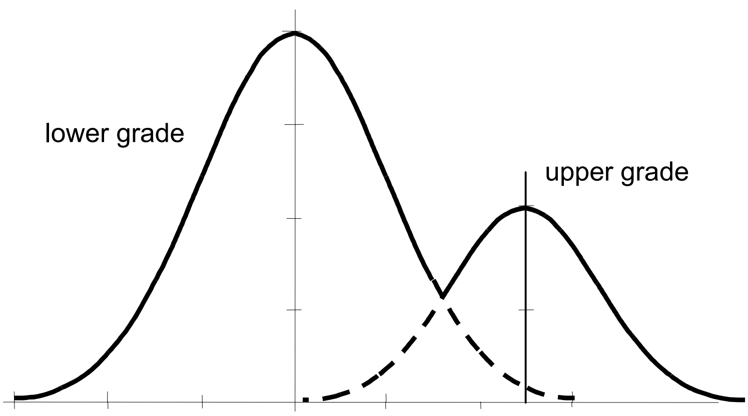

Figure 2. Distortion of probability density function due to combination of two material grades.

Specific values of material and product properties are given in material oriented EN 1992 to 1999 where also appropriate partial factors are specified. Unless suitable statistical information are available a conservative value of partial factors shall be normally used.

3 BASIC STATISTICAL TECHNIQUE

3.1 Normal and general lognormal distribution

As most material properties are random variables of considerable scatter, applied characteristic values should always be based on appropriate statistical parameters or fractiles. Commonly, for a given material property X the following statistical parameters are considered: the mean μ_X, standard deviation σ_X, coefficient of skewness (asymmetry) α_x. In some cases also other statistical parameters, e.g. the lower or upper distribution limit are taken into account. In case of a symmetrical distribution (e.g. the normal distribution) the coefficient $\alpha_X = 0$ and the normal distribution characterised by the mean μ_X and standard deviation σ_X is usually considered. This type of distribution is indicated in Figure 3 by a full line.

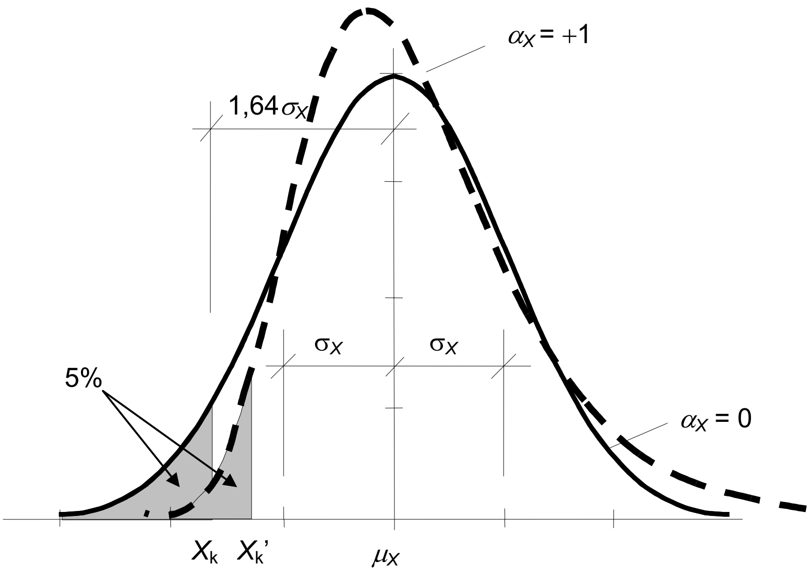

Figure 3. Normal and lognormal distribution

The characteristic and design values of material properties are defined as specified fractiles of the appropriate distribution. Usually the lower 5% fractile is assumed for the characteristic strength X_k and a smaller fractile probability (around 0,1%) is considered for the design value X_d. If the normal distribution is assumed, the characteristic value X_k, defined as the 5% lower fractile, is derived from the statistical parameters μ_X and σ_X as

$$X_k = \mu_X - 1{,}64\ \sigma_X \tag{1}$$

where the coefficient -1,64 corresponds to the fractile probability 5%. The statistical parameters μ_X, σ_X and the characteristic value X_k are shown in Figure 3 together with the Normal probability density function of the variable X (full line). The coefficient -3,09 should be used when the 0,1 % lower fractile (design value) is considered.

Generally, however, the probability distribution of the material property X may have asymmetrical distribution, usually with positive or negative skewness α_X. The dashed line in Figure 3 shows the general three parameter (one-sided) lognormal distribution having a positive coefficient of skewness $\alpha_X = 1$. The lower limit of the definition domain is then $x_0 = \mu_x - 1{,}34\ \sigma_x$. In case of asymmetric distribution a fractile X_p corresponding to the probability P may be then calculated from the general relationship

$$X_p = \mu_X + k_{P,\alpha}\ \sigma_X \tag{2}$$

where the coefficient $k_{P,\alpha}$ depends on the probability P and on the coefficient of skewness α_X. Assuming the three parameter lognormal distribution, selected values of the coefficient $k_{P,\alpha}$ for determination of the lower 5% and 0,1% fractiles are indicated in Table 1.

Table 1. The coefficient $k_{P,\alpha}$ for determination of the lower 5% and 0,1% fractile assuming three parameter lognormal distribution

Coefficient of skewness α_X	-2,0	-1,0	-0,5	0,0	0,5	1,0	2,0
Coefficient $k_{P,\alpha}$ for P =5%	-1,89	-1,85	-1,77	-1,64	-1,49	-1,34	-1,10
Coefficient $k_{P,\alpha}$ for P =0,1%	-6,24	-4,70	-3,86	-3,09	-2,46	-1,99	-1,42

It follows from Table 1 and equation (2) that the lower 5% and 0,1% fractiles for the normal distribution (when $\alpha_X = 0$) may be considerably different from those corresponding to an asymmetrical lognormal distribution. When the coefficient of skewness is negative, $\alpha_X < 0$, the predicted lower fractiles for lognormal distribution are less (unfavourable) than those obtained from the normal distribution with the same mean and standard deviation. When the coefficient of skewness is positive, $\alpha_X > 0$ (see Figure 3), the predicted lower fractiles for lognormal distribution are greater (favourable) than those obtained from the normal distribution.

3.2 Lognormal distribution with the lower bound at zero

A popular lognormal distribution with the lower bound at zero, which is used frequently for various material properties, has always a positive skewness $\alpha_X > 0$ given as

$$\alpha_X = 3\ V_X + V_X^3 \tag{3}$$

where V_X denotes the coefficient of variation of X. When, for example, $V_X = 0,15$ (a typical value for in situ cast concrete) then $\alpha_X \cong 0,45$. For this special type of distribution the coefficients $k_{P,\alpha}$ can be estimated from data indicated in Table 1 taking into account actual skewness α_x given by equation (3). However, in this case of the lognormal distribution with lower bound at zero the fractile can be determined from the following equation

$$X_P = \mu_X\ \exp(k_{P,0}\ \sqrt{\ln(1+V_X^2)})/\sqrt{(1+V_X^2)} \tag{4}$$

which is often approximated (for $V_X < 0,2$) by a simple formula

$$X_P = \mu_X \exp(k_{P,0}\ V_X) \tag{5}$$

Note that $k_{P,0}$ is the coefficient taken from Table 1 for the skewness $\alpha_x = 0$ (as for the normal distribution). As mentioned above usually the probability $P = 0,05$ is assumed for the characteristic value, thus $X_{0,05} = X_{\mathrm{k}}$, and the probability $P = 0,001$ is approximately considered for the design value, thus $X_{0,001} \cong X_{\mathrm{d}}$. Relative values of these important fractiles (related to the mean μ_X) determined using equation (5) are shown in Figure 4, where the ratios $X_{0,05}/\mu_X$ and $X_{0,001}/\mu_X$ are plotted as functions of the coefficient of variation V_X. Figure 4 also shows corresponding skewness α_x given by equation (3).

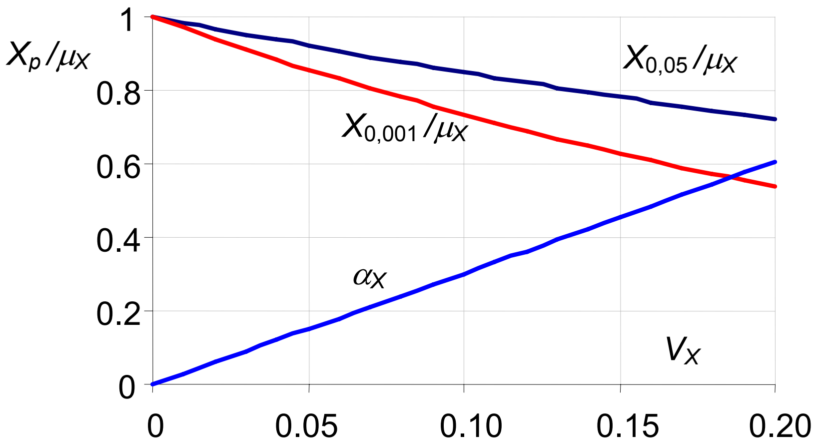

Figure 4. The skewness α_x and fractiles $X_{0,05}$ and $X_{0,001}$ (the characteristic and design values) as fractions of the mean μ_X for lognormal distribution with lower bound at zero versus coefficient of variation V_X.

The skewness α_x shown in Figure 4 should be used as a sensitive indicator for verification of suitability of the lognormal distribution with lower bound at zero. If the actual skewness determined from available data is considerably different from that indicated in Figure 4 (which is given by equation (3) for a given V_X) then a more general three parameter lognormal or other types of distribution (for example the distribution of minimum values, type III, called also Weibull distribution) should be used. Nevertheless simple expression (2) with the coefficients $k_{P,\alpha}$ taken from Table 1 may provide a good approximation or a control check. If the actual skewness is small, say $|\alpha_X|<0,1$, then the normal distribution may be used as an approximation (expression (2) with the coefficients $k_{P,0}$).

However, when the normal distribution is used and the actual distribution has a negative coefficient of skewness, $\alpha_x < 0$, the predicted lower fractiles will then have an unfavourable error (i.e. will be greater than the correct values). For the case when the correct distribution has a positive coefficient of skewness, $\alpha_x > 0$, the lower fractiles, estimated using the normal distribution, will have a favourable error (i.e. will be less than the correct values). However, In the case of the 5% lower fractile value (commonly accepted for the characteristic value) with the coefficient of skewness within the interval <-1, 1> the error is relatively small (about 6% for a coefficient of variation less than 0,2).

Considerably greater differences may occur for the 0,1% fractile value (which is approximately considered for design values) when the effect of asymmetry is more significant than in case of 5% fractile. For example, in the case of a negative asymmetry with α_x = -0,5 (extreme case but still indicated by statistical data for strength of some grades of steel and concrete), and a coefficient of variation of 0,15 (adequate to concrete), the correct value of the 0,1% fractile value corresponds to 78% of the value predicted assuming the normal distribution. When the coefficient of variation is 0,2, then the correct value decreases to almost 50 % of the value determined assuming the normal distribution.

However, when the material property has a distribution with a positive skewness, then the estimated lower fractile values obtained from the normal distribution may be considerably lower (and therefore conservative and uneconomical) than the theoretically correct value corresponding to appropriate asymmetrical distribution. Generally, the consideration of asymmetry to determine properties is recommended whenever the coefficient of variation is greater than 0,1 or the coefficient of skewness is outside the interval <-0,5, 0,5>. This is one of the reasons why the design value of a material property should be preferably determined on the basis of the characteristic value which is not significantly sensitive to the distribution asymmetry.

When the upper fractiles representing upper characteristic values are needed, equation (2) may be used provided that all numerical values for the coefficient of skewness α_x and $k_{p,\alpha}$ given in Table 1 are taken with the opposite sign. However, in this case the experimental data should be carefully checked to avoid the possible effect of material not passing the quality test for the higher grade and affecting the lower grade when included in the experimental data (see Figure 2).

The above operational rules are applicable when the theoretical model for the probability distribution is known (for example based on extensive experimental data and previous experience). If, however, only limited experimental data are available, then a more complicated statistical technique should be used (see Annex D of EN 1990 [1]) to take account of statistical uncertainty due to limited information. In general the statistical uncertainty leads to more conservative estimates.

Example

Consider a concrete having the mean $\mu_X = 30$ MPa and standard deviation $\sigma_X = 5$ MPa (the coefficient of variation $V_X = 0{,}167$). Then the 5% fractile (the characteristic value) is:

- assuming normal distribution (equation (2))

$$X_{0,05} = \mu_X - k_{P,0}\ \sigma_X = 30 - 1{,}64\times 5 = 21{,}7 \text{ MPa} \tag{6}$$

- assuming lognormal distribution with the lower bound at zero (equation (5))

$$X_{0,05} = \mu_X \exp(k_{P,0}\ w_X) = 30\times\exp(-1{,}64\times 0{,}167) = 22{,}8 \text{ MPa} \tag{7}$$

The 0,1% fractile (the design value) is

- assuming normal distribution (equation (2))

$$X_{0,001} = \mu_X - k_{P,0}\ \sigma_X = 30 - 3{,}09\times 5 = 14{,}6 \text{ MPa} \tag{8}$$

- assuming lognormal distribution with the lower bound at zero (equation (5))

$$X_{0,001} = \mu_X \exp(k_{P,0}\ w_X) = 30\times\exp(-3{,}09\times 0{,}167) = 17{,}9 \text{ MPa} \tag{9}$$

Obviously the difference caused by the assumed type of distribution is much larger (23%) in case of 0,001 fractile than in case of 0,05 fractile. Compared to the normal distribution the 0,05 fractile (the characteristic value) for the lognormal distribution is by 5% larger, the 0,001 fractile (the design value) by 23%.

Note that in accordance with equation (3) the lognormal distribution with the lower bound at zero has a positive skewness

$$\alpha_X = 3\ w_X + w_X^3 = 3 \times 0{,}167 + 0{,}167^3 = 0{,}5 \tag{10}$$

which should be checked against actual data. As a rule, however, a credible skewness cannot be determined due to lack of available data (the minimum sample size to determine the skewness should be at least 30 units). Then the lognormal distribution with the lower bound at zero is recommended to be considered as a first approximation.

4 GEOMETRICAL DATA

Geometrical data are generally random variables. In comparison with actions and material properties their variability can in most cases be considered small or negligible. Such quantities can be assumed to be non-random and as specified on the design drawings (e.g. effective span, effective flange widths). However, when the deviations of certain dimensions can have a significant effect on actions, action effects and resistance of a structure, the geometrical quantities should be considered either explicitly as random variables, or implicitly in the models for actions or structural properties (e.g. unintentional eccentricities, inclinations, and curvatures affecting columns and walls). Relevant values of some geometric quantities and their deviations are usually provided in Eurocodes 2 to 9. Selected informative values for geometric quantities describing the shape, size and overall arrangement of structures are indicated in Appendix to this Chapter.

The manufacturing and the execution process (e.g. setting out and erection) together with physical and chemical causes will generally result in deviations in the geometry of a completed structure, compared to the design. Generally two types of deviations may occur:

(a) initial (time independent) deviations due to loading, production, setting out and erection;
(b) time dependent deviations due to loading and various physical, chemical causes.

The deviations due to manufacturing, setting out and erection are also called induced deviations; the time dependent deviations due to loading and various physical and chemical causes (creep, effect of temperature and shrinkage) are called inherent deviations (or deviations due to the inherent properties of structural materials).

For some building structures (particularly when large-span precast components are used) the induced and inherent deviations may be cumulative for particular components of the structure (e.g. joints and supporting lengths). In design, the effects of cumulative deviations with regard to the reliability of the structure including aesthetic and other functional requirements should be taken into account.

The initial deviations of a dimension may be described by a suitable random variable and the time dependent deviations may be described by the time dependent systematic deviations of the dimension. To clarify these fundamental terms Figure 5 shows a probability distribution function of a structural dimension a, its nominal (reference) size a_{nom}, systematic deviation $\delta a_{\mathrm{sys}}\ (t)$, limit deviation Δa and the tolerance width $2\Delta a$.

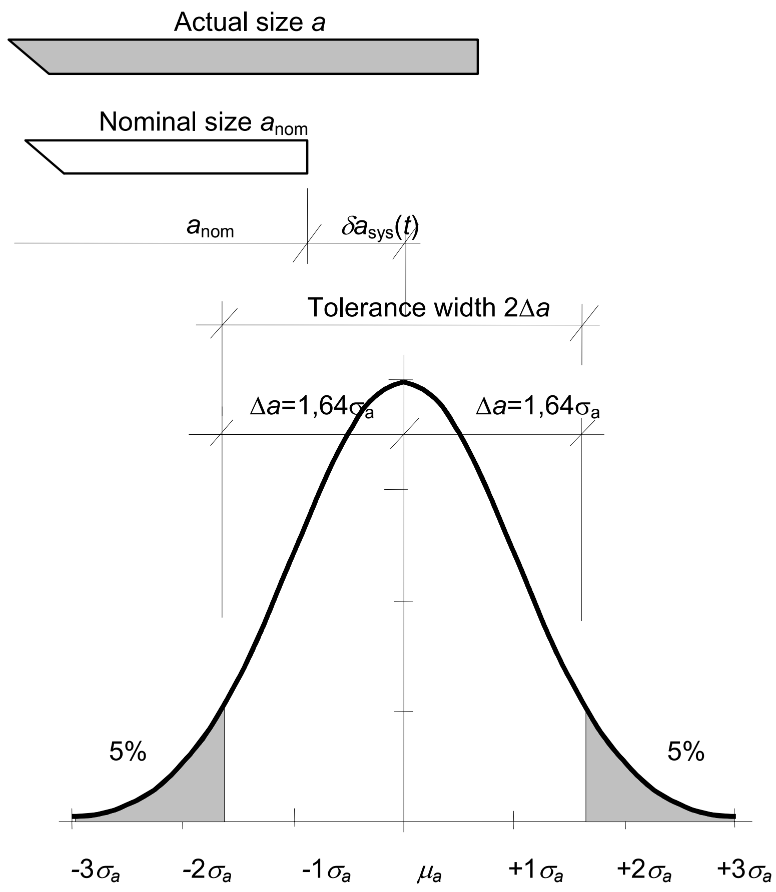

Figure 5. Characteristics of a dimension *a*.

The nominal (reference) size a_{nom} is the basic size which is used in design drawings and documentation, and to which all deviations are related. The systematic deviation $\delta a_{\text{sys}}(t)$ is a time dependent quantity representing the time dependent dimensional deviations. In Figure 8 the limit deviation Δa is associated with the probability 0.05, which is the probability commonly used to specify the characteristic strength. In this case the limit deviation is given as Δa = 1,64 σ_a. In special cases, however, other probabilities may be applied and instead of the coefficient 1,64 other values should be used. Generally a fractile a_p of a dimension a corresponding to the probability p may be expressed as

$$a_p = a_{\text{nom}} + \delta a_{\text{sys}}(t) + k_p\, \sigma_a \tag{11}$$

where the coefficient k_p depends on the probability p and assumed type of distribution (see also discussion in 3).

Example
Consider a simple assembly shown in Figure 6. A prefabricated horizontal component is erected on two vertical components. The relevant dimensions describing the setting out, erection and dimensions of the component are obvious from Figure. 6. For the supporting length b (which is one of the lengths b_1 and b_2 indicated in Figure 6) of the horizontal

component the nominal value b_{nom}= 85 mm is designed. Taking into account deviations in the setting out, manufacturing and erection it was assessed that the supporting length has approximately the normal distribution with the systematic deviation $\delta a_{sys}(t)$ = 0 and the standard deviation σ_b = 12 mm.

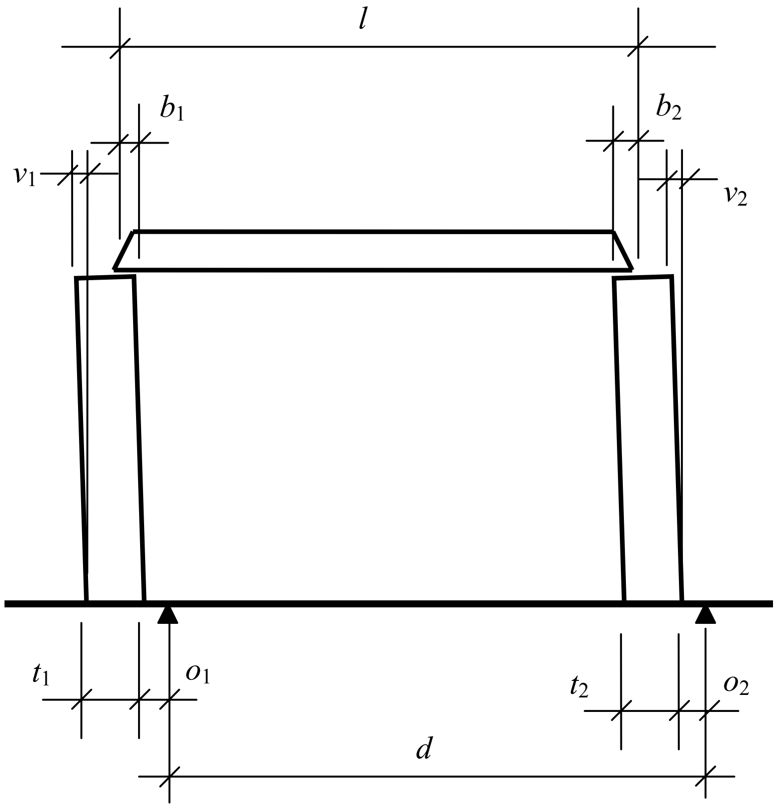

Figure 6. A horizontal component erected on two vertical components.

It follows from equation (11) that the lower 5% fractile of the length is

$$b_{0,05} = 85 - 1{,}64\times 12 = 65 \text{ mm} \tag{12}$$

and the upper 95% fractile is

$$b_{0,95} = 85 + 1{,}64\times 12 = 105 \text{ mm} \tag{13}$$

Therefore, with the probability 0,90 the supporting length will be within the interval 85±20 mm and the corresponding eccentricity of the loading transmitted by the component to the supporting vertical member may differ by ± 10 mm from an assumed value.

The initial and time dependent deviations may lead to considerable variations in the shapes and sizes of structures and their parts as follows:

(a) in the shape and size of cross sections, support areas, joints, etc.
(b) in the shape and size of components
(c) in the overall shape and size of the structural system.

For cases (a) and (b) where the variation of structural dimensions can affect the safety, serviceability and durability of the structures, specified permitted deviations or tolerances are given in relevant EN 1992 to 1999. These tolerances are denoted as normal tolerances and should be taken into account if other smaller or larger tolerances are not specified in the design. When deviating from EN 1992 to 1999 with regard to tolerances, the design should carefully include implications of other deviations with regard to structural reliability. For case (c) some informative tolerances are given in Appendix B to this Chapter.

REFERENCES

[1] *EN 1990 Eurocode - Basis of structural design*. CEN 2002.

[2] *ISO 2394 General principles on reliability for structures*, ISO 1998.

[3] *JCSS: Background documentation, Part 1 of EC 1 Basis of design*, 1996.

[4] *Gulvanessian, H. – Calgaro, J.-A. – Holický, M.: Designer's Guide to EN 1990, Eurocode: Basis of Structural Design*; Thomas Telford, London, 2002, ISBN: 07277 3011 8, 192 pp.

[5] JCSS: *Probabilistic model code.* JCSS working materials, http://www.jcss.ethz.ch/, 2001.

[6] *EN 1991-1-1 Eurocode 1 Actions on structures. Part 1-1 General actions. Densities, self-weight, imposed loads for buildings*, CEN 2002

APPENDIX A TO CHAPTER VII. MATERIAL PROPERTIES

Basic material properties

The most important material properties introduced in design calculations describe the fundamental engineering aspects of building materials:

(a) strength f
(b) modulus of elasticity E
(c) yield stress (if applicable) σ_y
(d) limit of proportionality ε_y
(e) strain at rupture ε_f.

Figure 7 shows a typical example of a one dimensional stress (σ) - strain (ε) diagram together with the above mentioned fundamental quantities.

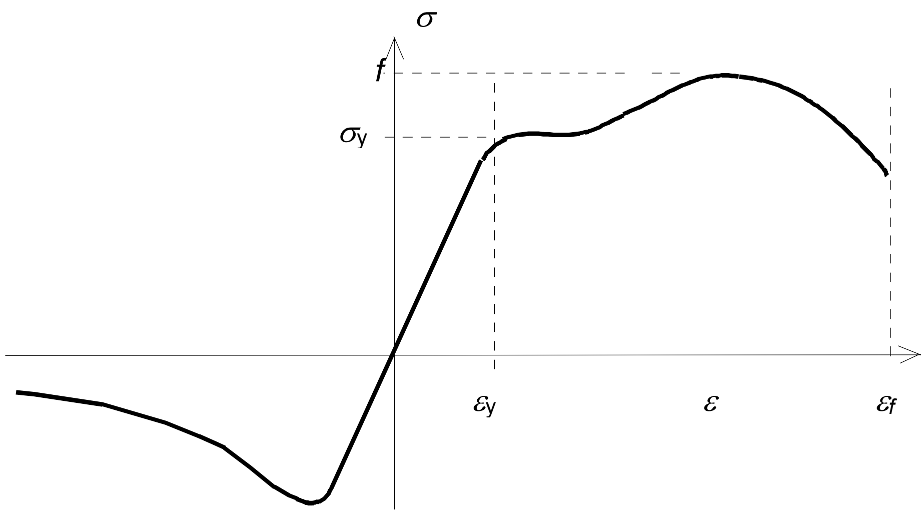

Figure 7. One dimensional σ - ε diagram

In design calculations, strength parameters are usually introduced by the lower 5% fractiles representing the characteristic values, stiffness parameters are introduced by their mean values. However, when stiffness affects the structural load bearing capacity, an appropriate choice of the relevant partial factor should compensate for the mean value choice. As mentioned above in this Chapter, in some cases the upper characteristic value for the strength is of importance (e.g. for the tensile strength of concrete when the effect of indirect action is calculated). Some of the above mentioned properties, e.g. strain at rupture, may be introduced in design calculations implicitly by appropriate conditions for validity of theoretical models of cross section or structural member behaviour.

In addition to the basic material properties indicated above in the one dimensional stress σ - strain ε diagram (see Figure 7), other important aspects need to be considered:

(a) multi-axial stress condition (e.g. the Poisson ratio, yield surface, flow and hardening rules, crack creation and crack behaviour);
(b) temperature effects (e.g. coefficient of expansion, effect on material properties including extreme conditions);

(c) time effect (e.g. effect of internal and external influences, creep, creep rupture, consolidation of soils, fatigue deterioration);
(d) dynamic effects (e.g. mass density and material damping, loading rate);
(e) humidity effects (e.g. shrinkage, effect on strength, stiffness and ductility);
(f) effects of notches and flaws (e.g. unstable check growth, brittle fracture, stress intensity factor, effect of ductility and crack geometry, toughness).

Fatigue behaviour

A very important material property is the fatigue resistance of structural materials and members. This time dependent effect of repeated loading of structural members is generally investigated by simplified tests, where the members are subjected to load variations of constant amplitude until excessive deformations or fracture due to cracks occur. The fatigue strength is then defined by characteristic $\Delta\sigma$ - N curves that represent the 5% fractiles of failure. The test evaluation is carried out in accordance with Annex D of EN 1990 and appropriate provisions of EN 1992 to 1999. The characteristic $\Delta\sigma$ - N curves are normally represented in a double logarithmic scale as indicated in Figure 8. The corresponding equation has the form

$$\Delta\sigma^m N = C = \text{constant} \tag{A.1}$$

where $\Delta\sigma$ represents the stress range calculated from the load range using appropriate material and geometrical properties taking into account stress concentration factors; and N corresponds to the number of cycles. In some cases the stress concentration factors are introduced as an explicit coefficient of the nominal stress range.

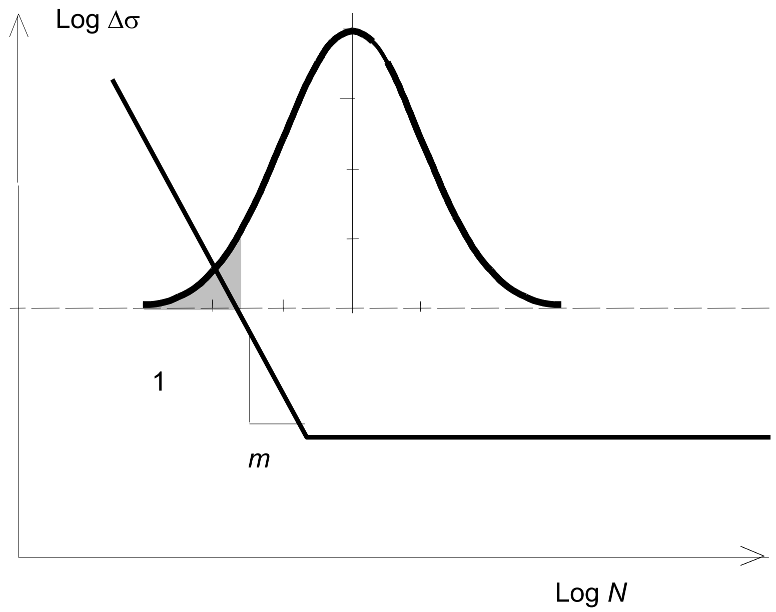

Figure 8. The characteristic Δσ - N curve

Generally, the $\Delta\sigma$ - N curves are dependent on stress concentration and metallurgical aspects. Further details concerning the characteristic $\Delta\sigma$ - N curves for different materials may be found in ENs 1992 to 1999.

APPENDIX B TO CHAPTER VII.
TOLERANCES FOR THE OVERALL IMPERFECTIONS

Completed structures after erection should satisfy criteria specified in Table 1 and Figure 9 to 12. Each criterion shall be considered as a separate requirement to be satisfied independently of any other tolerance criteria.

Table 1. Normal tolerances after execution

Criterion	Permitted deviation
Deviation of distance between adjacent columns	± 5mm
Inclination of a column in a multi-storey building related to storey height *h* (see Figure 9)	$0{,}002h$
Horizontal deviation of column location in a multi-storey building at a floor level Σh from the base, where Σh is the sum of *n* relevant storey heights, relating to a vertical line of the intended column base location (see Figure 10)	$0{,}0035\Sigma h / \sqrt{n}$
Inclination of a column of the height *h* in a single storey building other than a portal frame and not supporting a crane gantry (see Figure 11)	$0{,}0035h$
Inclination of columns of the height *h* in a portal frame not supporting a crane gantry (see Figure 12)	mean $0{,}002h$, individual $0{,}001h$

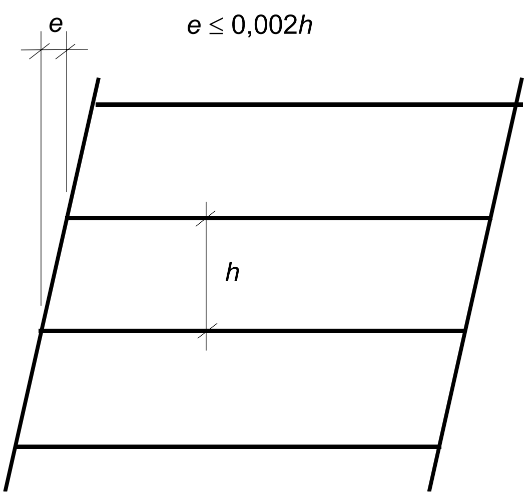

Figure 9. Inclination of a column between adjacent floor levels

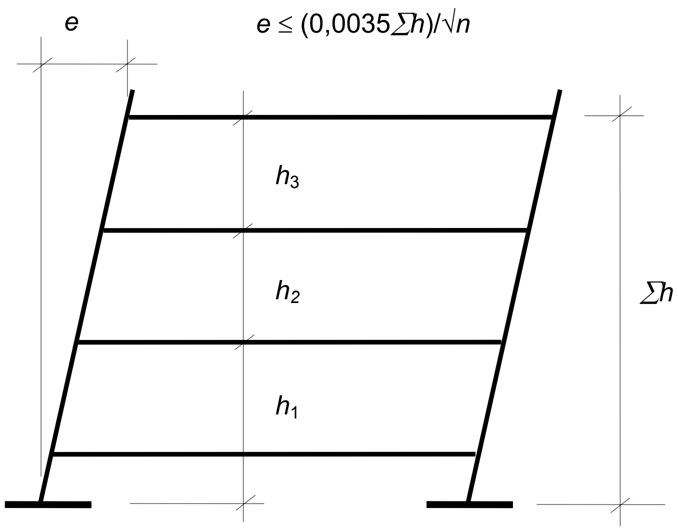

Figure 10. Location of a column at any floor level

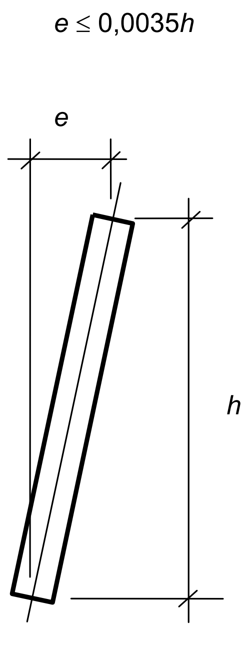

Figure 11.Inclination of a column in a single storey building.

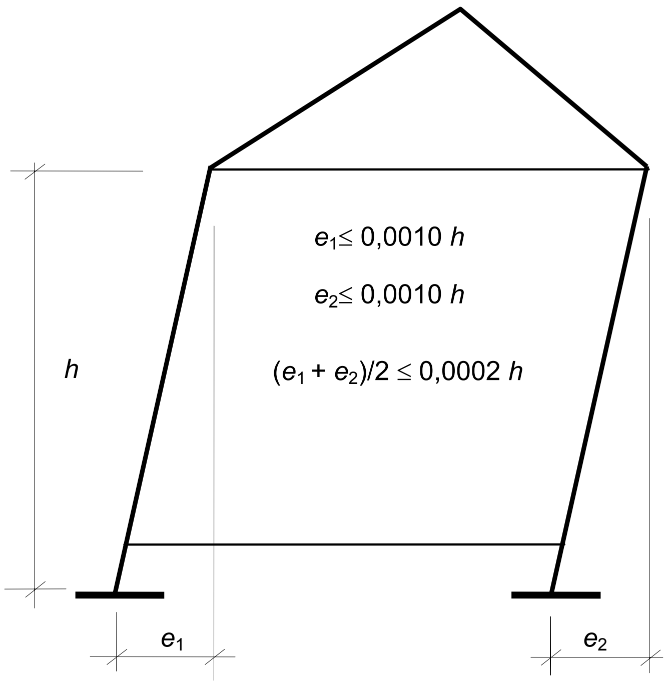

Figure 12. Inclination of columns of a portal frame.

CHAPTER VIII: LOAD COMBINATIONS ACCORDING TO EN 1990

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Summary

Newly available European standard EN 1990 Eurocode - Basis of structural design provides alternative combination rules for actions that are illustrated by examples of analysis of simple building structures. Partial factors for actions and ψ factors used for verification of the ultimate limit states including static equilibrium and for verification of the serviceability limit states are considered in accordance with the recommendations provided in EN 1990. Resulting load effects due to alternative combination rules are presented as bending moment envelops, in the case of static equilibrium also as shear forces. Comparison of obtained action effects indicates that alternative combination rules may lead to considerably diverse load effects differing up to 18 %. It appears that further investigation concerning decision about the preferable alternative, to be made in the National Annexes to EN 1990, should take into account economic, commercial and other aspects including laboriousness of design analysis.

1 INTRODUCTION

1.1 Background documents

The standard EN 1990 Basis of structural design [1] is the fundamental document for the whole system of Eurocodes. The document is available since April 2002 and its implementation into the systems of national standards is expected within one or two years.

EN 1990 [1] provides principles for design and verification of structures with regards to safety, serviceability and durability. The aim of this Section is to illustrate combination rules provided in EN 1990 [1] for both the ultimate limit states and serviceability limit states considering permanent loads and imposed loads in accordance with EN 1991-1-1 [2] and climatic actions due to wind and snow. Alternative combination rules for ultimate limit states are compared using simple examples of structures.

1.2 General principles

Two basic sets of conditions should be considered in accordance to the partial factor method. Particularly it should be verified that the design load effect does not exceed

1) the design resistance of structure in ultimate limit states,

2) the relevant criteria for serviceability limit states.

The general condition of structural reliability with respect to ultimate or serviceability limit states may be expressed by the following inequality

$$E_\mathrm{d} < R_\mathrm{d}, \tag{1}$$

where E_d denotes the design value of the load effect E, R_d the design value of the resistance R.

For the selected design situations and identified limit states, critical load cases should be determined. In accordance with EN 1990 [1], a load case is compatible load arrangements, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions.

2 VERIFICATION OF LIMIT STATES

2.1 Verification of static equilibrium and strength

Four types of ultimate limit states are distinguished in EN 1990 [1], denoted by symbols EQU, STR, GEO and FAT:

- EQU covers loss of static equilibrium of a structure as a rigid body in which:
  - minor variations in the value of the spatial distribution of action from a single source are significant, and
  - the strength of construction materials or ground are generally not governing;
- STR covers internal failure or excessive deformation of the structure or structural members, in which the strength of construction materials of the structure governs;
- GEO covers failure or excessive deformation of the ground in which the strengths of soil and rock are significant in providing resistance,
- FAT covers fatigue failure of the structure or structural elements.

The limit states GEO and FAT are not considered in the three examples introduced in this Chapter.

In the case of ultimate limit state of the type EQU (static equilibrium) the design values E_{d} and R_{d} may be symbolically written as

$$E_{\mathrm{d}} = E_{\mathrm{d,dst}},\ R_{\mathrm{d}} = E_{\mathrm{d,stb}} \tag{2}$$

where $E_{\mathrm{d,dst}}$ denotes the design value of the destabilising actions, $E_{\mathrm{d,stb}}$ denotes the design value of the stabilising actions. The verification of ultimate limit states of type EQU is possible to be expressed as (EN 1990, expression (6.7))

$$E_{\mathrm{d,dst}} < E_{\mathrm{d,stb}} \tag{3}$$

The partial factors for permanent and variable actions are given in EN 1990 [1], Annex A1, Table A1.2(A). This table indicated recommended set of partial factors (for favourable and unfavourable permanent actions, leading and accompanying actions) considering load combination rule given in EN 1990 [1] by expression (6.10) only (see combination rule A in the following Section 3). It should be noted that the other combination rules given in EN 1990 [1] by expressions (6.10a) and (6.10b) are not allowed for verification of static equilibrium.

In addition to the γ values recommended in Note 1 of Table A1.2(A) in EN 1990 [1], an alternative set of γ values described in Note 2 may be allowed by the National Annex. The Note 2 refers to the case when the verification of static equilibrium also involves the resistance of structural members. Application of both these sets of γ values is illustrated in the example 1 described in Section 4.

In the case of the limit state of the type STR (internal failure) the design values E_{d} and R_{d} may be written as

$$E_{\mathrm{d}} = \gamma_{E\mathrm{d}}\,\mathrm{E}(F_{\mathrm{d}}, X_{\mathrm{d}}, a_{\mathrm{d}}),\ R_{\mathrm{d}} = \mathrm{R}(F_{\mathrm{d}}, X_{\mathrm{d}}, a_{\mathrm{d}})/\gamma_{R\mathrm{d}}, \tag{4}$$

where $\gamma_{E\mathrm{d}}$ denotes a partial factor accounting for uncertainties in action effect model E, $\gamma_{R\mathrm{d}}$ denotes a partial factor accounting for uncertainties in the resistance model R, F_{d} denotes design values of actions F, X_{d} design values of material properties X, and a_{d} design values of geometric data a (often equal to the nominal values). Note, that the load effect E generally depends also on material properties X_{d} including strength and stiffness (for example in the cases of indirect actions due to imposed deformations).

2.2 Verification of serviceability limit state

The verification of serviceability limit states, which presently becomes more and more important, is based in common cases (e.g. in evaluating of deflection or crack width) on inequality (EN 1990, expression (6.13))

$$C_\mathrm{d} \geq E_\mathrm{d} \tag{5}$$

where C_d is the serviceability constraint, for example admissible deflection, crack width, local stress or acceleration.

3 COMBINATION OF ACTIONS

3.1 General

To verify structural reliability, the design situations and relevant limit states shall be specified first. Then the load arrangements (the position, magnitude and direction) of free actions and the critical load cases (combination of compatible load arrangements) shall be determined. The critical load cases obviously depend on the type and location of structural shape (column, beam, slab) and on the overall configuration of structur.

Assuming preliminary design of a structure is available (i.e. basic topology and structural materials are proposed) practical procedure to verify structural reliability (strength and serviceability) may follow four steps:

1. Selection of relevant design situations and limit states.
2. Determination of compatible load arrangements and critical load cases.
3. Calculation of design values of action effects for relevant ultimate and serviceability limit states.
4. Verification of structural resistance (for specified reliability conditions).

The detail procedure of the first three steps is illustrated by examples given in this contribution. The last fourth step (verification of structural resistance), which concerns material oriented Eurocodes EN 1992 to EN 1999, is not discussed here.

3.2 Combinations of actions in persistent and transient design situations

Combination of actions in persistent and transient design situations are based on :

– design value of leading variable action,

– design value of accompanying variable action.

The fundamental combination of actions A for ultimate limit states (STR) is given in EN 1990 [1] by expression (6.10):

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} \text{ "+" } \gamma_P\, P \text{ "+" } \gamma_{Q,1}\, Q_{k,1} \text{ "+" } \sum_{i>1} \gamma_{Q,i}\psi_{0,i}\, Q_{k,i} \tag{6}$$

Alternative combination B is composed of two expressions (6.10a), (6.10b) in EN 1990 [1]:

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} \text{ "+" } \gamma_P\, P \text{ "+" } \gamma_{Q,1}\psi_{0,1}\, Q_{k,1} \text{ "+" } \sum_{i>1} \gamma_{Q,i}\psi_{0,i}\, Q_{k,i} \tag{7}$$

$$\sum_{j\geq 1} \xi\, \gamma_{G,j}\, G_{k,j} \text{ "+" } \gamma_P\, P \text{ "+" } \gamma_{Q,1}\, Q_{k,1} \text{ "+" } \sum_{j\geq 1} \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i} \tag{8}$$

Combination C is given by two expressions (6.10a$_\mathrm{mod}$), (6.10b), thus, of expression (8) and modified relationship (7), where only permanent loads are considered:

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} \tag{9}$$

In expression (8) ξ denotes the reduction factor for adverse permanent actions G. Alternative combination of actions should be determined by Member States of CEN in National annex. Standard EN 1990 [1] gives no recommendations with respect to choice of one of these three alternatives. Following examples clearly show that resulting effects of actions, which were determined according to particular approaches, might considerably differ.

Recommended values of partial factors for actions γ and reduction factors ψ are given in EN 1990 [1].

When the alternative combinations B or C are used it may be useful to know which of the twin of expressions (6.10a), (6.10b) or (6.10a$_{\mathrm{mod}}$), (6.10b) is decisive. For example when one permanent action G and two variable actions (e.g. imposed load Q and wind W) are considered only, the limit value of the load ratio $\chi = (Q_\mathrm{k} + W_\mathrm{k})/(G_\mathrm{k} + Q_\mathrm{k} + W_\mathrm{k})$ may be determined [3]. Assuming that G_k, Q_k and W_k denotes the load effects of the characteristic actions (not the actions themselves) then the limit (boundary) value χ_{lim} for combinations B and C are given as

$$\chi_{\mathrm{lim,B}} = \frac{\gamma_G(1-\xi)(1+k)}{\gamma_G(1-\xi)(1+k)+\gamma_Q(a-\psi_Q)+\gamma_W k(b-\psi_W)} \quad \text{for combination B} \tag{10}$$

$$\chi_{\mathrm{lim,C}} = \frac{\gamma_G(1-\xi)(1+k)}{\gamma_G(1-\xi)(1+k)+\gamma_Q a+\gamma_W kb} \quad \text{for combination C} \tag{11}$$

In equations (10) and (11) ξ denotes the reduction coefficient (usually $\xi = 0{,}85$) and $k = W_\mathrm{k}/Q_\mathrm{k}$ is the ratio between variable actions W_k and Q_k. For the ratio $k \leq (1-\psi_Q)/(1-\psi_W)$ the auxiliary parameters $a = 1$ and $b = \psi_W$ (action Q is leading) and for $k > (1-\psi_Q)/(1-\psi_W)$ the parameters $a = \psi_Q$ a $b = 1$ (wind W is leading).

Relationships (10) or (11) may be used to determine which of the twin of expressions (6.10a), (6.10b) or (6.10a$_{\mathrm{mod}}$), (6.10b) is decisive: if $\chi < \chi_{\mathrm{lim}}$ then (6.10a) or (6.10a$_{\mathrm{mod}}$) should be used, if $\chi > \chi_{\mathrm{lim}}$ then (6.10b) should is decisive. Application of equations (10) and (11) is shown in the example described in Section 4.2.

3.3 Combination of actions for accidental and seismic design situations

The load combination for verification of structure in accidental design situation may be symbolically written as

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } A_\mathrm{d} \text{ "+" } (\psi_{1,1} \text{ or } \psi_{1,2}) \sum_{i>1} \psi_{2,i}\, Q_{k,i} \tag{12}$$

The choice between $\psi_{1,1}Q_{\mathrm{k},1}$ or $\psi_{2,1}Q_{\mathrm{k},1}$ depends on the type of accidental design situation (impact, fire or survival after an accidental event or situation). Further guidance may be found in the relevant Parts of EN 1991 to EN 1999.

Combinations of actions for accidental design situations should either

- involve an explicit accidental action A (fire or impact), or
- refer to a situation after an accidental event ($A = 0$).

For fire situations, apart from the temperature effect on the material properties, A_d should represent the design value of the indirect thermal action due to fire.

The load combination for verification of structure in seismic design situation may be symbolically expressed as

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } A_{Ed} \text{ "+" } \sum_{i>1} \psi_{2,i}\, Q_{k,i} \tag{13}$$

where A_{ed} is a seismic action arising due to earthquake ground motions.

3.4 Combination of actions for serviceability limit states

Combinations of actions that should be applied for verification of the serviceability limit states depend on a character of action effects. Three different types of load effects are recognised in EN 1990 [1]: irreversible, reversible and long-term effects. The corresponding load combinations are symbolically written as

a) characteristic combination of actions (EN 1990, expression (6.14))

$$\sum_{j\geq 1} G_{\mathrm{k},j} \text{ "+" } P_{\mathrm{k}} \text{ "+" } Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1} \psi_{0,i} Q_{\mathrm{k},i} \tag{14}$$

normally used for verification of irreversible limit states;

b) frequent combination (EN 1990 (expression (6.15))

$$\sum_{j\geq 1} G_{\mathrm{k},j} \text{ "+" } P_{\mathrm{k}} \text{ "+" } \psi_{1,1} Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1} \psi_{2,i} Q_{\mathrm{k},i} \tag{15}$$

normally used for verification of reversible limit states;

c) quasi-permanent combination (EN 1990 (expression (6.16))

$$\sum_{j\geq 1} G_{\mathrm{k},j} \text{ "+" } P_{\mathrm{k}} \text{ "+" } \sum_{i\geq 1} \psi_{2,i} Q_{\mathrm{k},i} \tag{16}$$

normally used for verification of long term effects, and appearance of the structure, e.g. when creep of concrete is considered.

In accordance with Annex A1 of EN 1990 [1] all partial factors for serviceability limit states are equal to unity. The above mentioned load combinations differ by diverse use of ψ_0, ψ_1 and ψ_2 factors. For example ψ_0 is applied to reduce non-leading actions in the characteristic combinations, ψ_1 and ψ_2 are applied in the frequent combinations and ψ_2 is used in the quasi-permanent combinations. Note that depending on the verified structural property (deflection, crack width) and number of independent actions, each load combination may lead to several load cases. Following examples (analysed using software Amses [4]) show practical applications of above described combination rules.

4 EXAMPLES

4.1 Cantilevered beam

Geometry and material. Cantilevered beam considered in the first example is indicated in Figure 1. The reinforced concrete beam having cross-section 0,30×0,40 m (width × depth) is made of concrete C20/25.

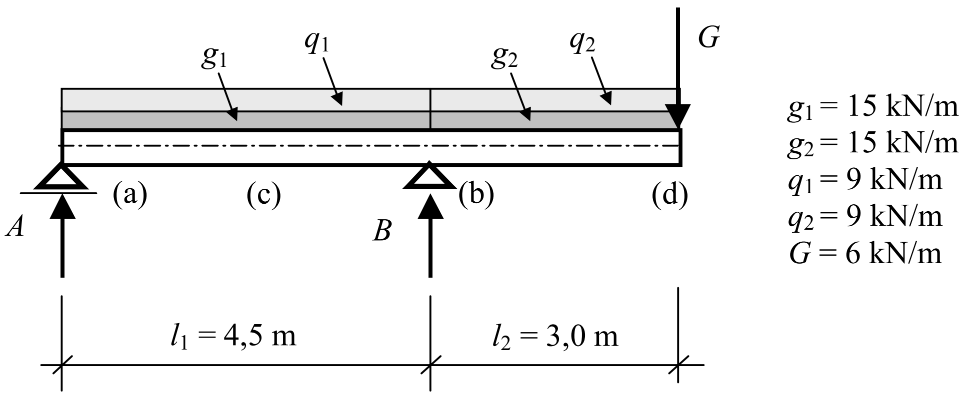


Figure 1. Cantilevered beam.

Uniform permanent load of the beam g_1 and cantilever g_2 (assumed to be from one source or independent), concentrated permanent load G, and imposed loads q_1 and q_2 (Category B - office areas) are considered only. The quantities indicated in Figure 1 denote the charactersiutic values (in order to simplify notation the subscript "k" is left out).

Whether the permanent actions g_1 and g_2 are from one source or not (i.e. they are independent) should be verified considering particular conditions of the structure (weight of structural and nonstructural components acting on both parts of the beam). Nevertheless, it will be shown that mutual independence of g_1 and g_2 is a safe assumption that leads to a considerably greater bending moment in midspan point (c) than the assumption that g_1 and g_2 are from one source.

Ultimate limit states (static equilibrium EQU and limit state of rupture STR), and serviceability limit states (characteristic, frequent and quasi-permanent combinations) are to be verified. Table 1 shows the critical load cases and appropriate factors (γ_G, γ_Q, $\gamma_Q \times \psi$ or $\xi \times \gamma_G$) assuming γ_G = 1,35, γ_Q =1,50, ψ = 0,70 and ξ = 0,85 relevant to verification of the equilibrium, bending resistance (shear is not considered) and deflection of the beam. If the permanent loads g_1 and g_2 could be considered as being from one source, then the factors of both actions would be the same as indicated in Table 1 by the values in brackets (when these are different from the case of independent permanent actions). Note that the assumption of g_1 and g_2 being from one source (and both q_1 and q_2 acting) would lead to the maximum shear forces at point (b) (not shown here).

Table 1. Load cases and factors γ_G, γ_Q, $\gamma_Q \times \psi$ or $\xi \times \gamma_G$ corresponding to relevant expressions in EN 1990 [1] indicated in brackets, if g_1 and g_2 are actions from one source then factors in brackets should be applied.

Load case	Bending moment in *)	Limit state	Factors γ_G, γ_Q, $\gamma_Q \times \psi$ or $\xi \times \gamma_G$ assuming γ_G = 1,35, γ_Q =1,50, ψ = 0,70 and ξ = 0,85 for actions				
			g_1	g_2	q_1	q_2	G
1	-	Equilibrium, exp. (6.7), (6.10)	0,90	1,10	0	1,50	1,10
2	-	Equilibrium, exp. (6.7), (6.10)	1,15	1,35	0	1,50	1,35
3	-	Equilibrium, exp. (6.7), (6.10)	1,00	1,00	0	1,50	1,00
4	(c)	Ultimate, exp. (6.10)	1,35	1,00 (1,35)	1,50	0	1,00
5	(b)	Ultimate, exp. (6.10)	1,00 (1,35)	1,35	0	1,50	1,35
6	(c)	Ultimate, exp. (6.10a)	1,35	1,00 (1,35)	1,50×0,7	0	1,00
7	(c)	Ultimate, exp. (6.10b)	1,15	1,00 (1,15)	1,50	0	1,00
8	(b)	Ultimate, exp. (6.10a)	1,00 (1,35)	1,35	0	1,50×0,7	1,35
9	(b)	Ultimate, exp. (6.10b)	1,00 (1,15)	1,15	0	1,50	1,15
10	(c)	Ultimate, exp. (6.10a$_{mod}$)	1,35	1,00 (1,35)	0	0	1,00
11	(b)	Ultimate, exp. (6.10a$_{mod}$)	1,00 (1,35)	1,35	0	0	1,35
12	-	Serviceability, exp. (6.14)	1,00	1,00	1,00	0	1,00
13	-	Serviceability, exp. (6.14)	1,00	1,00	0	1,00	1,00
14	-	Serviceability, exp. (6.15)	1,00	1,00	1,00×0,5	0	1,00
15	-	Serviceability, exp. (6.15)	1,00	1,00	0	1,00×0,5	1,00
16	-	Serviceability, exp. (6.16)	1,00	1,00	1,00×0,3	0	1,00
17	-	Serviceability, exp. (6.16)	1,00	1,00	0	1,00×0,3	1,00

Note: *) Only the load cases 4 to 11 are directly related to a bending moment in a particular point (and its vicinity) of the beam.

Load effects. If the support (a) of the beam shown in Figure 1 can not transmit tensile forces, static equilibrium EQU of the beam should be checked using equation (3) (expression

(6.7) in EN 1990 [1]). In accordance to this equation the following condition should be satisfied

$$\gamma_{g1}\, g_1\, l_1^2/2 > \gamma_{g2}\, g_2\, l_2^2/2 + \gamma_{q2}\, q_2\, l_2^2/2 + \gamma_G\, G\, l_2$$

As already indicated in Section 2, two alternative sets of partial factors are provided in EN 1990, Annex A1 [1], Table A1.2(A) as illustrated below. Both these sets are independent of the assumption concerning dependency of the permanent actions g_1 and g_2. Thus, the sets of partial factors provided in Table A1.2(A) do not distiquishe between the case when g_1 and g_2 are from one source and the case when g_1 and g_2 should be considered as independent.

In the load case 1 (see Table 1) factors 0,9 for favourable and 1,1 for unfavourable permanent actions are considered (as indicated in Note 1 in Table A1.2(A), Annex A1 of EN 1990 [1]). In the load case 2 (see Table 1) factors 1,15 and 1,35 are used (in accordance with Note 2 in Table A1.2(A), Annex A1 of EN 1990 [1]) provided that applying $\gamma_G = 1$ to both the favourable and the unfavourable parts of permanent actions does not give a more unfavourable load effect (verified in this example 1 by the load case 3, see Table 1).

Figure 2 shows results obtained for the ultimate limit states EQU. It appears that the cantilevered beam should be provided by an anchor at the point (a). The load case 1 seems to be more severe (tensile force 4,03 kN) than load case 2 (tensile force 0,34 kN, not indicated in Figure 2). Note that the load case 3 leads to more favourable effect than the cases 1 and 2 (compressive force 1,25 kN, indicated in Figure 2). Thus, in the alternative approch indicated in the Note 2 in Table A1.2(A), Annex A1 of EN 1990 [1], the load case 2 is decisive.

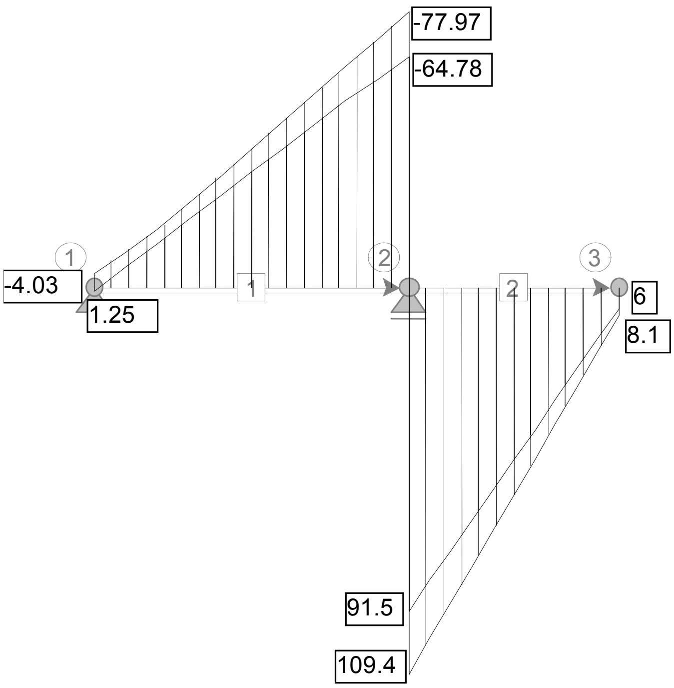


Figure 2. Shear forces [kN] according to expression (6.7) for equilibrium verification.

From the comparison of bending moments for ultimate limit states STR (Figures 3 and 4) it follows that the assumption of independent g_1 and g_2 leads to a considerably greater positive moments (negative moments are not affected) than the assumption of g_1 and g_2 being from one source (for the combination A by more than 20 %, see Figures 3a and 3b). Assuming the independent g_1 and g_2 Figures 3b and 4b indicate that the positive moments for combination A (Figure 3b) are about 18 % greater than those for combination B or C (Figure 4b). The difference between the negative moments of the combinations A and B in point (b) is about 11 %. Load combinations B and C are in this example identical because expression (6.10b) is decisive in both cases while expressions (6.10a) and (6.10a$_{mod}$) are not effective.

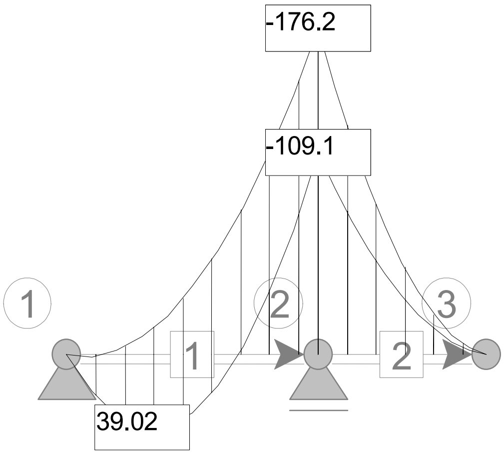


Figure 3a. Bending moment envelopes [kNm] according to expression (6.10) assuming g_1, g_2 being from one source.

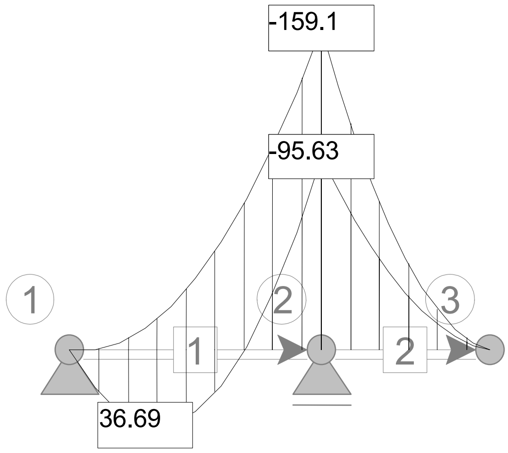


Figure 4a. Bending moment envelopes [kNm] according to exp. (6.10a), (6.10b) and (6.10a$_{mod}$), (6.10b) assuming g_1, g_2 being from one source.

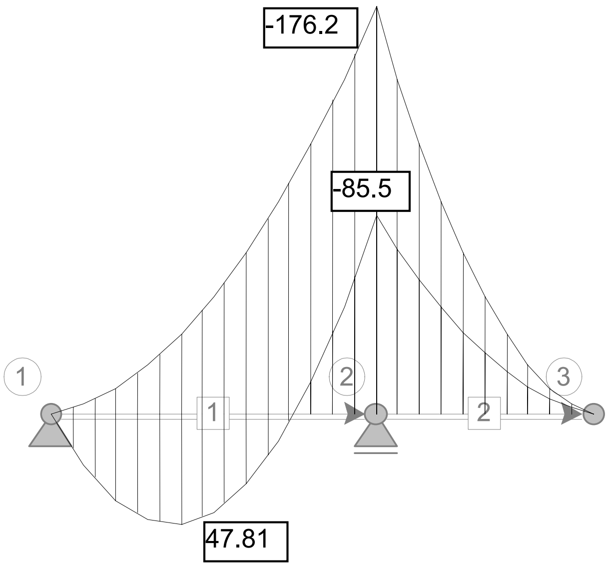


Figure 3b. Bending moment envelopes [kNm] according to expression (6.10) assuming g_1, g_2 independent.

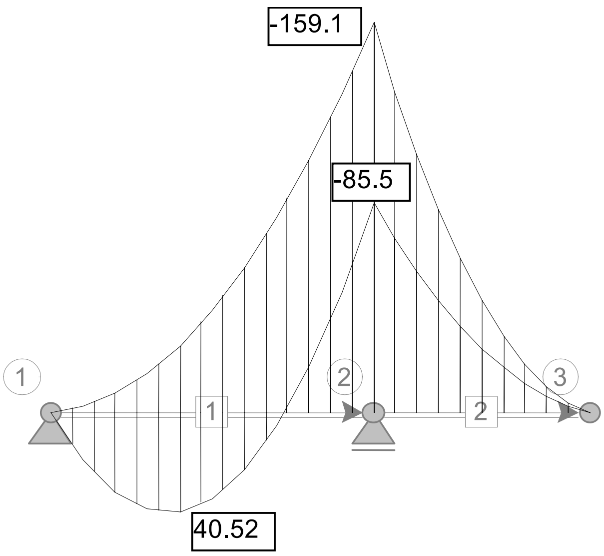


Figure 4b. Bending moment envelopes [kNm] according to exp. (6.10a), (6.10b) and (6.10a$_{mod}$), (6.10b) assuming g_1, g_2 independent.

Deflections. Three combinations (called in EN 1990 [1] characteristic, frequent and quasi-permanent) of serviceability limit states are considered in Table 1. The characteristic load combination is described in EN 1990 [1] by expression (6.14) (load cases 12 and 13), the frequent combination is described in EN 1990 [1] by equation (6.15) (load cases 14 and 15), the quasi-permanent combination described in EN 1990 [1] by expression (6.16) (load cases

16 and 17). Deflection lines and the extreme deflections at a midspan point (c) and at the end point (d) due to characteristic and quasi-permanent load combinations are shown in Figure 5. Deflection lines were determined assuming the modulus of elasticity 29 GPa and creep coefficient 2,5 (in case of quasi-permanent load cases 16 and 17).

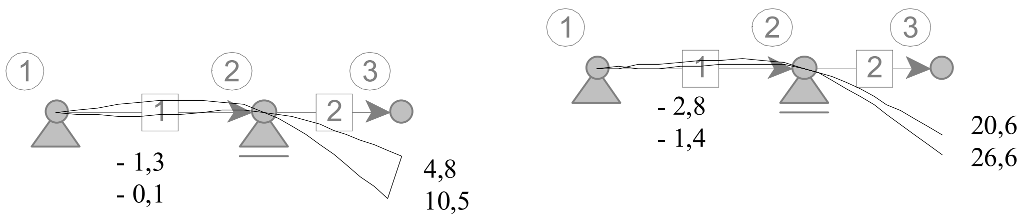


Figure 5. Deflection lines [mm] corresponding to the characteristic load cases 12 and 13 (left) and quasi-permanent cases 16 and 17 (right).

Figure 5 indicates that the deflection at the cantilever end (d) may violate criteria for structural performance. If, for example, the cantilever supports a brittle cladding components, cracks and other performance deficiencies may occur. Note that slightly lower deflection, as that due to characteristic combination, were obtained for the frequent combination described in EN 1990 [1] by equation (6.15) and covered by load cases 14 and 15 (see Table 1).

4.2 Continuous beam of three spans

Geometry and material. Three span continuous beam of the cross-section 0,25×0,40 m made of concrete C 20/25 (modulus of elasticity 29 GPa) is loaded by permanent g (a single action of one origin) and imposed load q as indicated in Figure 6.

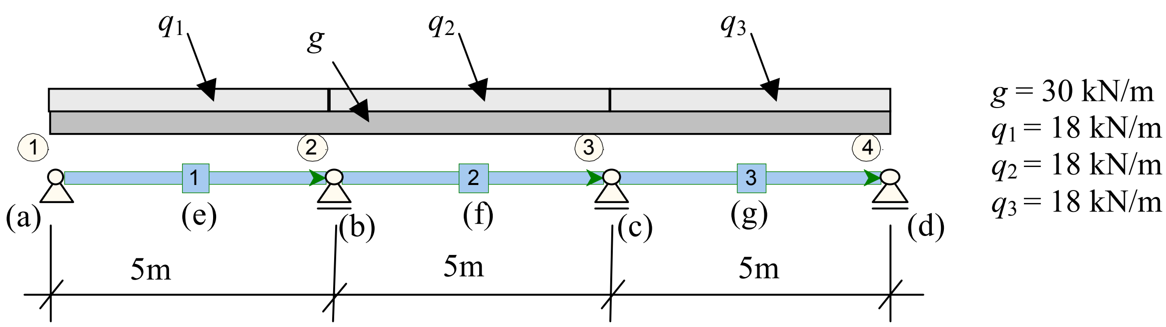


Figure 6. Continuous beam.

Load cases. The uniform permanent action g (considered as a single action from one source for the whole beam) and three independent imposed actions q_1, q_2 and q_3 are considered. In the example the load effects needed for verification of ultimate and serviceability limit states (characteristic and quasi-permanent combinations) are analysed. The quantities indicated in Figure 1 denote the characteristic values (similarly as in example 1 the subscript "k" is omitted).

Ultimate limit states (of the type STR) verified using general expression (6.8) and load combinations (6.10) given in EN 1990 [1] is checked using the total of seventeen load cases, for which appropriate factors γ are indicated in Table 2.

Equations (10) and (11) may be used to identify the decisive expression in load combinations B and C introduced in Section 3.2. For the case of one variable action (imposed load Q only) the relationships (10) and (11) may be simplified as follows:

$$\chi_{\lim,B} = \frac{\gamma_G(1-\xi)}{\gamma_G(1-\xi)+\gamma_Q(a-\psi_Q)}$$

$$\chi_{\lim,C} = \frac{\gamma_G(1-\xi)}{\gamma_G(1-\xi)+\gamma_Q a}$$

Here $a = 1$ (the auxiliary quantity); $\gamma_G = 1{,}35$; $\gamma_G = 1{,}5$; $\psi_Q = 0{,}7$;$\xi = 0{,}85$. The load ratio χ becomes

$$\chi = Q_k/(G_k+Q_k)$$

where Q_k and G_k denote action effects due to the characterstic values of permanent and variable and actions g and q. The following criteria apply for application of twin expressions (6.10a) and (6.10b):

- if $\chi < \chi_{\lim,B}$ or $\chi < \chi_{\lim,C,}$ then expression (6.10a) or (6.10a_{mod}) is to be used,
- if $\chi > \chi_{\lim,B}$ or $\chi > \chi_{\lim,C,}$ then expression (6.10b) is decisive.

Evaluation of these criteria is shown in Table 3.

Table 2. Load cases and factors $\gamma_Q \times \psi$ or $\xi \times \gamma_G$ for continuous beam of three spans, expressions given in EN 1990 are indicated in brackets.

Load case	Bending moment in point *)	Limit state	Factors $\gamma_Q \times \psi$ or $\xi \times \gamma_G$ for actions g	q_1	q_2	q_3
1	(e)	Ultimate, exp. (6.10)	1,35	1,50	-	1,50
2	(f)	Ultimate, exp. (6.10)	1,35	-	1,50	-
3	(b)	Ultimate, exp. (6.10)	1,35	1,50	1,50	-
4	-	Ultimate, exp. (6.10)	1,35	1,50	1,50	1,50
5	(b)	Ultimate, exp. (6.10a)	1,35	0,7×1,50	0,7×1,50	-
6	(b)	Ultimate, exp. (6.10b)	0,85×1,35	1,50	1,50	-
7	(e)	Ultimate, exp. (6.10a)	1,35	0,7×1,50	-	0,7×1,50
8	(e)	Ultimate, exp. (6.10b)	0,85×1,35	1,50	-	1,50
9	-	Ultimate, exp. (6.10a)	1,35	0,7×1,50	0,7×1,50	0,7×1,50
10	-	Ultimate, exp. (6.10b)	0,85×1,35	1,50	1,50	1,50
11	(f)	Ultimate, exp. (6.10a)	1,35	-	0,7×1,50	-
12	(f)	Ultimate, exp. (6.10b)	0,85×1,35	-	1,50	-
13	-	Ultimate, exp. (6.10a_{mod})	1,35	-	-	-
14	-	Serviceability (6.14)	1,00	1,00	-	1,00
15	-	Serviceability (6.14)	1,00	-	1,00	-
16	-	Serviceability (6.16)	1,00	0,3×1,00	-	0,3×1,00
17	-	Serviceability (6.16)	1,00	-	0,3×1,00	-

Note: *) Only some load cases are directly related to a bending moment in a particular point (and its vicinity) of the beam.

Table 3. The limit values $\chi_{\lim,B}$ and $\chi_{\lim,C}$ for load combinations B and C.

M in point: (see Fig. 6)	Moment due to G_k [kNm]	Moment due to Q_k [kNm]	χ	$\chi_{\lim,B}$ eq. (10)	$\chi_{\lim,C}$ eq. (11)	Decisive expression
b	75	52,5	0,412	0,31	0,119	(6.10b)
e	60	42	0,412	0,31	0,119	(6.10b)
f	18,75	33,75	0,643	0,31	0,119	(6.10b)

Thus in the alternative combination rules B and C expression (6.10b) is decisive. Note that the ratio of the characteristic values of the original actions $q/(g+q)$ = 0,375 is considerably different from the ratios of the load effects indicated in Table 3.

Bending moments. The resulting bending moments of the beam are shown in Figures 7 and 8.

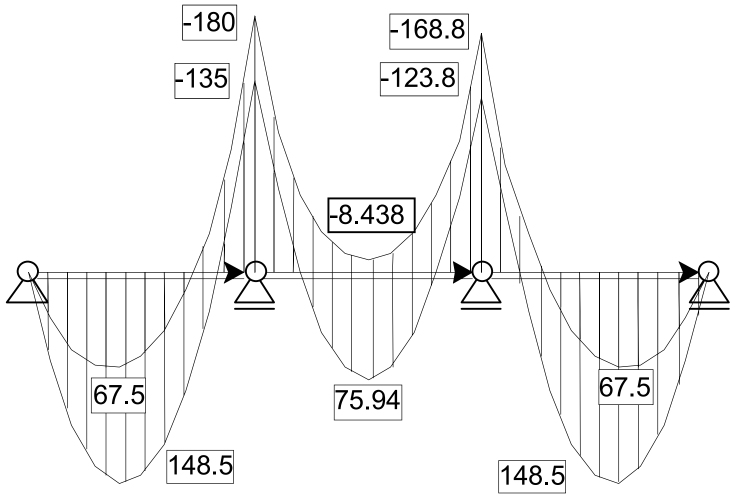


Figure 7. Bending moment envelope [kNm] according to combination A (expression (6.10)).

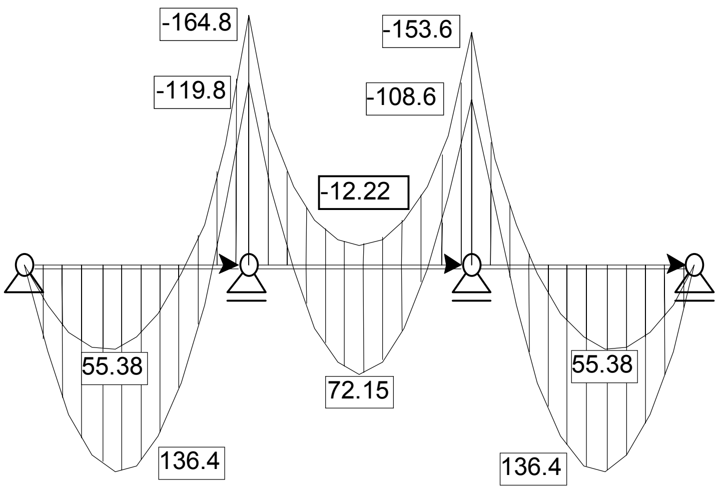


Figure 8. Bending moment envelope [kNm] according to combination B and C (expressions (6.10b)).

The results of analysis considering ultimate limit states STR indicate that the internal moment according to the combination A (Fig. 7) is in points (e) and (b) (see Fig. 6) greater about 11 % than according to the combination B (Fig. 8), resp. C (Fig. 9). The numerical values in point (f) are for the combination A greater about 5 % than according to the combinations B and C. The combinations B and C are also equal in this case, the expressions (6.10a) and (6.10a_{mod}) are not expressed in envelope.

The results of analysis taking into account serviceability limit states are shown in Figures 9 and 10. Considering characteristic load combination, the deflection lines corresponding to the load cases 14 and 15 (indicated in Table 2) are shown in Figure 9. Both deflection lines were determined for the modulus of elasticity 29 GPa (without time dependent creep effect).

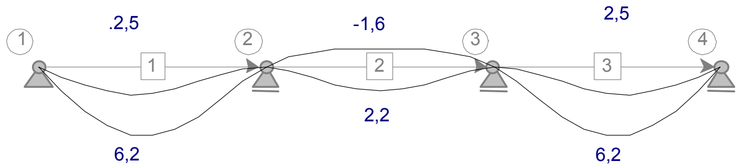


Figure 9. Deflection lines [mm] corresponding to the load cases 14 and 15 specified in Table 2.

Considering quasi-permanent load cases 16 and 17 (see Table 2) the extreme deflection lines are indicated in Figure 10. Both deflection lines were determined for the modulus of elasticity 29 GPa and creep coefficient 2,5.

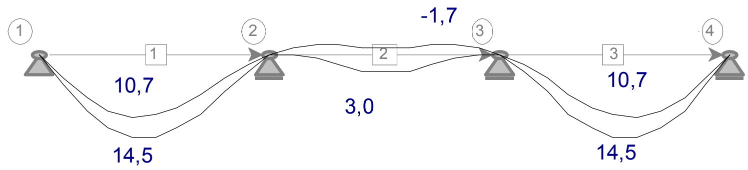


Figure 10. Quasi-permanent deflection lines [mm] corresponding to the load cases 16 and 17 specified in Table 2.

Note that the maximum deflection 14,5 mm is about $L/340$ (where L is the length of one span of the beam), which seems to be quite satisfactory (serviceability constraint $L/250$ is normally considered as sufficient). However, in some cases more detail analysis of deflection may be required taking into account specific conditions (type of reinforcements, creep, performance requirements).

4.3 Cantilevered frame

Geometry and material properties. The cantilevered frame indicated in Figure 11 is exposed to five independent actions: permanent load g, imposed load q_1 and q_2 (Figure 12) and climatic actions due to wind W and snow s (Figure 13). It is assumed that the identical frame is located every 6 m along the longitudinal direction of a building. The total height of the frame is 15 m, foundations are 3 m below the terrain, and the top of the frame is 12 m above the terrain. In a preliminary design of the frame two types of cross-sections are considered:

- columns in the first storey, middle columns in the second to fourth storey, and all beams 0,30 × 0,60 m,
- edge columns of the second to fourth storey 0,30 × 0,30 m.

The frame is made of concrete C 20/25 (modulus of elasticity 29 GPa). A creep coefficient 2,5 is considered when determining long term deflection under quasi-permanent load combination.

The ultimate limit state of structural resistance (STR) and serviceability limit states (characteristic and quasi permanent combination) shall be verified. Note that other actions (imposed load in cantilevered part of the frame only) may be needed when limit state of static equilibrium (EQU) of the frame should be verified (in the considered frame in Figure 11 the limit state of static equilibrium EQU is obviously satisfied).

Load cases. The characteristic value of permanent load g imposed on beams is determined assuming equivalent thickness of the floor slab 0,20 m (representing the slab - about 0,16 m, beams, floor and other permanent loads). Thus, for loading width of 6 m the characteristic value of the uniform load of the beam is

$$g_k = 0{,}20 \times 25 \times 6 = 30 \text{ kN/m}$$

Note that possible reduction factors α_A and α_n, which may be used when designing particular structural elements to reduce imposed load, are not considered here (their effect in this simple example is insignificant).

The characteristic value q_k of imposed load for office areas (3 kN/m^2) and loading width of 6 m is

$$q_k = 3 \times 6 = 18 \text{ kN/m}$$

The characteristic value of wind load is derived assuming the wind speed v = 26 m/s, thus the reference pressure is

$$q_{ref} = 1{,}25 \times v^2/2 = 1{,}25 \times 26^2/2 = 422{,}5 \text{ N/m}^2$$

In addition the following parameters are assumed: the exposure coefficient C_e = 2,5 (corresponding to the height of the structure 12 m above the terrain of category II), the external pressure coefficient $c_{pe,10}$ = 0,8 on the pressure side and the factor $c_{pe,10}$ = –0,3 on the suction side. Thus, for the loading width 6 m and height 3 m (one storey) we get the following pressure force W_{kp} and suction force W_{ks} acting at the frame nodes as indicated in Figure 13:

$$W_{kp} = 0{,}4225 \times 2{,}5 \times 0{,}8 \times 6 \times 3 \cong 15{,}2 \text{ kN}$$

Load effects. In the following load effects for verification of ultimate limit states (STR) and serviceability limit states (characteristic and quasi-permanent combination of actions) are analysed. The total of 16 load cases, indicated in Table 4, are considered. It should be noted that additional load cases might be needed for the verification of the ultimate limit state of equilibrium (EQU), which are not considered here (the imposed load should be considered in the cantilevered part of the frame).

Table 4. Load cases and appropriate factors $\gamma_Q \times \psi$ or $\xi \times \gamma_G$, expressions given in EN 1990 are indicated in brackets.

Load case	Limit state	Factors $\gamma_Q \times \psi$ or $\xi \times \gamma_G$ for actions g	q_1	q_2	W	s
1	Ultimate, exp. (6.10)	1,35	1,50	-	0,6×1,5	0,5×1,50
2	Ultimate, exp. (6.10)	1,35	-	1,50	0,6×1,5	0,5×1,50
3	Ultimate, exp. (6.10)	1,35	1,50	1,50	0,6×1,5	0,5×1,50
4	Ultimate, exp. (6.10)	1,35	0,7×1,50	0,7×1,50	1,50	0,5×1,50
5	Ultimate, exp. (6.10a)	1,35	0,7×1,50	-	0,6×1,5	0,5×1,50
6	Ultimate, exp. (6.10b)	0,85×1,35	1,50	-	0,6×1,5	0,5×1,50
7	Ultimate, exp. (6.10a)	1,35	-	0,7×1,50	0,6×1,5	0,5×1,50
8	Ultimate, exp. (6.10b)	0,85×1,35	-	1,50	0,6×1,5	0,5×1,50
9	Ultimate, exp. (6.10a)	1,35	0,7×1,50	0,7×1,50	0,6×1,5	0,5×1,50
10	Ultimate, exp. (6.10b)	0,85×1,35	1,50	1,50	0,6×1,5	0,5×1,50
11	Ultimate, exp. (6.10b)	0,85×1,35	0,7×1,50	0,7×1,50	1,50	0,5×1,50
12	Ultimate, exp. (6.10a_{mod})	1,35	-	-	-	-
13	Serviceability, exp. (6.14)	1,00	1,00	-	0,6×1,5	0,5×1,50
14	Serviceability, exp. (6.14)	1,00	-	1,00	0,6×1,5	0,5×1,50
15	Serviceability, exp. (6.16)	1,00	0,3×1,00	-	-	-
16	Serviceability, exp. (6.16)	1,00	-	0,3×1,00	-	-

The resulting bending moments envelopes determined using load combination A, B and C are shown in Figures 14 to 16. To achieve better legibility of these figures numerical values of bending moments are indicated for all horizontal beams, and for columns in the lowest floor only.

It follows from Figures 14 to 16 that the bending moments obtained in some cross-sections from combination A (Figure 14) are greater (up to 15 %) than those obtained from combination B (Figure 15) or combination C (Figure 16).

It is interesting to note that in this example the bending moments corresponding to combinations B and C are almost identical. The only exception is the upper horizontal member where the extremes corresponding to combination B (Figure 15) are slightly greater than those corresponding to combination C (Figure 16). This difference is due to the following reason: in the case of combination B, expression (6.10a) is decisive (load case 9), while in the case of combination C, expression (6.10b) is decisive (load case 11). However, in the remaining members of the frame combinations B and C lead to the same bending moments given by load cases 6, 8, 10 and 11, all of them corresponding to expression (6.10b).

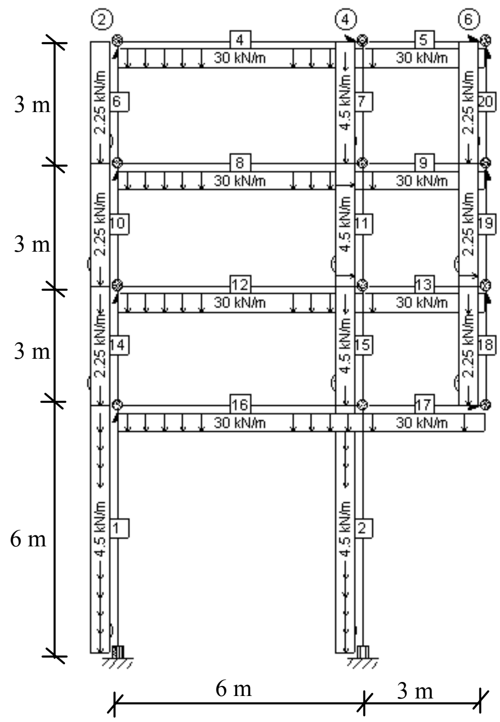
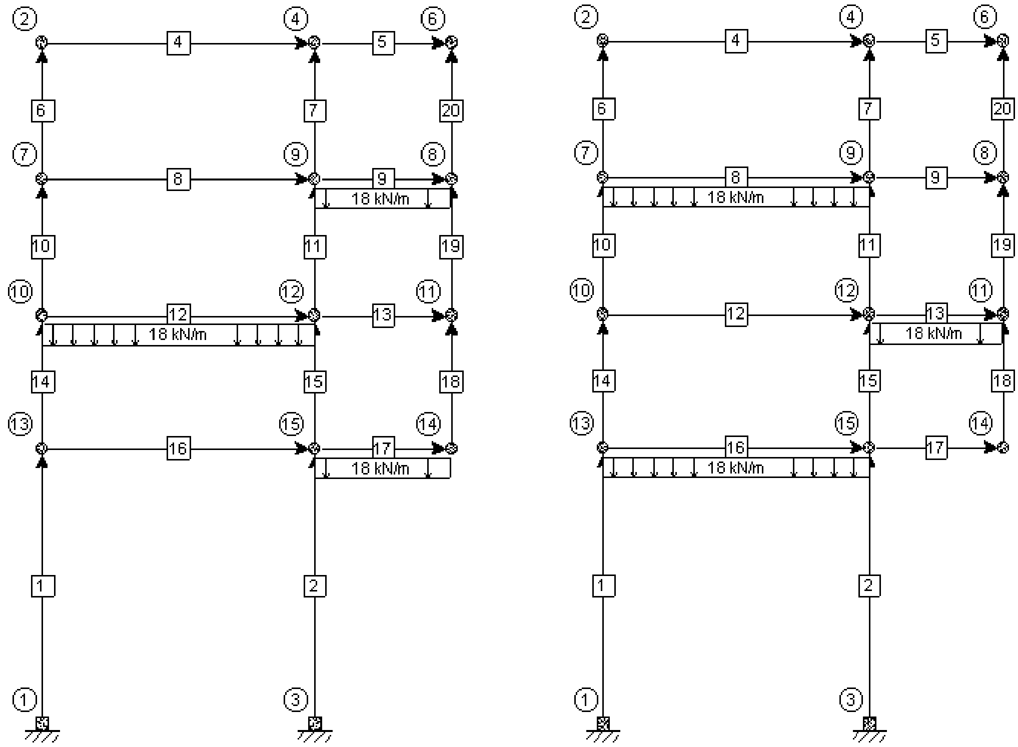
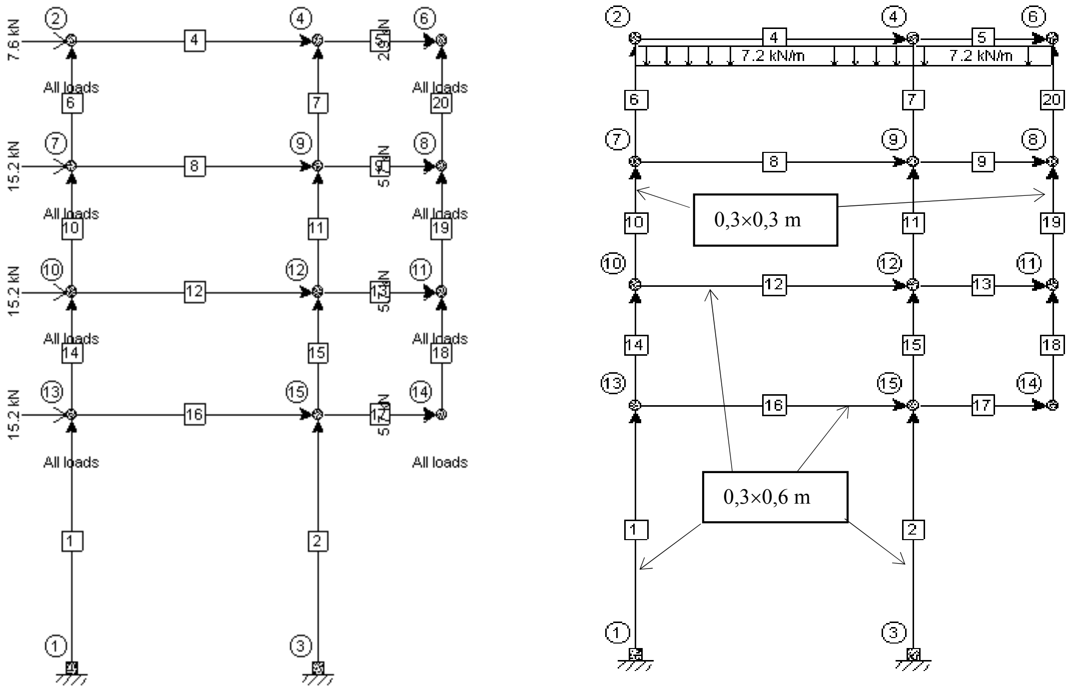


Figure 11. Cantilevered frame - permanent load *g*.



Imposed load q_1 Imposed load q_2

Figure 12. Cantilevered frame – imposed load *q*



Wind actions W Snow loads s

Figure 13. Cantilevered frame – climatic actions

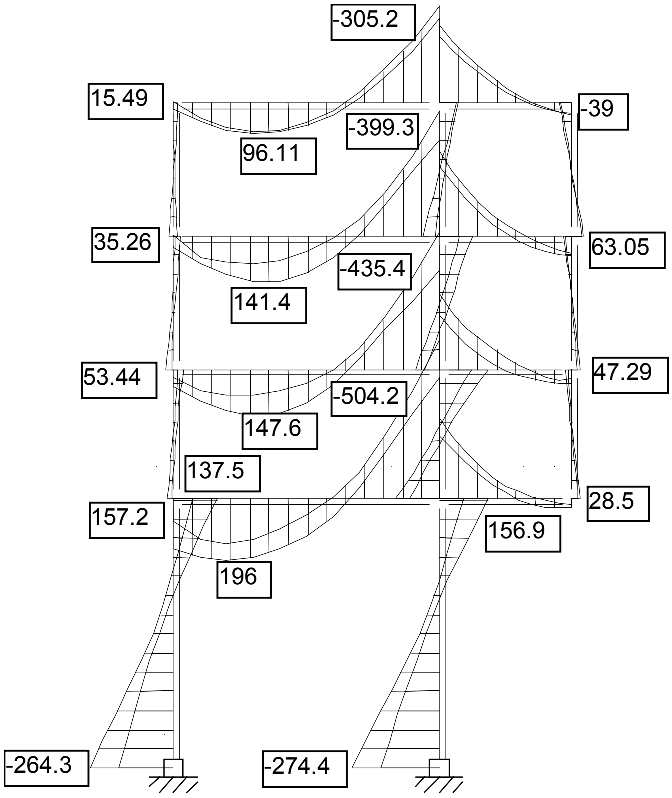


Figure 14. Bending moment envelope [kNm] for combination A (expression (6.10)).

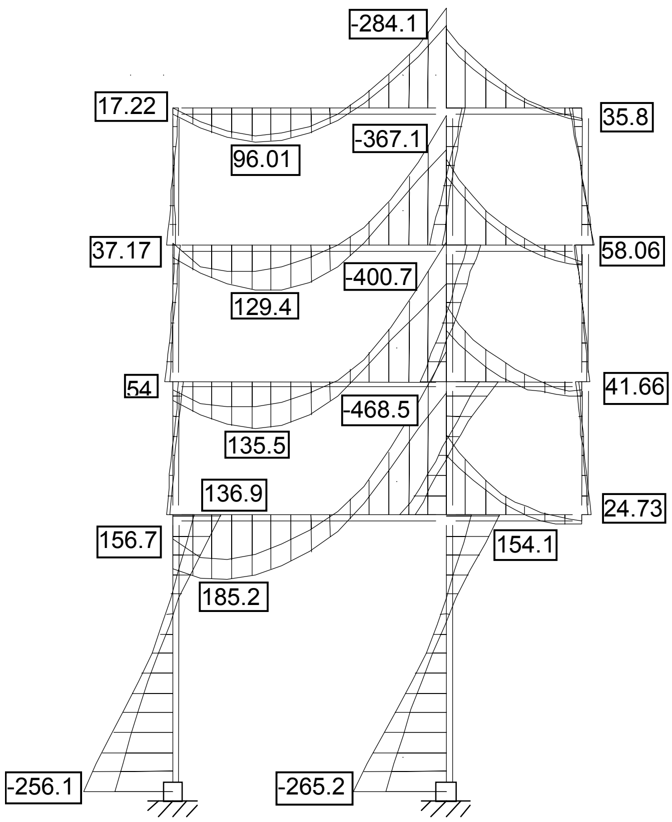


Figure 15. Bending moment envelope [kNm] for combination B (expr. (6.10a),(6.10b)).

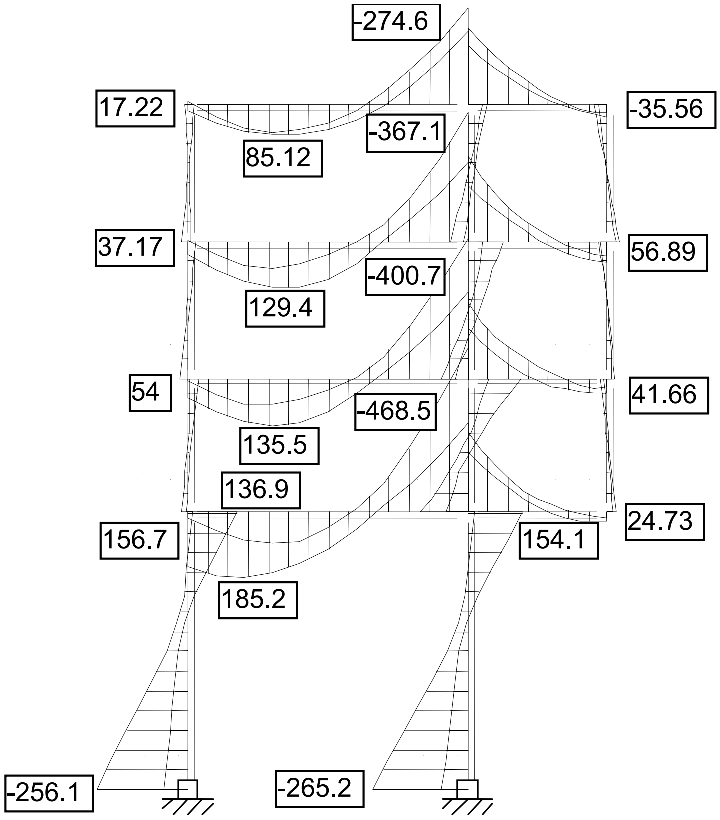


Figure 16. Bending moment envelope [kNm] for combination C (exp. (6.10a$_{mod}$),(6.10b)).

5 CONCLUDING REMARKS

EN 1990 provides two sets of partial safety factors for the ultimate limit states EQU static equilibrium regardless the origin of permament actions (whether they are from one source or not). Results obtained indicate that the partial factors 0,9 for favourable and 1,1 for unfavourable permanent actions lead to more severe load effects than the alternative set of partial factors 1,15 and 1,35.

The examples of selected structural members verified for the limit states of rupture STR indicate that combination rule A (corresponding to expression (6.10) in EN 1990) is easier to apply than combinations B and C (corresponding to twin expressions (6.10a), (6.10b) and (6.10a$_{mod}$), (6.10b) in EN 1990). However, selected examples show that the design procedure A leads to considerable greater load effects (up to 18 % greater) than procedures B and C. Thus, the combination rule A will ensure greater reliability of structures than combinations B and C. Nevertheless, the design procedure A would increase the material consumption compared with the procedures B and C and, therefore, would unfavourably affect the initial costs of structures and the overall economy of the building. On the other hand application of the combination rules B and C might be more complicated than the use of combination A.

It appears, that the decisions concerning recommendation on combination of actions, that are being prepared by national authorities (and will be provided in National annexes to EN 1990), represent demanding tasks. Obviously, in addition to structural reliability several other aspects should be taken into account as well. For example due attention shoud be payed to economical, ecological, and other consequences including laboriousness, time consumption, and transparency of design analysis.

REFERENCES

[1] *EN 1990 Eurocode - Basis of structural design*. European Comittee for Standardisation, 04/2002.
[2] *EN 1991-1-1 Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self weight, imposed loads for buildings*. European Comittee for Standardisation, 04/2002.
[3] *Holický M., Marková J: Reliability of Concrete Elements Designed for Alternative Load Combinations Provided in Eurocodes*, Prague, Acta Polytechnica 2003/1.
[4] Amses Frame 2D, version 1.61, Celja, Slovenia, 2002 (http://www.ainet-sp.si)

APPENDIX A

GUIDANCE PAPER L

APPLICATION AND USE OF EUROCODES

EUROPEAN COMMISSION
ENTERPRISE DIRECTORATE-GENERAL

Single Market : regulatory environment, standardisation and New Approach
Construction

Brussels,
10 April 2003
ENTR/G5 PB

GUIDANCE PAPER L

(concerning the Construction Products Directive - 89/106/EEC)

APPLICATION AND USE OF EUROCODES

(Revision April 2003)

(originally issued following consultation of the Standing Committee on Construction at the 53rd meeting on 19 December 2001 and written procedure ended on 25 January 2002, as document CONSTRUCT 01/483 Rev.1)

Preface

EN Eurocodes can be used to determine the performance of structural components and kits, which are construction products. In that context, EN Eurocodes relate to the Construction Product Directive (89/106/EC)

Furthermore, the Commission considers that the use of EN Eurocodes as the design method for buildings and civil engineering works is the recommended means of giving a presumption of conformity with the essential requirements N°1 and aspects of N°2, in the sense of article 2.1 of the Construction Products Directive

The Member States represented in the Standing Committee on Construction have expressed their opinion and their support by endorsement of this Guidance Paper, which becomes one of the series of Guidance Papers dealing with specific matters related to the implementation of the Directive.

These papers are not legal interpretations of the Directive.

They are not judicially binding and they do not modify or amend the Directive in any way. Where procedures are dealt with, this does not in principle exclude other procedures that may equally satisfy the Directive.

They will be primarily of interest and use to those involved in giving effect to the Directive, from a legal, technical and administrative standpoint.

They may be further elaborated, amended or withdrawn by the same procedure leading to their issue.

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This Guidance Paper "application and use of Eurocodes" has been prepared by the European Commission services in close co-operation with the authorised Representatives of the Member States (Eurocode National Correspondents). The Commission will monitor the matters related to this Guidance Paper. When necessary, the Guidance Paper will be reviewed in the light of the experience made in its application.

Summary

• Abbreviations

CPD	Construction Products Directive (see references)
PPD	Public Procurement Directives (see references)
SCC	Standing Committee on Construction (articles 19 and 20 of the CPD)
ID	Interpretative Documents (article 11 of the CPD)
ENV	European pre-standard
ENV Eurocode	Version of Eurocode published by CEN as a pre-standard ENV (for subsequent conversion into EN)
NAD	National Application Document for the use of ENV Eurocodes at the National level
EN	European standard
EN Eurocode	Version of Eurocode approved by CEN as a European standard
hEN	Harmonised European standard for a construction product (to enable CE Marking)
NDP	Nationally Determined Parameter
DAV	Date of availability of the EN standard
DoW	Date of withdrawal of a conflicting national standards
CEN	Comité Européen de Normalisation (European Standardisation Organisation)
CEN/MC	CEN Management Centre
NSB	National Standards Body (CEN Member)
EOTA	European Organisation for Technical Approval (article 9.2 of the CPD)
ETA	European Technical Approval
ETAG	European Technical Approval Guideline
EEA	European Economic Area
EC	European Commission services

• Definitions

Approval Body	Body authorised to issue European Technical Approvals (Article 10 of the CPD), Member of EOTA)
Boxed Value	The Boxed Value, used at the ENV stage together with the National Application Documents, offered a National choice for a value. It has to disappear in the EN Eurocodes
Construction Works	Building and Civil Engineering Works
European Technical Approval (ETA)	Favourable technical assessment of the fitness for use of a product for an intended use, based on the fulfilment of the Essential Requirements for building works for which the product is used (article 8, 9 and 4.2 of the CPD) An ETA can be issued on the basis of a Guideline (article 9.1 of the CPD) or without guideline (article 9.2 of the CPD)
European Technical Approval Guideline (ETAG)	Document used as the basis for preparing ETAs, which contains specific requirements for the products within the meaning of the Essential Requirements, the test procedures, the methods of assessing and judging the results of the tests, the inspection and conformity procedures, written by EOTA on the base of a mandate received from the Commission (article 9.1 and 11 of the CPD)
National Annex (to an EN Eurocode Part)	Annex to an EN Eurocode Part containing the Nationally Determined Parameters (NDPs) to be used for the structural design of buildings and civil engineering works in a Member State.
National Application Document (NAD)	The NADs, which were used at the ENV stage, expressed national choices, in particular wherever "Boxed Values" (see above) were given in the ENV Eurocodes
National Provisions	National laws, regulations and administrative provisions, imposed by all levels of public authorities, or private bodies acting as a public undertaking or as a public body on the basis of a monopoly position.
Nationally Determined Parameter (NDP)	A National choice left open in a EN Eurocode about values (where symbols are given in the EN Eurocodes), classes or alternative procedures permitted within the EN Eurocodes

Technical Specifications	Harmonised European Standards (hENs) and European Technical Approval (ETAs) for construction products (article 4.1 of the CPD)
Structure	Load-bearing construction, i.e. organised assembly of connected parts designed to provide mechanical resistance and stability to the works (ID 1, clause 2.1.1)
Structural	Relating to a structure
Structural material	Material or constituent product with properties which enter into structural calculations or otherwise relate to the mechanical resistance and stability of works and parts thereof, and/or to their fire resistance, including aspects of durability and serviceability
Structural component	Components to be used as load-bearing part of works designed to provide mechanical resistance and stability to the works and/or fire resistance, including aspects of durability and serviceability, (ID 1, clause 2.1.1).
Structural kit	Kit consisting of structural components to be assembled and installed on site. The assembled system made from the structural kit is a "structure".
Material hEN or ETA	The hEN or ETA for a material or constituent product, with properties which enter into structural calculations of works or otherwise relate to their mechanical resistance and stability and/or fire resistance, including aspects of durability and serviceability, such as concrete, reinforcing steel for concrete, certain structural steel products, fire protection materials.
Component hEN or ETA	hEN or ETA for a prefabricated structural component or a kit consisting of structural components, such as prefabricated concrete components, prefabricated stairs or timber frame building kits, with properties determined by calculation applying methods which are used also for structural design of works.

• References

CPD	Construction Products Directive 89/106/EEC, as amended by CE Marking Directive 93/68/EEC
PPD	Public Procurement Directives. This Guidance Paper refers to the Council Directive 93/37/EEC of 14 June 1993 concerning the co-ordination of procedures for the award of public works contracts
Guidance Paper C	The treatment of kits and systems under the Construction Products Directive (CONSTRUCT 96/175 Rev.2, 3 Feb. 1997 – Rev. Aug 2002)
Guidance Paper D	CE Marking under the CPD (CONSTRUCT 97/220 Rev.5, 10 Dec. 1998 – Rev. Aug 2002)
Guidance Paper E	Levels and classes under the CPD (CONSTRUCT 99/337 Rev.1, 1 Jul 1999 – Rev. Aug 2002)
Guidance Paper F	Durability aspects under the CPD (CONSTRUCT 99/367, 1 Jul 1999 – Rev. Aug 2002)
Guidance Paper J	Transitional Arrangements under the CPD (CONSTRUCT 01/477, 22 May 2001 – Rev. Aug 2002)
Guidance paper K	The attestation of conformity systems and the role and tasks of the notified bodies in the field of the Construction Product Directive (CONSTRUCT 00/421, 5 July 2000 – Rev. Aug 2002)

Part 1: General

1.1 Aims and benefits of the Eurocode programme

1.1.1. The Eurocodes provide common design methods, expressed in a set of European standards, which are intended to be used as reference documents for Member States to:

- prove the compliance of building and civil engineering works or parts thereof with Essential Requirement n°1 Mechanical resistance and stability (including such aspects of Essential Requirement n°4 Safety in use, which relate to mechanical resistance and stability) and a part of Essential Requirement n°2 Safety in case of fire, including durability, as defined in Annex 1 of the CPD

- express in technical terms , these Essential Requirements applicable to the works and parts thereof;

- determine the performance of structural components and kits with regard to mechanical resistance and stability and resistance to fire, insofar as it is part of the information accompanying CE marking (e.g. declared values).

1.1.2. EN Eurocodes are intended by the European Commission services, and the Member States, to become the European recommended means for the structural design of works and parts thereof, to facilitate the exchange of construction services (construction works and related engineering services) and to improve the functioning of the internal market.

In approving the mandate to CEN to prepare the EN Eurocodes, Member States have recognised Eurocodes as an acceptable means to achieve these aims and to prove compliance of construction works with the respective Essential requirements, in their territory. However, following the spirit of the new approach, Members States may recognise also other means as being acceptable for these purposes (see 2.1.7).

The Commission expects CEN to publish all of the standards[1] constituting the different parts of the EN Eurocodes, and expects the Member States to implement these standards as an acceptable means for the design of works, in their territory.

1.1.3 The intended benefits and opportunities of Eurocodes are to:

[1] At present the program contains 58 Parts

- provide common design criteria and methods to fulfil the specified requirements for mechanical resistance, stability and resistance to fire, including aspects of durability and economy,
- provide a common understanding regarding the design of structures between owners, operators and users, designers, contractors and manufacturers of construction products
- facilitate the exchange of construction services between Members States,
- facilitate the marketing and use of structural components and kits in Members States,
- facilitate the marketing and use of materials and constituent products, the properties of which enter into design calculations, in Members States,
- be a common basis for research and development, in the construction sector,
- allow the preparation of common design aids and software,
- increase the competitiveness of the European civil engineering firms, contractors, designers and product manufacturers in their world-wide activities.

1.2 Background of the Eurocode programme

1.2.1. In 1975, the Commission of the European Community decided on an action programme in the field of construction based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

1.2.2. Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the structural design of construction works which, in the first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

1.2.3. For fifteen years, the Commission, with the help of a Steering Committee containing Representatives of Member States, conducted the development of the Eurocodes programme, which led to the publication of a set of first generation European codes in the 80's.

1.2.4. In 1989, the Commission and the Member States decided, on the basis of an agreement with CEN[2], endorsed by the SCC, to transfer the preparation and the publication of the Eurocodes to CEN through a Mandate, in order that they would, in the future, have the status of European Standards.

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[2] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (CONSTRUCT 89/019).

Note: This links the Eurocodes with the provisions of the Council's Directives and Commission's Decisions dealing with European standards (e.g. the CPD and Public Procurement Directives initiated to assist with setting up the internal market).

1.2.5. Originally, the Eurocodes were elaborated by CEN as 62 pre-standards (ENVs). Most were published between 1992 and 1998, but, due to difficulties in harmonizing all the aspects of the calculation methods, the ENV Eurocodes included "boxed values" which allowed Members States to chose other values for use on their territory. National Application Documents, which gave the details of how to apply ENV Eurocodes in Member States, were, generally, issued with a country's ENV.

The conversion of ENVs into European standards started in 1998. Publication of the EN Eurocode Parts is expected between 2002 and 2006.

1.2.6. The Eurocodes, insofar as they concern construction works, have a direct relationship with Interpretative Documents[3], referred to in Article 12 of the CPD[4]. Therefore, technical aspects arising from the Eurocodes have to be taken into account by CEN Technical Committees, EOTA Working Groups and EOTA Bodies working on product specifications, with a view to achieving full compatibility between the product specifications and the EN Eurocodes.

1.2.7. The European Commission has supported, from the beginning, the elaboration of Eurocodes, and contributed to the funding of their drafting. It continues to support the task mandated to CEN to achieve the publication of EN Eurocodes. It will watch the implementation and use of the EN Eurocodes in the Member States.

1.3 Objectives of the Guidance Paper

1.3.1. This Guidance Paper expresses, with the view of achieving the aims and benefits of the Eurocode programme mentioned in 1.1, the common understanding of the Commission and the Member States on:

- The application of EN Eurocodes in the structural design of works (chapter 2).

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[3] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for hENs and ETAs.

[4] According to Art. 12 of the CPD the interpretative documents shall:

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g methods of calculation and of proof, technical rules for project design, etc.;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approval.

The Eurocodes, de facto, play a similar role in the field of the ER 1 and a part of ER 2.

- The use of EN Eurocodes in harmonised standards and European technical approvals for structural construction products (chapter 3). A distinction is made between:

  – products with properties which enter into structural calculations of works, or otherwise relate to their mechanical resistance and stability, including aspects of durability and serviceability, and which for this reason should be consistent with the assumptions and provisions made in the EN Eurocodes ("structural materials" are the most concerned - see chapter 3.2)

  – products with properties which can directly be determined by methods used for the structural design of works, and thus should be determined according to the EN Eurocode methods (prefabricated "structural components and kits" are the most concerned - see chapter 3.3).

1.3.2. The objectives of this document are to:

- Give guidance on the elaboration, implementation and use of the EN Eurocodes

- Provide, for the writers of EN Eurocodes, the framework in which they will elaborate or finalise the EN Eurocodes on the basis of the existing ENV Eurocodes

- Provide, for the writers of product specifications, the framework in which they will make reference to incorporate, or to take into account, the EN Eurocode Parts in harmonised standards and European technical approvals for structural products as explained in 1.3.1,

- Allow for the inclusion in EN Eurocodes and in technical specifications for structural products the necessary parameters or classes or allowance for levels to enable the Member States to choose the level of safety, durability and economy applicable to construction works, in their territory,

- Provide to Member States and the authorities concerned the elements needed to prepare public contracts, in respect of the Public Procurement Directive

1.3.3. This Guidance Paper considers all the issues and conditions related to the satisfactory implementation of the EN Eurocodes, as well as their links to the implementation of the CPD.

1.3.4. This Guidance Paper is intended for enforcement authorities, regulators, national standards bodies, technical specification writers, notified bodies and industry.

1.3.5. In the context of this Guidance Paper, references to Member States also apply to the European Free Trade Association (EFTA) States, members of the European Economic Area EEA. References to specification writers apply to CEN and CENELEC as well as to EOTA and the EOTA bodies issuing ETAs.

Part 2: Use of EN Eurocodes for structural design of works

2.1 National Provisions for the structural design of works

2.1.1. The determination of the levels of safety[5] of buildings and civil engineering works and parts thereof, including aspects of durability and economy[6], is, and remains, within the competence of the Member States.

2.1.2. Possible differences in geographical or climatic conditions (e.g. wind or snow), or in ways of life, as well as different levels of protection that may prevail at national, regional or local level in the sense of article 3.2 of the CPD[7], will be taken into account, in accordance with Guidance Paper E, by providing choices in the EN Eurocodes for identified values[8], classes[9], or alternative methods[10], to be determined at the national level (named Nationally Determined Parameters). Thus allowing the Member States to choose the level of safety, including aspects of durability and economy, applicable to works in their territory.

2.1.3. When Member States lay down their Nationally Determined Parameters, they should:

– choose from the classes included in the EN Eurocodes, or

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[5] The word safety is encompassed in the Eurocodes in the word reliability

[6] The introductory provisions of Annex I of the CPD lay down: *"The products must be suitable for construction works which (as a whole and in their separate parts) are fit for their intended use, account being taken of economy, and in this connection satisfy the following essential requirements where the works are subject to regulations containing such requirements. Such requirements must, subject to normal maintenance, be satisfied for an economically reasonable working life. The requirements generally concern actions which are foreseeable."* Aspects of economy include aspects of serviceability.

[7] Article 3.2 of the CPD says that for each essential requirement classes may be established in the interpretative documents and the technical specifications (hENs and ETAs) *"in order to take account of possible differences in geographical or climatic conditions or in ways of life as well as different levels of protection that may prevail at national, regional or local level"*. This applies to the Eurocodes in so far as they give concrete form to ER 1 and a part of ER 2.

[8] "*Choices about values"* will be made where symbols are given in the EN Eurocodes in order to identify a value to be determined nationally

[9] Generally, the classes to be envisaged should have the status of *"technical classes"* in the sense of guidance paper E (see articles 4.2, 4.3 and 4.4 of the Guidance paper). *"Regulatory classes*" should only be envisaged in cases in which this is necessary to ensure full implementation in the Member States.

[10] *"Choices about methods"* will be made where alternative methods of calculation are included in the EN Eurocodes which are identified to be chosen nationally

- use the recommended value, or choose a value within the recommended range of values, for a symbol where the EN Eurocodes make a recommendation[11], or
- when alternative methods are given, use the recommended method, where the EN Eurocodes make a recommendation,
- take into account the need for coherence of the Nationally Determined Parameters laid down for the different EN Eurocodes and the various Parts thereof.

Member States are encouraged to co-operate to minimise the number of cases where recommendations for a value or method are not adopted for their nationally determined parameters. By choosing the same values and methods, the Member States will enhance the benefits listed in 1.1.3

2.1.4. The Nationally Determined Parameters laid down in a Member State should be made clearly known to the users of the EN Eurocodes and other parties concerned, including manufacturers.

2.1.5. When the EN Eurocodes are used for the design of construction works, or parts thereof, the Nationally Determined Parameters of the Member State on whose territory the works are located shall be applied.

Note: Any reference to a EN Eurocode design should include the information on which set of Nationally Determined Parameters was used, whether or not the Nationally Determined Parameters that were used correspond to the recommendations given in the EN Eurocodes (see 2.1.3).

2.1.6. National Provisions should avoid replacing any EN Eurocode provisions, e.g. Application Rules, by national rules (codes, standards, regulatory provisions, etc.).

When, however, National Provisions do provide that the designer may – even after the end of the coexistence period - deviate from or not apply the EN Eurocodes or certain provisions thereof (e.g. Application Rules), then the design will not be called “a design according to EN Eurocodes”.

2.1.7. When Eurocode Parts are published as European standards, they will become part of the application of the Public Procurement Directive.

In all cases, technical specifications shall be formulated in public tender enquiries and public contracts by referring to EN Eurocodes, in combination with the Nationally Determined Parameters applicable to the works concerned, apart from the exceptions expressed in article 10.3 (Directive 93/37, article 10.2).

However, in application of the PPD, and following the spirit of the New Approach, the reference to EN Eurocodes is not necessarily the only possible reference allowed in a Public contract.. The PPD foresees the possibility for the procuring entity to accept other proposals, if their equivalence to the EN Eurocodes can be demonstrated by the contractor.

[11] see EN 1991-1.1 – foreword – National standards implementing EN Eurocodes

Consequently, the design of works proposed in response to a Public tender can be prepared according to:

– EN Eurocodes (including NDPs), which give a presumption of conformity with all legal European requirements concerning mechanical resistance and stability, fire resistance and durability, in compliance with the technical specifications required in the contract for the works concerned;

– Other provisions expressing the required technical specification in terms of performance. In this case, the technical specification should be detailed enough to allow tenderers to know the conditions on which the offer can be made and the owner to choose the preferred offer. This applies, in particular, to the use of national codes, as long as Member States maintain their use in parallel with EN Eurocodes (e.g. a Design Code provided by National Provisions), if also specified to be acceptable as an alternative to an EN Eurocode Part by the Public tender.

2.2 Indications to writers of EN Eurocodes

2.2.1. When preparing the EN Eurocodes for the design and execution of works, CEN/TC 250 shall provide for National choices as relevant, in accordance with 2.1.2.

2.2.2. When converting the ENV Eurocodes into EN Eurocodes:

– "Boxed values" which do not relate to safety levels and differences referred to in 2.1.2 should be transformed into unique values.

– "Boxed values" which relate to safety levels and differences referred to in 2.1.2 should be replaced by Nationally Determined Parameters. Where relevant, the possible range for these Parameters should be given for information. “Boxed values“ which have an influence on the level of serviceability or durability should be treated as Nationally Determined Parameters.

Note: This request satisfies the requirement of the Mandate to eliminate the “boxed values” or, where necessary, to transform them into classes.

2.2.3. The EN Eurocodes should be formulated in such a way that they can easily be referred to in hENs, ETAGs and ETAs for construction products, in particular those for structural components and kits. Therefore, reference in EN Eurocodes to other standards should only be made when, and as far as is necessary, technical criteria are to be defined; the references should be unambiguous. In order to prevent ambiguity, the normative text should not contain “open ends” or allow different interpretations. General references should be avoided.

2.2.4. Where EN Eurocodes give technical classes or threshold values (in the sense of Guidance Paper E), it should be made clear that these classes or threshold values are applicable only to the design of works. They may not be relevant for harmonised specifications for structural components or kits, which must have the possibility to include other classes or threshold values, as appropriate, such as those that have been used up to now, for structural components legally placed on the market[12].

2.2.5. The EN Eurocodes should be formulated in such a way that the reader of the ENs will be aware that, by definition, design "according to the EN Eurocodes" means compliance with all of the EN Eurocodes provisions, i.e. Principles and Application Rules, together with the respective Nationally Determined Parameters.

Note: Providing the possibility of deviating from, or not applying the EN Eurocodes or certain provisions thereof (e.g. Application Rules) is not a matter to deal with in the EN Eurocodes themselves, but only for the National Provisions implementing them (see 2.1.6).

2.2.6. The EN Eurocodes should be formulated in such a way that a proper distinction is made between calculation methods and administrative provisions on which the National Annex can give information.

2.2.7. In order to improve the transparency and the applicability of the Eurocodes system, each EN Eurocode Part shall include the full list of the symbols, classes or methods for which a choice or determination at national level is possible (NDPs - see 2.3.3).

2.2.8. No delay or objection should be caused as a result of including fundamental changes or new rules, during the conversion from the ENV to EN, in fields in which there is no, or not sufficient, practical experience in Member States.

2.2.9. References in an EN Eurocode Part to other Parts should, where possible, be made only to the EN version of those parts.

2.2.10. When specifying materials and constituent products in EN Eurocodes, CEN/TC 250 shall take account of the following:

- Materials and constituent products with properties which enter into the calculation of structures (e.g. by characteristic values), or otherwise relate to the mechanical resistance and stability and/or fire resistance of the works, including aspects of their durability, should be specified in EN Eurocodes by reference to the respective product hENs, or ETAs. If an hEN or ETA is not yet available or is not foreseen, see footnote 30 and 34.

- For the transitional period during which hENs or ETAs for materials or constituent products are not available or are not binding (i.e. during the co-existence period), EN Eurocodes should, as far as practicable, give, in an informative part, information regarding the properties of materials and constituent products necessary for the structural design of works, according to the EN Eurocodes, and they should state that the respective material and constituent product specifications may be subject to the National Provisions of the Member State in which the works are located[13].

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[12] This applies e.g. to the concrete cover to reinforcing steel which, according to existing national rules for pre-cast concrete components, may be less than the minimum concrete cover for in situ works according to EN Eurocodes

[13] For as long as references to the respective hEN has not been published in the Official Journal of the European Communities or the letter from the Commission informing Member States on the endorsement of the

2.3 National Annexes of the EN Eurocode Parts

2.3.1. When a Eurocode Part is circulated by CEN for publication as an EN, the final text of the approved EN, according to CEN rules, is made available by CEN Management Centre to CEN members (the NSBs) in the 3 official languages (English, French and German)[14].

Each NSB shall implement this EN as a national standard by publication of an equivalent text (i.e. a version translated into another language) or by endorsement of one of the 3 language versions provided by CEN Management Centre (by attaching an "endorsement sheet"), within the timescale agreed for publication.

The National standard transposing the EN Eurocode Part, when published by a National Standards Body (NSB), will be composed of the EN Eurocode text (which may be preceded by a National title page and by a National Foreword), generally followed by a National Annex.

2.3.2. The National Standards Bodies should normally publish a National Annex, on behalf of and with the agreement of the national competent authorities.

A National Annex is not necessary if an EN Eurocode Part contains no choice open for Nationally Determined Parameters, or if an EN Eurocode Part is not relevant for the Member State (e.g. seismic design for some countries).

<u>Note:</u> As stated by the CEN Rules, the National Annex is not a CEN requirement (a NSB can publish an EN Eurocode Part without one). However, in the context of this Guidance Paper, the National Annex serves for NSBs to publish the Nationally Determined Parameters, which will be essential for design.

2.3.3. The National Annex may contain[15], directly or by reference to specific provisions, information on those parameters which are left open in the Eurocodes for national choice, the Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e:

- values and/or classes where alternatives are given in the EN Eurocode,
- values to be used where a symbol only is given in the EN Eurocode,
- country specific data (geographical, climatic, etc.), e.g. a snow map,
- the procedure to be used where alternative procedures are given in the EN Eurocode,

It may also contain the following:

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respective ETA Guideline has not been sent to Member States and its period of coexistence has not yet ended (for further information see Guidance Paper J).

[14] This step correspond to the DAV – Date of Availability

[15] See EN 1990 and EN 1991 Part 1-1 – Foreword – National standards implementing Eurocodes

- decisions on the application of informative annexes, and,
- reference to non-contradictory complementary information to assist the user in applying the Eurocode.

2.3.4. A National Annex cannot change or modify the content of the EN Eurocode text in any way other than where it indicates that national choices may be made by means of Nationally Determined Parameters.

2.3.5. The National Annex of an EN Eurocode Part will normally be finalised when the safety and economy levels have been considered, i.e. at the end of the period allocated for the establishment of the Nationally Determined Parameters (see Annex A).

2.3.6. If a Member State does not choose any NDPs, the choice of the relevant values (e.g. the recommended value), classes or alternative method will be the responsibility of the designer, taking into account the conditions of the project and the National provisions.

2.3.7. The National Annex has an informative status. The content of a National Annex can be the basis for a national standard, via the NSB, and/or can be referred to in a National Regulation.

2.3.8. The National Annex can be amended, if necessary, according to CEN rules.

2.4 Packages of EN Eurocode Parts

2.4.1. The purpose of defining packages, by grouping Parts of EN Eurocode, is to enable a common date of withdrawal (DoW)[16] for all of the relevant parts that are needed for a particular design. Thus conflicting national standards shall have been withdrawn at the end of the coexistence period, after all of the EN Eurocodes of a package are available, and National Provisions will have been adapted by the end of the National Calibration period, as described in Annex A. Publication of the individual Parts in a Package is likely to occur over a long period of time so that, for many Parts, the coexistence period will be much longer than the minimum given in 2.5.5. When a National standard has a wider scope than the conflicting Eurocode Package, only that part of the National standard whose scope is covered by the Package has to be withdrawn.

When more than one package of EN Eurocodes is likely to be needed for the design of works the dates of withdrawal of the related Packages can be synchronised.

2.4.2. No Parts from EN 1990 or the EN 1991, EN 1997 or EN 1998 series form a package in themselves; those Parts are placed in each of the Packages, as they are material independent.

[16] At the date of withdrawal related to a new standard, all the specifications existing previously in the National collection of standards conflicting with the new standard have to be withdrawn and the national provisions have to be adapted to allow the legitimate use of EN Eurocodes

2.4.3. The list of the EN Eurocode Parts contained in the various Packages for each of the main materials, i.e. concrete, steel, composite concrete and steel, timber, masonry and aluminium, and their respective target dates, will be up dated and made available through the CEN/MC web-site[17] (see Annex C which presents the packages as they are currently foreseen)

2.5 Arrangements for the implementation of EN Eurocodes and period of co-existence with national rules for the structural design of works

2.5.1. The arrangements for the implementation of an EN Eurocode Part include, from the time the final draft[18] of the EN Eurocode is produced by the CEN/TC250, five periods:

Two periods before the date of availability (DAV):

- Examination period.
- CEN process period.

Three periods after the date of availability:

- Translation period,
- National calibration period,
- Coexistence period,

The detailed content of each of the five periods is given in the table and chart in Annex A.

The progress of each EN Eurocode (or package), within these periods, will be provided by CEN/MC on their web-site.

2.5.2. The following basic requirements need to be fulfilled by the EN Eurocode Parts in order to be referred to in the national provisions:

– Calculations executed on the basis of the Eurocode Part, in combination with the Nationally Determined Parameters, shall provide an acceptable level of safety.

– The use of the EN Eurocode Part, in combination with the Nationally Determined Parameters, does not lead to structures that cost significantly more, over their working life[19], than those designed according to National standards or provisions, unless changes in safety have been made and agreed.

2.5.3. The European Commission encourages Member States to implement EN Eurocodes in the framework of their National Provisions. During the coexistence period, the

[17] Address: http://www.cenorm.be/sectors/construction/eurocode.htm

[18] CEN/MC will communicate this date on its web-site

[19] see Interpretative Document 1, clause 1.3.5

construction regulation authorities should accept the use of EN Eurocodes, as an alternative to the previous rules (e.g. National codes, standards or other technical rules included, or referred to, in national provisions) for the design of construction works. Member States are also encouraged to adapt their national provisions to withdraw conflicting national rules before the end of the co-existence period.

2.5.4. When an EN Eurocode Part is made available, the Member States should:

- set officially, before the end of the National calibration period (see Annex A), the Nationally Determined Parameters to be applied on their territory. In the event of any unexpected obstacles to carrying out the calibration of an EN Eurocode Part, the Member State shall inform the Commission, when an extension of the period could be agreed by the SCC.

- adapt, as far as necessary, their National Provisions so that the EN Eurocode Part can be used on their territory:

  – as a means to prove compliance of construction works with the national requirements for "mechanical resistance and stability" and "resistance to fire", in the sense of Annex I of the CPD, and

  – as a basis for specifying contracts for the execution of public construction works and related engineering services. If no NDPs are to be produced for an EN Eurocode Part the co-existence period begins at DAV and ends at DoW. Thus the EN Eurocode is available and any existing national standard is still available, so that both can be used during this period.

At the end of the “coexistence period” of the last EN Eurocode Part of a Package, the Member States should have adapted all their National Provisions which lay down (or refer to) design rules within the scope of the relevant Package.

2.5.5. Owing to the need for operational Packages (as defined in 2.4), the reference to the coexistence period of a Package is defined as the coexistence period of the last Eurocode Part of that Package. In Member States intending to implement EN Eurocodes, the coexistence period of this last part should be three years. After the three years coexistence period of the last EN Eurocode Part of a Package, the whole Package-related former conflicting national standards will be withdrawn, i.e 5 years maximum after DAV[20]. Conflicting National Provisions that would not allow the use of the first parts of a Package should be arranged, in order to allow the legitimate use of those Parts.

2.5.6. In order to increase the overall transparency of the implementation of the EN Eurocodes, the Commission wishes to be informed, by the Member States, of the main phases: translation, national calibration and coexistence Period, for each EN Eurocode Part, and the adaptation of National Provisions.

<u>Note:</u> *the Commission intends to prepare, for this purpose, a “test reporting form” on the basis of the items mentioned in the Annex B.*

[20] It is intended that the end of the coexistence period for each package will be laid down by the Commission after consultation of Member States

Part 3: Use of EN Eurocodes in technical specifications for structural products

This part of the Guidance Paper only deals with such structural products, which are construction products in the sense of the CPD.

3.1 Distinction between specifications for material with properties to be determined by test and specifications for components with properties to be determined by calculation

3.1.1. It follows from the CPD[21] and the Interpretative Documents[22] that there is a need for consistency between the technical specifications for construction products (hEN and ETA) and the technical rules for works.

3.1.2. For construction products, which contribute to the mechanical resistance and stability and/or fire resistance of works, two types of properties are distinguished, according to the validation method:

- Properties to be determined by testing (generally in the case of structural materials and constituent products, such as concrete, reinforcing steel for concrete, fire protection material, etc.), and
- Properties to be determined by calculation following methods, which are also used for the structural design of works (generally for prefabricated structural components and kits, consisting of structural components, such as prefabricated concrete components, prefabricated stairs, timber frame buildings kits, etc.).

For both types of product properties the resulting values are to be "declared" in the information accompanying the CE marking[23] of the product and used in the structural design of works or parts thereof.

3.1.3. For the reference to, or use of, EN Eurocodes in harmonised product specifications a distinction is made in this Part 3 between:

- structural materials and constituent products with properties to be determined by testing, and
- prefabricated structural components and kits consisting of structural components with properties to be calculated according to EN Eurocode methods.

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[21] Article 2.1 and 3.3

[22] clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1

[23] By application of CPD and in conformity with the mandate given by the Commission

3.2 Indications to writers of hENs and ETAs for structural material and constituent products with properties to be determined by testing

3.2.1. For structural materials and constituent products, with properties which enter into structural calculations of works or otherwise relate to their mechanical resistance and stability and/or fire resistance including aspects of durability and serviceability, material hENs and ETAs shall meet the following:

- Material hENs and ETAs shall take the technical requirements of the EN Eurocodes into account so that the assumptions of design according to the EN Eurocodes are met. This applies in particular to the general principles and requirements given in EN 1990, Basis of structural design, e.g. with regard to the definition of values of material or product properties such as the characteristic value[24]

- Material hENs and ETAs will, therefore, have to lay down the methods for determining these properties and to specify the requirements for the factory production control and for the conformity attestation in such a way that each declared value or declared class corresponds, as far as practicable, to a defined statistical confidence (defined fractile and confidence level) and can, for the structural design of works, be taken as the “characteristic value”.

- In order to take into account "possible differences in geographical or climatic conditions or in ways of life, as well as different levels of protection that prevail at national, regional or local level" in the sense of Art. 3.2 of the CPD[7], levels and classes[9] may have to be established in the material hENs and ETAs, in accordance with Guidance Papers E and F, taking into account the established competence of the Member States concerning the levels of safety, including aspects of durability and economy. The Member States may then choose the levels and classes to be observed in their territory.

Note: Harmonised specifications shall not exclude from the market products legally in use in at least one Member State. Therefore, materials hENs or ETAs may include specific provisions deviating from the EN Eurocode provisions, provided that the declared values remain usable for the design of construction works, according to the EN Eurocodes.

3.2.2. When making provisions in material hENs or ETAs which determine the declared values or classes, CEN product TCs and EOTA WGs should be aware that:

- Uncertainties concerning declared values of “structural materials and products” will, in design calculations according to the EN Eurocodes, be allowed for by material partial safety factors,

- The value or class of a property or performance of a “structural material or constituent product”, which is needed in the design of works and parts thereof

[24] EN 1990, § 1.5.4.1 defines the *characteristic values* as *"Value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. This value generally corresponds to a specific fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances"*. However, often, the characteristic value takes also the confidence level into account.

(and is consequently important for the competitiveness of that material or product) will not be the declared characteristic value or class but the design value[25].

- Deciding on the safety factors, including the material partial factors, which are used to determine the design value from the characteristic value[24], remains the responsibility of Member States.

3.2.3. All of the provisions concerning the CE marking and the accompanying information on the properties of a product or material shall be given in the relevant hEN or ETA, in accordance with the mandates and the guidance papers of the Commission.

3.2.4. For material properties needed for the structural design of works, and that are linked to the Essential Requirements, the material hEN or ETA shall provide that all of their values or classes, relevant for the calculation or the design assumptions of the EN Eurocodes, are declared in the information accompanying the CE marking.

If one of those properties, for which values or classes have to be declared, is missing in the mandate, the CEN/TC or EOTA/WG shall inform the Commission so that the corresponding mandate can, if justified, be amended and, if needed, transitional arrangements can be made to enable the hEN, or ETA to be published without delay.

3.2.5. Provisions made in 3.2.1 to 3.2.4 with regard to ETAs shall also be taken into account by EOTA in the preparation of the ETA Guidelines (ETAGs), as appropriate.

3.3 Indications to writers of hENs and ETAs for structural components and kits with properties to be determined according to EN Eurocodes[26]

3.3.1. Introduction

The hENs and ETAs for structural components or kits, hereinafter referred to as “component hENs and ETAs”, shall provide for one, or several, or all[27], of the following methods to determine the properties relating to the essential requirements N°1“mechanical resistance and stability” (including such aspects of Essential Requirement n°4 Safety in use, which relate to mechanical resistance and stability) and aspects of Essential Requirement n°2 “resistance to fire”, to be declared as information accompanying the CE marking:

– Method 1: Indication of geometrical data of the component and of properties of the materials and constituent products used, according to 3.3.2.

[25] According to EN1990, § 1.5.4.2 and 1.6, the *design value* of a material or product property is defined as *“value obtained by dividing the characteristic value by a partial factor γ_m (for material property) or γ_M (for material property also accounting for model uncertainties and dimensional variation) or, in special circumstances, by direct determination”*

[26] Properties of structural components and kits can also be determined by testing. The methods to be applied are those which will be given in the hEN or ETA for the structural component or kit concerned.

[27] For a given product, one or several properties can be subject to one of these methods, and other properties can be subject to another of these methods

– Method 2: Determination of properties by means of the EN Eurocodes (with the results expressed as characteristic values or design values) according to 3.3.3

– Method 3: Reference to design documents of the works or client's order according to 3.3.4.

CE marking and the accompanying documents for such a product shall provide all of the information necessary to use the product in works, or to integrate the product characteristics into the structural design of works or parts thereof.

Products that have declared values determined according to EN Eurocode calculation methods, following the harmonised technical specifications, and that are CE marked on this basis, must be allowed to be placed on the market and used for the purpose for which they are intended in all Member States (see CPD article 6.1).

3.3.2. Method 1

The component hEN or ETA provides that the CE marking shall be accompanied by the following information:

– the geometrical data (dimensions and cross sections, including tolerances) of the structural component or, in the case of kits, of the installed system and the components of the kit, and

– the properties of the materials and constituent products used[28] that are needed to determine, according to the National Provisions, valid in the place of use, or possible use, load-bearing capacities and other properties, including aspects of durability and serviceability, of the structural component (or, in the case of kits, of the assembled system) installed in the works - see 3.3.3 (f)

The adequacy of the respective provisions should be verified in consultation with CEN/TC 250.

It is intended that examples for the application of method 1, and examples of CE marking, developed by product CEN/TCs or EOTA/WGs, will be made publicly available by the Commission services, in their web-site.

3.3.3. Method 2

The component hEN or ETA uses EN Eurocode methods as the means of determining the properties of the structural component or kits relating to the essential requirements "mechanical resistance and stability" or "resistance to fire" in terms of characteristic values or design values, taking into account the following:

3.3.3.1. General

(a) Component hENs, and ETAs shall comply with the principles and requirements given in EN 1990 Basis of structural design e.g. with regard to the definition of

[28] The properties of the materials and constituent products used should be indicated by reference to the respective product specification.

values of material or product properties such as the characteristic value[24] and the design value[25]. Thus component hENs and ETAs will have to:

– define the properties of structural components and kits, which relate to "mechanical resistance and stability" or "resistance to fire" that are to be used in the structural design of works, and

– lay down the methods for determining those properties and specify the requirements for the factory production control and for the conformity attestation,

in such a way that each declared value or declared class corresponds, as far as practicable, to a defined statistical confidence (defined fractile and confidence level) and can, for the structural design of works, be taken as the “characteristic value” or “design value”.

(b) Component hENs, and ETAs shall use the methods given in the specific EN Eurocodes, as far as applicable.

The adequacy of the provisions of components hENs and ETAs concerning the indication of properties related to mechanical resistance and stability and resistance to fire should be verified in consultation with CEN/TC 250.

Nevertheless, harmonised specifications shall not exclude from the market products legally in use in at least one Member State. Therefore, a component hEN or ETA may include specific provisions deviating from the EN Eurocode provisions, provided that the component or, in the case of kits, the assembled system, remains usable for works designed according to EN Eurocodes.

Note: EN Eurocode methods referred to in hENs and ETAs have the same status as a test method described in a supporting standard and referred to in an hEN or ETA. By use of a reference, the respective EN Eurocode clauses become part of the harmonised product specification.

(c) Component hENs and ETAs shall take into account the established competence of the Member States concerning the levels of safety, including aspects of durability and economy[29], and of country specific data related to "differences in geographical or climatic conditions or in ways of life or different levels of protection that prevail at National, regional or local level" in the sense of Art. 3.2 of the CPD[7] For this purpose, appropriate levels and classes[9], which give the possibility of national choices for the respective parameters and which can be referred to in the National Provisions, may have to be given in the component hENs and ETAs, taking into account the relevant Nationally Determined Parameters.

With respect to these levels and classes, Guidance Paper E applies with the provisions concerning threshold levels (section 3; minimum/maximum values), classes of product performance (section 4) and possible National requirements concerning levels of product performances (section 5). As structural components and kits are prefabricated (parts of) works bearing the CE marking according to

[29] which includes the aspects of serviceability in the sense of EN Eurocodes

the CPD, also section 2 of Guidance Paper E applies. The levels and classes should be presented in such a way that the Member States' choice is not predetermined (e.g. by the name given to a certain level or class).

Member States are encouraged to co-operate to minimise the number of classes and levels to be introduced in hENs and ETAs by specification writers for "structural components and kits"

(d) As far as durability is concerned, Guidance Paper F on durability applies also to structural components or kits and their properties related to the Essential Requirements "mechanical resistance and stability" or "resistance to fire". For parameters that have an influence on the durability of the works, the Component hENs and ETAs shall also give the possibility for national choices by means of levels or classes according to Guidance Paper E.

(e) The use of EN Eurocode provisions in component hENs and ETAs taking the Nationally Determined Parameters into account in the component hEN or ETA by appropriate levels and classes, if relevant (see 3.3.3.2, note 2), may be done by:

– Referring, in the component hEN or ETA, to the respective EN Eurocode Part(s) indicating the relevant sections or clauses (this method is preferred), or

– Incorporating the respective EN Eurocode provisions in the component hEN, or ETA, where necessary with appropriate adaptation or simplification,

(f) Component hENs and ETAs should specify the materials and constituent products to be used by referring to the respective product hEN[30] or ETAs (for transitional arrangements, see 3.3.3.3). This applies to any material or constituent product, which is to be considered as a construction product in the sense of the CPD and the properties of which:

– enter into the calculation of properties of the structural component or kit, by the characteristic value, or

– relate indirectly to the mechanical resistance and stability of the works, in particular with regard to durability aspects[31], even if they do not enter into the calculation.

(g) All rules related to the CE Marking and the accompanying information on the properties of structural components or kits must be given, with the details necessary for the application by the manufacturers, in the component hEN or

[30] In specific cases, to be identified by the Commission and Member States, component hENs or ETAs may refer to European product standards which do not, or not yet, have the status of harmonised standard in the sense of the CPD, for instance EN 206 "concrete".

[31] e.g. concrete admixtures, possibly having a negative effect concerning corrosion of reinforcing steel, aggregates possibly leading to alkali-silica reaction, or structural steel which, depending on its composition, could be more or less sensitive to corrosion, or fire protection materials to reduce temperature of structural products

ETA, in application of the mandate given by the Commission and in accordance with Guidance Paper D.

The provisions concerning the "indications to identify the characteristics of the product" and the "guidance to specification writers regarding the identification of product characteristics" (clauses 3.6 and 4 of Guidance Paper D) apply also to properties related to the essential requirements "mechanical resistance and stability" and "resistance to fire". Thus, the hEN or ETA shall provide that the information accompanying the CE marking of a structural component or kit, shall include the levels or classes of the properties related to the essential requirements “mechanical resistance and stability” and “resistance to fire”, expressed in terms of declared values or declared classes, including the design assumptions used by the manufacturer. It will be up to the manufacturer of such prefabricated parts of works to choose, in each case, levels and classes according to the intended use (see 3.3.3.1 (c) and (d) as well as 3.3.3.2).

(h) When making provisions in hENs or ETAs for structural components or kits that determine the declared values or classes, CEN product TCs and EOTA bodies should be aware that:

- the values or classes of performance of the structural component or kit, which are essential for the design of works (and, consequently, for the competitiveness of the structural component or kit) will not be the characteristic values but the design values;
- uncertainties concerning declared values or classes of the CE-marked structural component or kit will, according to the EN Eurocodes (but also according to the prevailing national design rules), be taken into account in calculations of the works by material partial factors applicable to the structural component or, in the case of a kit, to the installed system;
- laying down the material partial factors, applicable to the structural component or, in the case of a kit, to the installed system, remains the responsibility of the Member States.

3.3.3.2. Expression of properties related to “mechanical resistance and stability” and “resistance to fire”

The properties related to "mechanical resistance and stability" and "resistance to fire" and the information accompanying the CE marking should be specified in component hENs or ETAs as simply as possible with regard to the needs of fulfilling the National Provisions. This may be done by expressing the properties in terms of:

(a) characteristic values for strength and other cross section properties from which the load-bearing capacities and other aspects[32] of the structural component (or, in the case of kits, of the assembled system) installed in the works, taking into account the National Provisions, can be calculated, or

[32] for instance *thermal insulation*, for fire separating elements

(b) design values provided that the NDPs applicable to works have been taken into account by appropriate levels and classes, which correspond to sets of NDPs (see 2.1.2 to 2.1.5 and 2.2.2).

Note 1: To express a property of a structural component or kit by the "design value" involves that the set of NDPs, which are applicable to the component or kit in the end use conditions, are expressed in the hEN or ETA in terms of classes.
For this purpose, the classes will be defined in component hEN or ETA by the combination of NDPs applicable in Member States.
Normally, for a given structural component or kit and its intended use:

- *. a number of symbols, classes or alternative methods, which in EN Eurocodes have the status of NDPs, will not be relevant, and*
- *. the relevant NDPs will not always be different from one Member State to the other.*

This means that, in most cases, a reduced number of classes, in the component hEN or ETA will be sufficient to cover the NDPs and the differences of NDPs in the various Member States, applicable to the component or kit.
Note 2: Eventually, in particular cases, it may happen for a given component or kit that there is just one set of NDPs to be taken into account in the component hEN or ETA, which covers the end use conditions in all the Member States.

It is intended that examples for the application of method 2, and examples of CE marking, developed by product CEN/TCs or EOTA/WGs, will be made publicly available by the Commission services, in their web-site.

3.3.3.3. Transitional arrangements

The following transitional arrangements shall be taken into account in the drafting of component hENs or ETAs:

- For the period of time in which the respective EN Eurocodes are not yet available and, thus, cannot be referred to in the Component hEN or ETA or used by manufacturers of the structural component or kit, it is recommended to refer to[33], or to incorporate, as far as practicable, the relevant EN Eurocode provisions, in their latest version in consultation with CEN/TC 250. These provisions shall be replaced by references to the respective EN Eurocodes, when these become available.

- For the period of time in which the relevant material hENs, or ETAs, are not yet available and, thus, cannot be referred to in the component hEN or ETA, or used by manufacturers of structural components or kits, it is recommended to incorporate, as far as practicable, the material or product specification in the component hEN or ETA (preferably in Annexes), in consultation with the respective material TCs/WGs,[34]

Provisions in component hENs or ETAs for such transitional arrangements will be necessary until the co-existence periods relating the respective materials and constituent products have come to their end. For further information on "Transitional

[33] Reference can only be made to documents, which are publicly available.

[34] In most cases, such a preliminary harmonisation of the structural materials or constituent products used will not be practicable. Eventually, further mandates for hENs or ETAGs, or green light for an ETAs without guideline could be provided for by the European Commission.

Arrangements" applicable to hENs and ETAs for materials and constituent products, see Guidance Paper J.

3.3.4. Method 3

For cases in which a structural component or kit is produced in accordance with the design details (drawings, material specifications, etc.) prepared by the designer of the works following the National Provisions, component hENs or ETAs shall provide, where relevant, that the information to accompany the CE marking with regard to the product properties can be given by making reference, in an unambiguous way, to the respective design documents of the works, e.g. using a position number.

For cases in which the producer has designed and produced a structural component or kit following the provisions of the client's order, in accordance with the National Provisions applicable to the works, the component hEN or ETA shall provide, where relevant, that the information to accompany the CE marking with regard to the product properties can be given by making reference, in an unambiguous way, to the drawings and material specifications linked to the client's order.

3.3.5. Attestation of conformity

Concerning the conformity attestation of structural components and kits, as of any other construction product, all of the tests and procedures shall be performed and documented according to the provisions of the CPD to be transposed into the technical specification of the product (see Guidance Paper K, clause 2.4).

Therefore, component hENs or ETAs shall contain the necessary provisions to define the tasks of the manufacturer and the Notified Bodies with regard to the attestation of conformity of the product.

Properties of a structural component or kit, which relate to "mechanical resistance and stability" and "fire safety" and which are determined by calculation, are subject to the procedure of attestation of conformity, as is any other property.

Within the systems of attestation of conformity referred to in Annex III of the CPD, in the case of method 2, the checking of calculations shall be considered as a part of the "initial type testing" of the product.

3.3.6. Application to ETAs

Provisions made in 3.3.2 to 3.3.5 with regard to ETAs should also be taken into account by EOTA in the preparation of the ETA Guidelines (ETAGs), as appropriate.

Part 4: Future actions related to the Eurocode Programme

4.1 Education

4.1.1. To build on the strong pedigree of the EN Eurocodes described above, the Commission recognises the importance of building on this with programmes of education to help the professions to implement the EN Eurocodes.

4.1.2. Aspects of education that need to be covered, include:

– informing and making the profession as a whole aware of the EN Eurocodes

– providing continuing professional development and training to the profession

– encouraging the production of handbooks, design aids, software etc to facilitate the implementation of the EN Eurocodes

– encouraging Universities and Technical Colleges to base their teaching of civil and structural engineering design on the EN Eurocodes

4.1.3 The Commission, in liaison with industry and Member States, will encourage:

– Publication of easily understandable "jargon free" booklets covering the EN Eurocodes;

– The holding of European seminars aimed at the profession as a whole as key EN Eurocodes become available as ENs (e.g. EN 1990:Basis of Design);

– Publication of documents on the adoption of the EN Eurocodes through Government or on behalf of Government

– The holding of meetings organised by professional and industry bodies to inform construction professionals and university teachers, to listen to and discuss their concerns, and to promote the opportunities offered by the EN Eurocodes.

– The arrangement of continuing professional development and training courses

– The development of aids to implementation

4.1.4 Central to any initiatives taken on education is the production of :

– Handbooks, worked examples and background documents;

– Software;

– Guides for everyday structures (e.g. normal buildings) based on the EN Eurocodes

– Publishing companies, software houses and trade organisations will carry out these important activities, mainly as commercial ventures. Encouragement to these bodies can be given by a strong commitment to implementation of the EN Eurocodes both by the EC and the Member States.

4.1.5 Member States should encourage the use of the EN Eurocodes in private contracts, particularly through education and information campaigns, regardless of what may be requested by National provisions.

4.2. Research with regard to EN Eurocodes

4.2.1. The Commission services recognises that, for the Construction sector to remain competitive in the world construction industry, it is essential that the EN Eurocodes, once published, should remain the most up to date, useable International Codes of Practice, meeting the requirements for a profession practising in a competitive environment.

4.2.2. The EN Eurocodes should be able to develop according to the innovative pressures of the market and the progress of scientific knowledge and methods.

4.2.3. The pressures from the market are generated by:

– new material and new products;

– new ways for procurement and execution of works;

– needs for economy whilst maintaining acceptable levels of safety.

The progress of the scientific knowledge and methods are generated by:

– the need to avoid disasters in the area of safety (eg seismic, fire);

– a knowledge of phenomena acquired in other domains (eg aeronautics for wind action);

– the answer to new economic or social needs (eg High Speed Railways, nuclear plants);

– the availability of powerful and widely-distributed tools for calculation (computers and software).

4.2.4. Initiatives for research arise from

– the industry or the users concerned;

– public authorities in charge of safety, economy, scientific development and education (for example, the development of NDPs)

– universities and research organisations experienced from their involvement as third parties.

4.2.5. In many cases there will be a mutual interest for both industry and public authorities (including the European Commission) in research and this should be reflected by agreements on common funding according to the following criteria:

- Industrial and user's sources - the main funding for research whose objectives are short-term benefits or particular advantages for special innovative companies and associated industries and users (e.g. unique verifications and ETA's).

- EC or National public funding - the main funding for research whose objectives are medium to long term benefits for the European construction industry (e.g. for improving technical specifications and design codes, harmonising models for actions and resistances, improving safety aspects).

4.3. Maintenance of EN Eurocodes

4.3.1. The maintenance of the EN Eurocodes is essential; the need for updating, revision and completion is strongly recognisedso that an improved second generation of EN Eurocodes can evolve. However, a period of stability should be observed before embarking on change[35] other than to correct errors.

4.3.2. Maintenance work will involve:

- Reducing open choices (NDPs)

- urgent matters of health and safety;

- correcting errors;

- ensuring the most up to date information is in the EN Eurocodes, recognising recent proven innovations and improvements in construction technology;

- feedback from use of the EN Eurocodes in the various Member States through CEN;

- requests from industrial organisations or public authorities to CEN members for revision.

4.3.3. The organisation of maintenance should start after the receipt of a positive vote on a draft EN Eurocode, a Maintenance Group should be formed by the relevant CEN/TC250 SC to:

- give further consideration of co-ordination items arising from the work of other Project Teams (this is necessary as the various parts of the EN Eurocodes are not being prepared simultaneously);

- provide explanations to questions arising from the use of the EN Eurocode, e.g. on background and interpretation of rules;

- collect comments and requests for amendment;

[35] No revision should be published until after the coexistence period has finished.

- prepare action plans for urgent revision in the case of safety related matters, or future systematic revisions according to the CEN procedure and as decided by CEN/TC250.

4.3.4. The strategy to provide adequate resources to support the maintenance of the EN Eurocodes should be decided by the European Commission, Member States, Industry and CEN seeking to find a balance between:

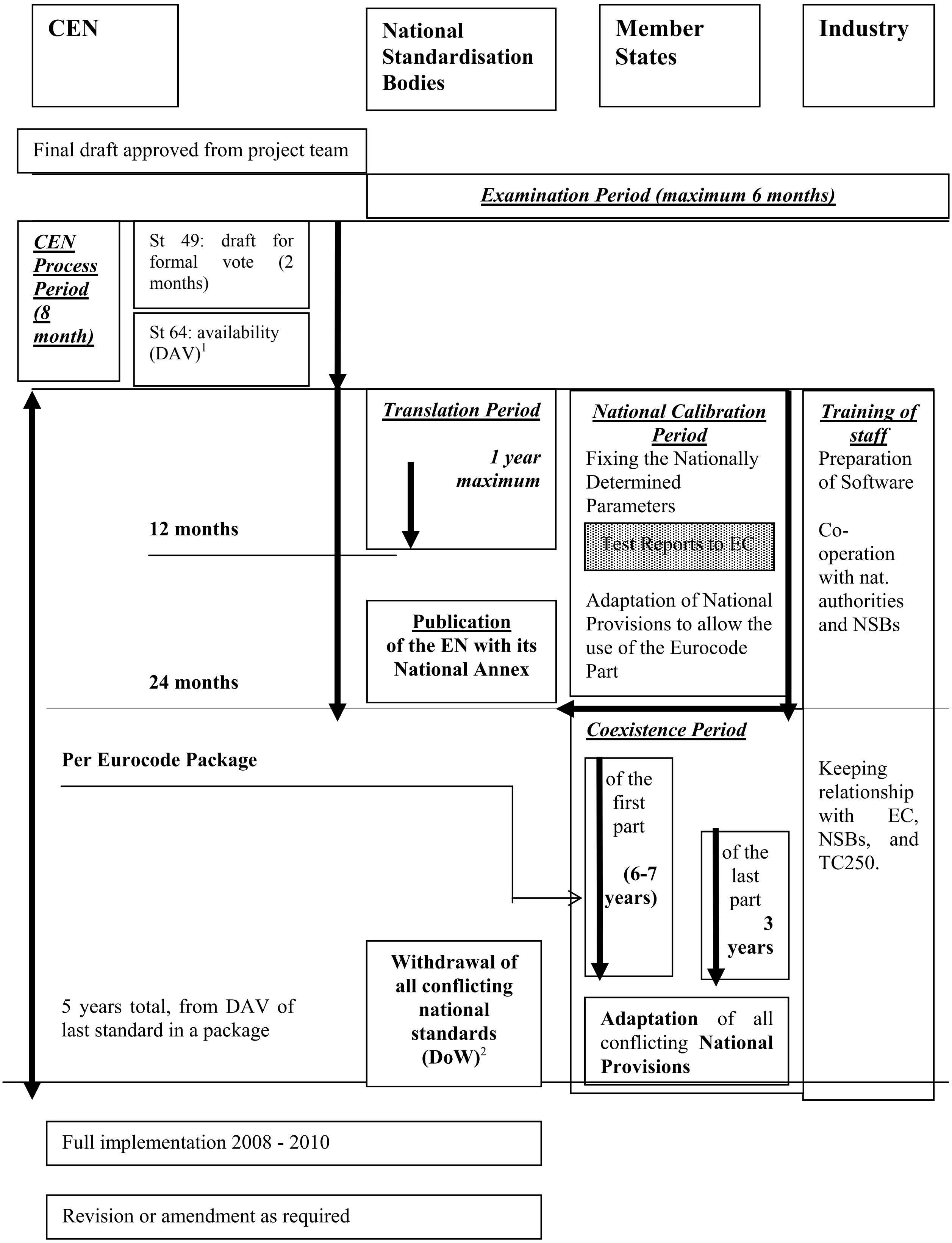
- the requirements for public safety
- the competitive demands of industry
- the availability of funds

Annex A

Arrangements for the implementation of the EN Eurocodes

Periods	Description	Action
Examination Period	After the final draft prepared by the Project Team is sent to the sub-committee for progressing to the vote, a period should be allowed for examination of the content of the Eurocode Part, by both competent authorities and Sub-committee members. After taking into account any comments generated from this examination, the Sub-committee approves the document to go to formal vote and sends it to CEN/MC (CEN stage 49). A maximum period for the examination, revision in the sub-committee and final approval to go to formal vote is 6 months.	CEN/ NSBs
CEN Process Period	After receiving the final draft (CEN stage 49), CEN/MC organises the formal vote and the ratification, leading to the date of availability (DAV) of the approved European standard. This process requires about 8 months, depending on editing, translation (translation of the EN Eurocode Parts to the other two official languages of CEN) and finalisation of the document prior to making it available to CEN members for publication	CEN/ NSBs
Translation Period	The translation of an Eurocode Part in authorised national languages may be started, at the latest when the National Standardisation Bodies have received the Eurocode from CEN (DAV). The maximum time allowed for translation is 12 months after DAV.	NSBs
National Calibration Period (in parallel with translation period)	A period of two (2) years after DAV is the maximum time allowed to fix the Nationally Determined Parameters. The SCC could, however, examine requests, for exceptions. At the end of this period, the national version of an EN Eurocode Part will be published, with the National Annex, which will include the Nationally Determined Parameters. At the end of this 2-year period, the Member States should have adapted their National Provisions so that this Eurocode Part can be used on their territory. The National Annex shall be sent to the EC services for information (see 2.5.6). During this period, the Member States shall inform the Commission about the result of the tests undertaken using this EN Eurocode Part (see 2.5.6 and Annex B).	MSs/ NSBs
Coexistence Period of a Eurocode Package	During the coexistence period, which starts at the end of the National Calibration period, the Eurocode Part can be used, just as the former national system (codes and provisions) can also be used. The coexistence period of an Eurocode Package will last up to a maximum time of three (3) years after the national publication of the last Part of a Package. At the end of the coexistence period of a Package, the NSBs shall withdraw all conflicting national standards, and the Member States shall make sure that all the Parts of the related Package can be used without ambiguity on their territories by adapting their National Provisions as necessary. Thus all conflicting National Standards[36] in a package should be withdrawn a maximum of 5 years after DAV of the last available standard in the package (see 2.5.5)	MS/ NSBs/ Industry

[36] The words "conflicting National Standards" mean standards whose scope covers the same subjects as those of the EN Eurocode Parts



1. DAV = Date of availability
2. DoW = Date of withdrawal of conflicting National Standard

Annex B

Items to be considered for the report on the EN Eurocode trial use

Note: Keep answers as short as possible; do not add the calculations and drawings themselves.

A **Title of the report: Include SUBJECT, MATERIAL, COUNTRY**

B **Basic Information**

Subject of report
Date of report
Author(s)
EN Eurocodes(s) used
Calibration study or design
Any National Code (or ENV Eurocode, with its NAD) used for comparison
Executive summary of work and results obtained

C **Description of the structure(s) designed**

Type of the construction works; is it an existing one, or new build?
Include small-scale figures to illustrate the construction works

D1. **The design (or the checking) of the structure using national codes and standards**

D1.1 The national codes and standards used:

1. Basis for the design
2. Actions
3. Materials

D1.2 Summary of the design checking operations
D1.3 Results

D2. **The design (or the checking) of the structure using EN Eurocodes**

D2.1 Which EN Eurocode Part used? List of NDPs and values or classes or alternatives methods used where NDPs are identified in the EN Eurocode Part.
D2.2 Summary of the design checking operations
D2.3 Results

E **Comparison between the two calculations (if relevant)**

F **Observations on use of EN Eurocodes**

Usability
Understandability
Clarity
Conciseness
Omissions
Level of complexity
Relative time to do calculations compared with National Code
Overall impression of EN Eurocode(s)

Annex C

Packaging of the EN EUROCODE Parts

(According to the actual understanding of CEN)[37]

Eurocode 2: Concrete Structures		
	Package 2/1	Building and Civil Engineering Structures, excluding bridges and liquid retaining and containment structures.
	Package 2/2	Bridges.
	Package 2/3	Liquid retaining and containment structures.
Eurocode 3: Steel Structures		
	Package 3/1	Building and Civil Engineering Structures, excluding bridges, silos, tanks and pipelines, steel piling, crane supporting structures, and towers and masts.
	Package 3/2	Bridges.
	Package 3/3	Silos, tanks and pipelines.
	Package 3/4	Steel piling.
	Package 3/5	Crane supporting structures.
	Package 3/6	Towers and Masts.
Eurocode 4: Composite Steel and Concrete Structures		
	Package 4/1	Building and Civil Engineering Structures, excluding bridges.
	Package 4/2	Bridges.
Eurocode 5: Timber Structures		
	Package 5/1	Buildings and Civil Engineering Structures, excluding bridges.
	Package 5/2	Bridges.
Eurocode 6 : Masonry Structures		
	Package 6/1	Building and Civil Engineering Structures, excluding bridges.
	Package 6/2	Simplified design.
Eurocode 9 : Aluminium		
	Package 9/1	All without fatigue.
	Package 9/2	With fatigue.

- Eurocode Parts from EN 1990, 1991, 1997 and 1998 do not appear as Packages, but are necessary parts of the Eurocode packages for design with particular materials, described above.
- Where a Eurocode Part appears in more than one Package, the DoW for that Part is the same as that for the Package with the DoW furthest in the future.

[37] This list should be up-dated by CEN as appropriate

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58
	Basis	Self load	Fire	Snow load	Wind load	Thermal actions	Const. Load	Accidental	Bridges	Cranes	Silos/Tanks	General Rules	Fire	Bridges	Liquid Retain	General Rules	Fire	Thingauge	Stainless Steel	Plated Struct.	Shells	Out of Plane	Joints	Fatigue	Fractures	Cables	Bridges	Gnl Rules (2)	Silos	Tanks	Pipelines	Piling	Crane	Towers	Chimneys	General Rules	Fire	Bridges	General Rules	Fire	Bridges	General Rules	Fire	Lateral Loads	Special Design	Simplified	General Rules	Lab Tests	Field Tests	General Rules	Bridges	Strength	Silo/Pipe/Tank	Foundations	Towers	General Rules	Fire	Fatigue
<= Packages	**EN 1990**	**EN 1991-1-1**	**EN 1991-1-2**	**EN 1991-1-3**	**EN 1991-1-4**	**EN 1991-1-5**	**EN 1991-1-6**	**EN 1991-1-7**	**EN 1991-2**	**EN 1991-3**	**EN 1991-4**	**EN 1992-1-1**	**EN 1992-1-2**	**EN 1992-2**	**EN 1992-3**	**EN 1993-1-1**	**EN 1993-1-2**	**EN 1993-1-3**	**EN 1993-1-4**	**EN 1993-1-5**	**EN 1993-1-6**	**EN 1993-1-7**	**EN 1993-1-8**	**EN 1993-1-9**	**EN 1993-1-10**	**EN 1993-1-11**	**EN 1993-2**	**EN 1993-3**	**EN 1993-4-1**	**EN 1993-4-2**	**EN 1993-4-3**	**EN 1993-5**	**EN 1993-6**	**EN 1993-7-1**	**EN 1993-7-2**	**EN 1994-1-1**	**EN 1994-1-2**	**EN 1994-2**	**EN 1995-1-1**	**EN 1995-1-2**	**EN 1995-2**	**EN 1996-1-1**	**EN 1996-1-2**	**EN 1996-1-3**	**EN 1996-2**	**EN 1996-3**	**EN 1997-1**	**EN 1997-2**	**EN 1997-3**	**EN 1998-1**	**EN 1998-2**	**EN 1998-3**	**EN 1998-4**	**EN 1998-5**	**EN 1998-6**	**EN 1999-1-1**	**EN 1999-1-2**	**EN 1999-2**
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2/3-Liquid																																																X	X									
Eurocode 3																																																										
3/1-Bldgs																																																X	X									
3/2-Bridg.																																																X	X									
3/3-Silos																																																X	X									
3/4-Piling																																																X	X									
3/5-Crane																																																X	X									
3/6-Towers																																																X	X									
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4/2-Bridg.																																																X	X									
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5/2-Bridg.																																															⊕	⊕	⊕									
Eurocode 6																																																										
6/1-Bldgs																																																X	X									
6/2-Simpl.																																																X	X									
Eurocode 9																																																										
9/1-No Fat.																																																X	X									
9/2-With Fat.																																																X	X									

Latest completion of a Package

X EN 1997 Parts 2 and 3 will be available during the period of National Examination or co-existence of the other parts; their availability Does not however need to affect the start of these periods of National Examination or co-existence

⊕ Foundations for timber structures are designed using packages other than 5/1 or 5/2