Plenary Session 5

Implementation and Evaluation

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Nonlinear Design and an Appropriate Safety Format

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Josef Eibl, born 1936, graduated in Civil Engineering at the University of Munich, doctor's degree at the University of Braunschweig in 1963, several activities in the industry, professor in Braunschweig and Dortmund, since 1982 professor and director of the institute of Concrete Structures and Building Materials at the University of Karlsruhe. Research fields: dynamic loads on RC structures, external prestressing, silos, containments, nonlinear material behaviour.

Summary

Inconsistencies of the nonlinear design concept in EC2 are demonstrated. A safety format based on the comparison of system capacities versus acting loads is proposed. The Probabilistic Finite Element Method (PFEM) is employed to evaluate the safety margin between the mean system capacity and the design load needed for practical engineering applications.

1. Review of the Current ULS-Safety Concept

Prior to the developement of nonlinear analysis techniques in a first step the structural engineer calculated internal section forces and moments applying the fictitious theory of elasticity. Then in a second step the cross-sections were designed for these internal forces and moments using realistic, physically nonlinear constitutive laws for concrete and steel. The safety check was done at the level of cross-sectional characteristics. Consequently two constitutive laws were used simultaneously in one design approach, an elastic one for the first step and a nonlinear, more realistic one for the second step.

The method of nonlinear analysis in Eurocode 2 basically follows the same design format. It demands first a nonlinear evaluation of the internal forces and moments using mean material values and then a cross-sectional design with lower material fractiles. Here two different constitutive laws are also used inconsistently.

This method implies, that at first a rather lengthy and tedious nonlinear computation has to be carried out with estimated values of steel (e.g. over the internal support of a continuous girder) to find the moments and normal forces on the basis of mean material values. At the following cross-sectional design more steel is required for the same cross-section than calculated because of the demand imposed by the lower steel fractile. Thus the former result of the nonlinear analysis is no more than one out of an infinite number of possible equilibrium states. The old inconsistency remains in the new concept.

But there are further reasons why the current concept in EC 2 is not reasonable at all.

- In statically indeterminate structures (hyperstatic beams, plates, shells) failure of a single cross-section usually does not govern the ULS of the system. A local material failure in a

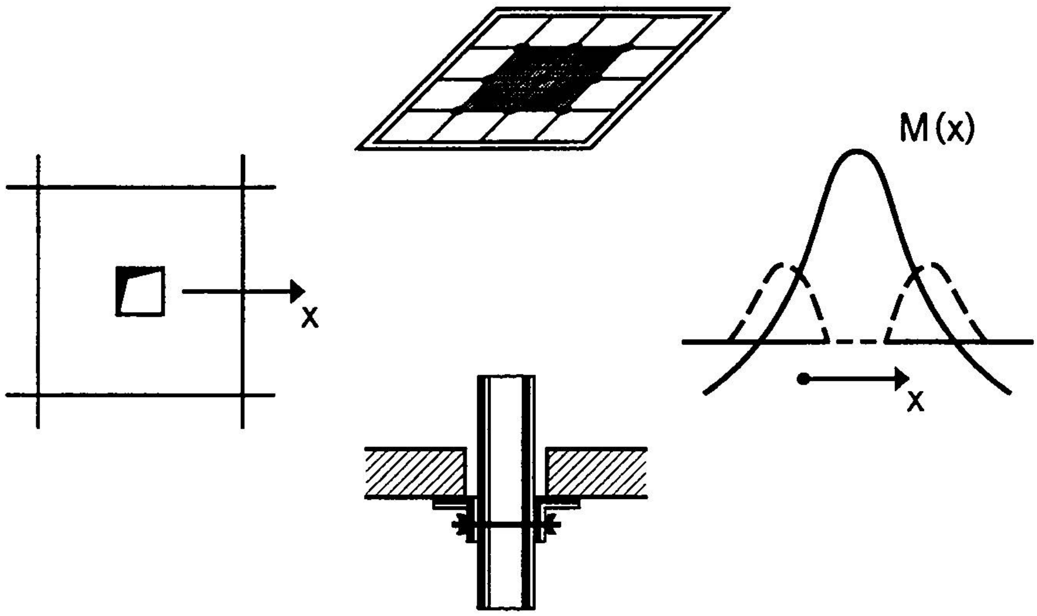

Fig. 1. Flat slab

slab e.g. can easily be absorbed by structural reserves as the example of the liftslab-technique (fig. 1) for the erection of flat slabs demonstrates.

- In case of complex statically indeterminate structures, also in beam systems, an eventual overstrength of the material due to a lower fractile-design in one section may lead to an unsafe result at other sections under different action effects, such as moments, shear and torsion (fig. 2). It is also known from the so called capacity-design in earthquake engineering, that low material values are by no means on the safe side in any case. In complex shell or plate structures it is not even known in advance whether the use of upper or lower fractiles at different locations is on the safe side.

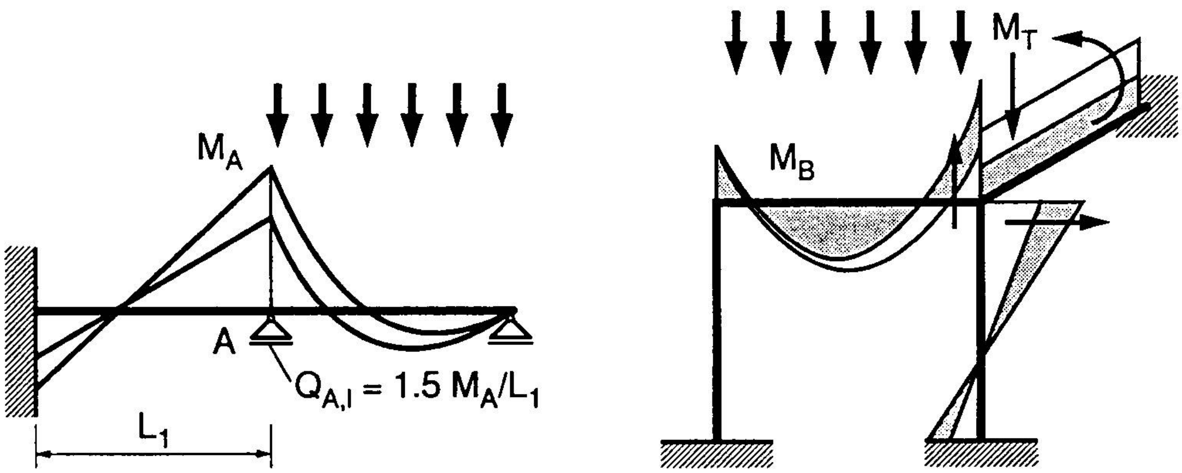

Fig. 2. Shear force and torsion influenced by underestimated flexural strength demonstrated at two systems

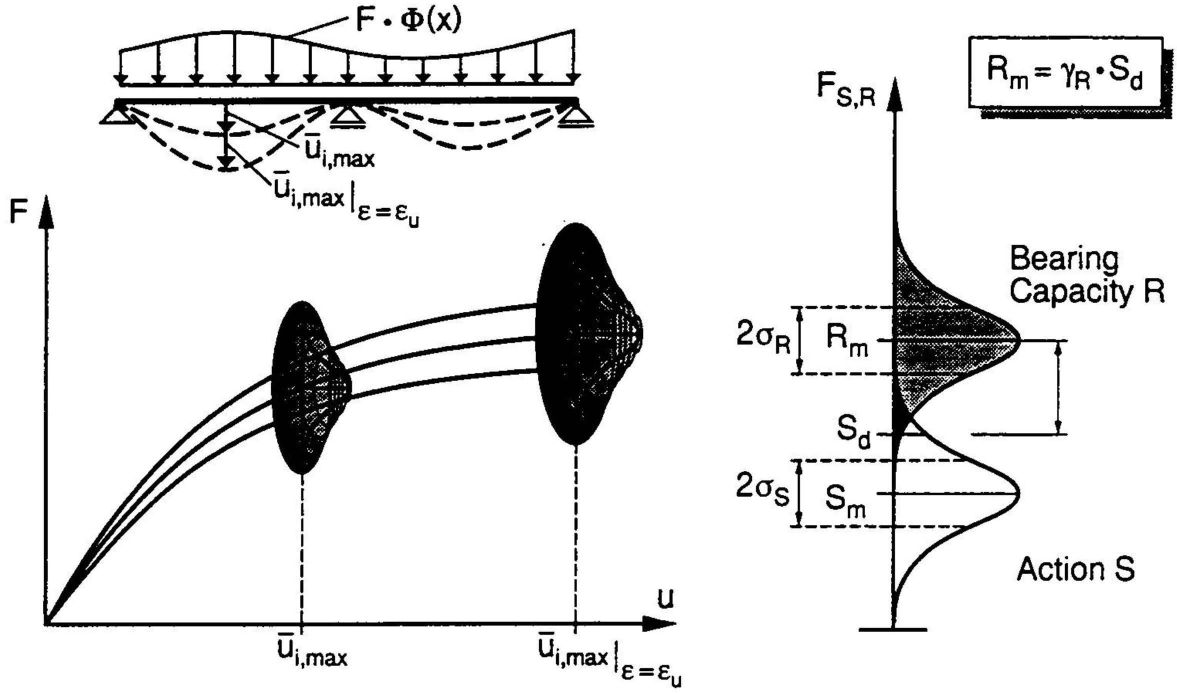

Fig. 3. Stochastic comparison of structural limit force and action force

- Global overstrength can also lead to an unsafe design e.g. in cases of differential settlement, temperature effects or composite structures.

These deficiencies of the safety format in EC2 have been addressed by the author in greater detail in several papers [1], [3], [4].

2. A New Safety Format

What the engineer finally needs at the ULS (Ultimate Limit State) is the realistic load-bearing capacity of the structure and, with regard to a rational safety format, its scatter resp. the density function of the system capacity. Then the probability of failure can be determined using also the density function of the acting loads, both distributions in terms of forces (fig. 3). To this end the system capacity and its scatter have to be calculated using a nonlinear constitutive model that corres-ponds to the real behavior of the structure. Such an analysis automatically includes the redistribution of sectional forces if reserves resp. redundancies are available in the system.

For such a safety check at the level of acting forces and resisting system capacities expressed in terms of forces one single γ-value is enough to guarantee an intended probability of failure, as will be shown. Whether different γ-values are used for different failure modes – steel or concrete failure – or whether different partial safety factors are introduced for the material and the action side is open to discussion. In the latter case both partial safety factors can always be multiplied to give again one global safety factor regarding the source of failure.

Such an approach is possible in detail – with simplifications for daily work – applying the

Probabilistic Finite Element Method ([6], [8]) combined with a special variational approach developed by the author and his coworkers. Details of this method cannot be given here. The reader is referred to [1].

For a better understanding of the available outcome an example is given in the following.

3. Example

The stochastic method addressed in section 2 was implemented into an existing finite element program for beams. The latter had been developed at our institute taking into account the physical nonlinearity of steel and concrete as well as the geometrical nonlinearity on the basis of small displacement theory. For further information it is referred to [2].

The following two-span girder was selected out of several other examples (Concrete C25/30 and Steel S500), its reinforcement chosen so that yielding of the steel governs failure. Therefore only the randomness of the yield stress of steel was considered. The coefficient of variation was assumed to be 10% in a perfectly correlated random field.

Fig. 4 shows the resulting system capacity and its scatter characterized by the mean value of the

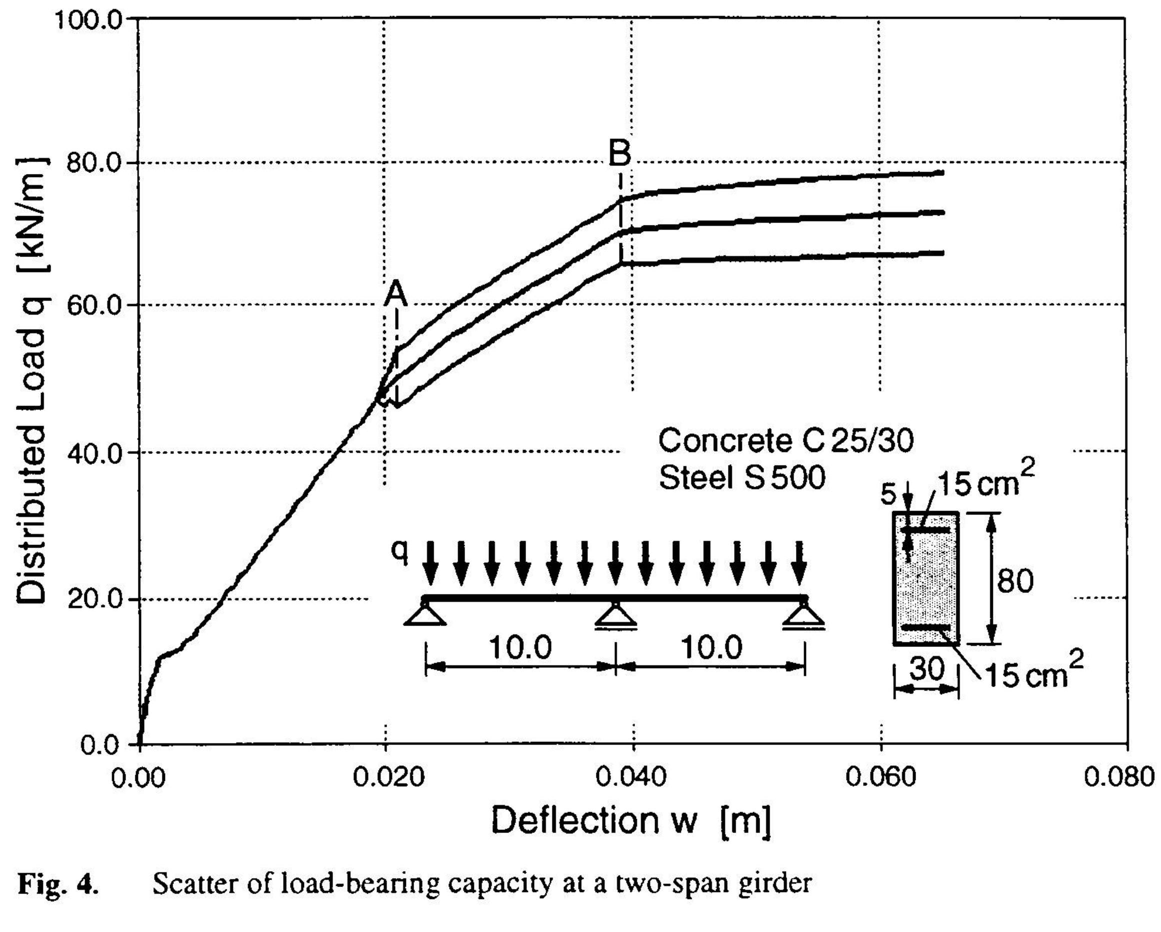

Fig. 4. Scatter of load-bearing capacity at a two-span girder

yield stress and its coefficient of variation $V_R \approx 8\%$.

The redistribution capacity of section forces amounts to about 40% after the first yielding of steel at the internal support between points A and B in fig. 4. The girder fails after reaching the ultimate strain in the compressive zone over the support. Nevertheless failure is initiated by ductile yielding of the reinforcement. Reaching the ultimate compressive strain of concrete is just a secondary effect of the large support rotation.

In table 1 the reliability index β [7] is given for different global safety factors γ_R according to fig. 3 between the mean value of the system capacity and the 99.98%-fractile of the acting load S_d. The latter together with the coefficients of variation for the loads $V = 0.1$ and $V = 0.2$ characterize its distribution function. Both gaussian as well as lognormal distributions for system capacity and load have been studied. A safety factor of $\gamma_R = 1.3$ between design load and the mean ultimate load capacity ensures the required safety level with a reliability index of $\beta = 4.7$ or a probability of failure of $p_f \approx 10^{-6}$ according to [5].

Table 1: Reliability index β: two-span girder

γ_R	β (R, S gaussian)		β (R, S lognormal)	
	$V_S = 0.10$	$V_S = 0.20$	$V_S = 0.10$	$V_S = 0.20$
1.0	2.41	2.92	2.71	3.13
1.3	4.45	4.58	4.78	4.36
1.7	6.31	6.27	6.90	5.61

Summing up, a consistent safety format for non-linear analysis is proposed by further developing the existing EC 2. Major deficiencies of the existing concept are eliminated.

References

[1] Eibl, J., Schmidt-Hurtienne, B.: *General Outline of a New Safety Format*; New Developments in Non-linear Analysis Methods, CEB-Bulletin d'Information Nº 229; August 1995.

[2] Eibl, J., Häußler, U., Retzepis, I.: *Zur numerischen Ermittlung der Spanngliedkräfte bei Vorspannung ohne Verbund*; Bauingenieur 65 (1990), pp. 227-233, Springer-Verlag, Berlin-Heidelberg 1990.

[3] Eibl, J.: *Safety Considerations for Nonlinear Analysis*; IABSE/IVBH-Colloquium "Structural Safety", Stuttgart 1991; IABSE/IVBH-Report Vol. 62 (1991), pp. 337-342.

[4] Eibl, J.: *Nichtlineare Traglastermittlung/Bemessung*, Beton- und Stahlbetonbau 87 (1992), pp. 137-139.

[5] DIN.: *Grundlagen zur Festlegung von Sicherheitsanforderungen für bauliche Anlagen*; Deutsches Institut für Normung e.V., Beuth-Verlag, Berlin-Köln, 1. Aufl. 1981.

[6] Liu, W.K., Belytschko, T., Mani, A..: *Random Field Finite Elements*; International Journal for Numerical Methods in Engineering, Vol. 23, pp. 1831-1845 (1986).

[7] Schuëller, G.I.: *Einführung in die Sicherheit und Zuverlässigkeit von Tragwerken*; Ernst & Sohn, Berlin-München 1981.

[8] Teigen, J.G., Frangopol, D.M. et. al.: *Probabilistic FEM for Nonlinear Concrete Structures*; Journal of Structural Engineering, Vol. 117, No. 9, pp. 2674-2707, Sept. 1991.

Eurocodes, Need or Nuisance ?

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Summary

In this paper a view is given on the role of standards in the building industry.
The question is discussed whether standards are really needed or form, as sometimes stated, a hindrance for the efficient flow of the building process.
Then an overview is given of the total system of harmonised European standards for steel structures, where Eurocodes form part of.
Finally activities are described undertaken to introduce this new system of harmonised standards and in this respect the possible role of modern computer based information transfer technology is discussed.

1. The role of structural standards

Standards currently play an important role in the building process. They contain written agreements on many aspects, so that partners automatically or simply by reference know the conditions. Standards may be considered as the 'rules of the game' for building.
Standardization may concern: administrative and legal conditions.
Standardization may also concern: uniform symbols, standard dimensions and physical properties and classification of products.
The third category concerns the standards relating to quality aspects such as design standards, standards for fabrication and erection and for testing and control. The Eurocodes belong to this last category.
Building regulations are not a modern invention. This may be illustrated by figure 1 showing the oldest known "Building regulation" in the world. If forms part of the code of laws of Hammourabi, King of Babylonia, and is dated 2200 BC.
It contains mainly legal requirements and these are rather simple and straightforward.
For example the first sentence reads: "If a builder builds a home for a man and does not make its construction firm and the house which he has build collapses and causes the death of the owner of the house - that builder shall be put to death".
Although this type of regulations possibly could help to reduce the overcapacity in the building industry, fortunately the modern building regulations are not so extreme. But on the other hand they are also not so simple and compact.

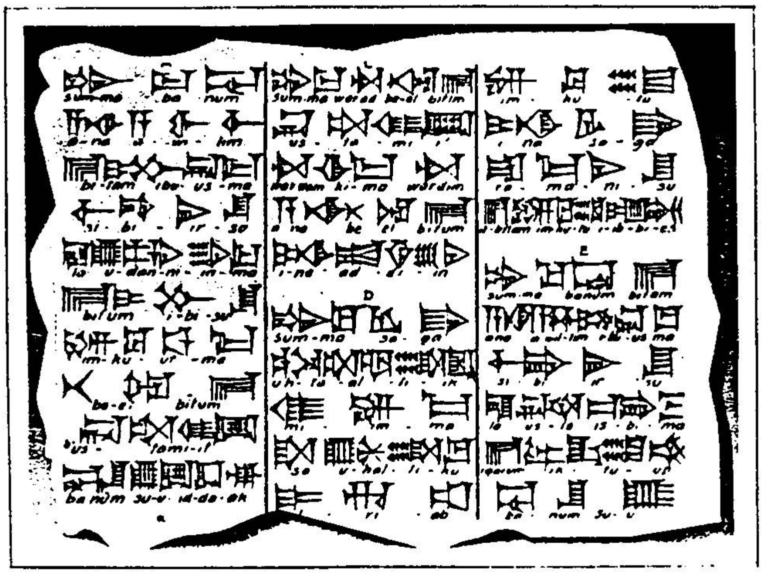

Figure 1. The oldest "building regulations" in the world

Just to give an idea I have checked how many standards are involved for the design and fabrication of a relatively simple steel structure for a building. This amounts between 30 and 35. For a more complicated structure such as a high-rise building and when also non structural aspects are taken into account the number is possibly close to 100.
Due to new developments the number of standards tends to increase.
Examples of such developments in the field of steel and composite structures are:

Product innovation
* Cold-formed steel products
* Open web sections
* Hybrid beams
* Slim floor construction
* Partly encased beams and columns
* Semi-automated connections
* Injection bolts

New materials
* High strength steel
* Super high strength steel
* High strength concrete
* Weather resistant steel
* Stainless steel
* Aluminium
* Coated sheeting

New design methods
* Plastic design
* Semi-rigid design
* Numerical methods
* CAD

New fabrication methods
* Special fasteners
* Welding methods
* Automation
* CAM

Allmost all of these topics were developed for use in structural steel design in the last decades and so were not covered by the standards of the past.

The structural standards formally describe the technical requirements for a building structure. Also they give methods to prove that the requirements are satisfied. Aspects to be covered are safety, serviceability and durability. The verification methods are based on analytical, experimental and empirical knowledge.
It is self-evident that the requirements and the required verification procedures have an important influence on the cost of the structure. As such an important role of standards is to avoid unfair competition between various manufacturers and various products and materials by defining uniform and harmonized requirements.
In many countries the standards have developed on one hand as source of knowledge for the designer but at the same time they serve as criterium for the evaluation of acceptance.
These two functions lead to different and contradictory requirements for the contents and presentation of standards.

The code-maker is confronted with the following requirements :
= The text shall be legal strict and not open for different interpretation but on the other hand the text must be easy to understand by designers;
= The rules must be simple but at the same time allow for economical and optimal designs;
= The rules must be suitable for hand calculations but also for computer-aided design;
= The scope should be well-defined but the standard should also be flexible so that new developments are not hindered by the standard;
= The size should be restricted but all new materials, products and systems should be included;
= Finally the code shall be modern but not changed frequently.

It will be clear that choices are necesary. And these choices have to be made by the engineering profession in a thorough discussion between all the interested parties. I am not convinced that this is now always the case !!

In conclusion my answer to the question posed in the title of this paper is :
= Yes, we need structural standards.
= Yes, they are a nuissance also. Especially so if not optimal attention is given to the structure, the scope and the presentation of the standards. Also instruments need to be developed for a better information transfer. For the future computer based knowledge transfer systems may help to overcome at least part of the problems.

It is obvious that an extra dimension to the nuissance is formed by the fact that different countries and regions all have their own rules and standards.
Fortunately in Europe the European Commission has initiated harmonisation of the structural standards for all European member states.
Hopefully this will be followed later by a world wide harmonisation.

2. Harmonized European standards

Stimulated and mandated by the Commission of the European Communities (CEC) the European Standards Organisation (CEN) has set up an action plan to develop a complete set of harmonized European building standards.
The complete set consists of design standards (Eurocodes), standards for fabrication and erection (in Euro-lingo called: execution) and product standards.
In figure 2 an overview is given of the set for steel structures for buildings. Similar sets are being developed for other materials as concrete, timber and masonry.

EUROPEAN STANDARDS

☐ DESIGN CEN/TC250/SC3	→	ENV 1993 - Pt 1.1 Eurocode 3 Design of Steel Structures
☐ EXECUTION CEN/TC/135	→	ENV 1090 - Pt 1 Execution of Steel Structures
☐ PRODUCTS	→	EN - Standards ↑ ISO-Standards

Figure 2. European standards for steel structures in buildings

☐ Design

The Commission of the European Communities (CEC) initiated the preparation of a set of European Codes - the Eurocodes - for the design of buildings and civil engineering structures. These codes are intended to establish a set of common rules as an alternative to the differing rules in force in the various Member States.
The advantages of having common rules are evident.

- The rules can be used in all European countries. This will make it possible to design a structure in one country for erection in another.
- Manufacturers will be able to design standard buildings for the whole European market with a single design.
- The use of common rules will make it easier for designers to work in other countries, without having to learn new Codes.
- The results of research carried out in all countries can be used for development of one set of design requirements.
- Handbooks, design aids and educational material can be produced for use all over Europe.

The Eurocode-programme is aiming at two dimensional harmonization:

(1) Harmonization across the borders of the European Countries;

(2) Harmonization between different construction materials, construction methods and types of building and civil engineering works to achieve full consistency and compatibility of the various codes with each other and to obtain comparable safety levels.

The EUROCODE-programme provides for a total set of nine volumes.

For the design of steel and composite structures the following volumes and parts are direct relevant:

ENV 1991: Eurocode 1 - Basis of design and actions on structures
Part 1 - Basis of design
Part 2 - Actions on structures
Part 3 - Traffic loads on bridges
Part 4 - Actions in silos and tanks
Part 5 - Actions induced by cranes and machinery

ENV 1993: Eurocode 3 - Design of Steel Structures
Part 1.1 - General rules and rules for buildings
Part 1.2 - Fire resistance
Part 1.3 - Cold formed thin gauge members and sheeting
Part 2 - Bridges and plated structures
Part 3 - Towers, masts and chimneys
Part 4 - Tanks, silos and pipelines
Part 5 - Piling
Part 6 - Crane structures

ENV 1994: Eurocode 4 - Design of composite steel and concrete structures
Part 1.1 - General rules and rules for buildings
Part 1.2 - Structural fire design
Part 2 - Bridges

ENV 1998: Eurocode 8 - Design provisions for earthquake resistance of structures
Part 1.1 - General rules
Part 1.2 - Building
Part 1.3 - Various materials and elements
Part 1.4 - Strengthening and repair
Part 2 - Bridges
Part 3 - Towers, masts and chimneys
Part 4 - Tanks, silos and pipelines
Part 5 - Foundation

□ Execution

The design procedures in EC3 and EC4 are only valid if the workmanship criteria during fabrication and erection given in Chapter 7 are satisfied. For example, the levels of initial geometric imperfections assumed in many of the strength rules are directly related to these criteria and are therefore invalid if they are exceeded.
A separate CEN committee, TC135 "Execution of Steel Structures" has drafted the fabrication and erection rules in close contact with CEN TC250/SC3.
These rules for fabrication and erection are given in ENV 1090.
The main reasons for developing a European Standard for execution of steel structures are:
- To transfer the requirements set during design from the designer to the constructor, i.e. to be a link between design and execution.
- To give instructions to the constructor on how to execute the physical work (fabrication, welding, bolting, erection, protective treatment) as well as to give requirements for

accuracy of the work.
The standard will thus serve as a document which gives standardized technical requirements when ordering a steel structure.
- To inform and serve as a checklist for the designer with respect to information which needs to be specified in the project specification for the particular project. It is foreseen and required that each project shall have a project specification which defines the technical requirements for that project. Such a project specification could be a single drawing for a minor project or a comprehensive package of documents for a complicated structure.

This standard for fabrication and erection will consist of the following parts:

ENV 1090: Execution of steel structures
Part 1 - General rules and rules for buildings
Part 2 - Rules for cold formed thin gauge members and sheeting
Part 3 - Supplementary rules for high strength steels
Part 4 - Supplementary rules for hollow section lattice structures
Part 5 - Supplementary rules for bridges and plated structures
Part 6 - Towers, masts and chimneys
Part 7 - Tanks, silos and pipelines
Part 8 - Piling
Part 9 - Crane structures

□ Products
The product standards are mainly equal with or derived from existing Euronorms or ISO-standards. The product standards may betwween more concern the following categories of products:
- Structural steel
- Sections and plates
- Bolts, nuts and washers
- Welding consumables
- Rivets
- Corrosion protection

An overview of the European standards for structural steel, sections and sheets is given in table 1.
European standardisation has led to new classifications for steel which have been published in EN 10020. Steels are classified in five types, contingent on their chemical composition, deoxidation method, and impact requirements.
As with classification, standardisation has led to new designations for steels. The system is given in EN 10027 and Information Circular ECISS/IC10.
The designation of structural steel is as follows:

S	275	J2	G2
structural application	minimum yield strength	impact symbol	deoxidation

In table 2 the standards for bolts, nuts, and washers are given. For washers the set of EN standards is not yet complete. The missing parts are for the time being replaced by the relevant ISO- standards. The classification system and general information on mechanical properties are given in EN 20898.

<table>
<tr><th>Products</th><th>Delivery conditions</th><th>Dimensions</th><th>Tolerances</th></tr>
<tr><td>I and H sections</td><td rowspan="7">EN 10025
EN 10113
EN 10155</td><td>missing</td><td>EN 10034</td></tr>
<tr><td>I sections-tapered flanges</td><td>missing</td><td>EN 10024</td></tr>
<tr><td>U sections</td><td>missing</td><td>missing</td></tr>
<tr><td>Angles</td><td>prEN 10056-1</td><td>EN 10056-2</td></tr>
<tr><td>T sections</td><td>missing</td><td>prEN 10055</td></tr>
<tr><td>Plates</td><td>not relvant</td><td>EN 10029
EN 10051</td></tr>
<tr><td>Strip</td><td>not relevant</td><td>EU 91</td></tr>
<tr><td>Hot formed hollow sections</td><td>EN 10210-1</td><td>prEN 10210-2</td><td>prEN 10210-2</td></tr>
<tr><td>Cold formed hollow sections</td><td>prEN 10219-1</td><td>prEN 10219-2</td><td>prEN 10219-2</td></tr>
<tr><td>Sheet</td><td>EN 10025
EN 10113
EN 10147</td><td>not relevant</td><td>EN 10131</td></tr>
<tr><td>Steel improv. deformation prop.</td><td>EN 10164</td><td>not relevant</td><td>not relevant</td></tr>
</table>

Table 1: Productstandards for steel and steel products

<table>
<tr><th>Class</th><th>Bolts</th><th>Nuts</th><th>Washers</th><th>Note</th></tr>
<tr><td>4.6</td><td>EN 24016
EN 24018</td><td>EN 24034</td><td rowspan="2">ISO 7091</td><td>Fully threaded</td></tr>
<tr><td>5.6</td><td>EN 24014
EN 24017</td><td>EN 24034</td><td>Fully threaded</td></tr>
<tr><td rowspan="2">8.8</td><td>EN 24014
EN 24017</td><td>EN 24032</td><td>ISO 7089
ISO 7090</td><td>Fully threaded</td></tr>
<tr><td>PrEN 781</td><td>PrEN 780</td><td>PrEN 784
prEN 785</td><td>Suitable for preloading</td></tr>
<tr><td rowspan="4">10.9</td><td rowspan="2">EN 24014
EN 24017</td><td>EN 24032</td><td rowspan="2">ISO 7089
ISO 7090</td><td rowspan="2"></td></tr>
<tr><td>EN 24033</td></tr>
<tr><td>prEN 781</td><td>prEN 780</td><td>prEN 784
prEN 785</td><td>Suitable for preloading</td></tr>
<tr><td>prEN 782</td><td>prEN 783</td><td>prEN 785</td><td>Suitable for preloading</td></tr>
</table>

Table 2: European standards for bolts, nuts, and washers

3. Introduction of the European standards

The complete system of harmonised European standards is the result of a huge effort to harmonise and improve engineering practice across Europe. Now the task is to put these standards in practice. This is not at all easy. As illustrated before the designer is confronted with a great number of new standards.
Especially the design standards (Eurocodes) are voluminous and complex. Also the content, presentation and format is different from the existing national standards.
Therefore the steel construction industry has set up a number of activities for the dissemination of the European standards.
These activities are targeted at two groups: the students - the designers of the future - and the engineering profession of today.

☐ **Students**

Within the framework of the European action programme COMETT (European Committee Action Programme for Education and Training for Technology) the ESDEP project was launched. The aim of ESDEP (European Steel Design Education Programme) was the development and introduction of teaching material on steel structures for use in EEC-states and fully based on the new European standards. The material has been collected from over 400 contributors in 20 countries and over 10 European languages. This material is an important tool for the introduction of the European standards in education.
The complete ESDEP comprises 196 lectures and 36 worked examples in 15 volumes. These are illustrated by over 2000 figures and supported by 1000 slides, 20 videos and computer aided learning software. The material is presented in a modular format that enables it to be used in a flexible way to suit the needs of both the teacher and the learners.

☐ **Engineering profession**

For the introduction of the European standards on steel structures in the engineering profession the ECCS (European Convention for Constructional Steelwork) plays an important role. By various committees, with membership from fabricators, designers and academics, a number of publications and design aids have been prepared as a help for the use of the new standards. Examples of this material are:

- *Essentials of Eurocode 3*
  This is an "abridged" version of Eurocode 3 - Pt.1.1. It is intended to be used for daily practical design work. The contents covers about 90% of the normal applications of steel structures for buildings. For comparison EC3-Pt.1-1 has about 350 pages and the Essentials only 60 pages. Much attention is given to a users-friendly presentation with many explanatory figures and tables.
- *Design examples to Eurocode 3*
  This publication presents a series of design examples which conform with the requirements of Eurocode 3.
- *Composite beams and columns to Eurocode 4*
  In this publication instructions are given for the design and verification of composite steel and concrete beams and columns according Eurocode 4. The document provides background information for the rules in EC 4 and contains design tables and worked examples.
- *Design manual for composite slabs*
  This is a similar publication as above related to composite slabs

Many other dissemenation activities on national as well as international level are going on. Most of the projects however result in printed material.

4. Use of IT technology to support the introduction.

In section 2 an overview is given off the set of standards required for the design and execution of steel structures. Together with the introduction material as presented in section 3 this represents thousands of pages of text, figures and tables.
The standards contain many cross-references and are related to the general building regulations.
It is not at all easy to manage all this information in daily practice. It can be expected that the traditional methods of disseminating standards are not fast enough en not sufficient to allow the design profession to rapidly adapt themselves to the technical and economical changes imposed by the new standards.
Modern IT technology can provide important tools to overcome the obstacles.
This idea is supported by the experience with a project in the Netherlands. TNO developed a CD-ROM containing the text of the building regulations, the supporting standards and product information. TNO also developed the retrieval module including hypertext links, keyword search etc. This system is fully operational and succesfully used in practice.

Recently a number of research institutes and industrial partners formulated a proposal for a project aiming at the development of a prototype of an integrated working environment for the users of Eurocodes.
The basic layout of the targeted system is shown in figure 3.

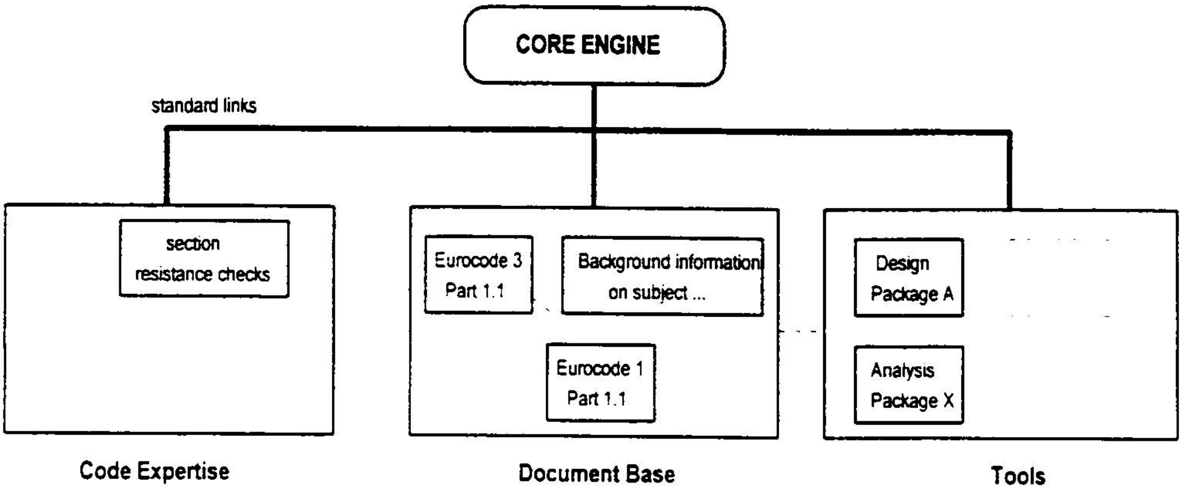

Figure 3. Integrated working environment for users of Eurocode 3

The author of this paper is convinced that this is a very promising development giving great changes to the engineering practice for an efficient use of the ever growing amount of technical information. Also it will provide an important tool for code makers to check the implications of changes and updates for cross references to or from other codes.
It is hoped that the industry and EEC will understand the importance of this development and provide the necesary means to start this project.

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THE EC1 AND THE NEW ITALIAN CODE ON ACTIONS

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Luca Sanpaolesi is involved in Structural Engineering studies, with particular regard to Actions. He has participated, since the beginning, to the development of the Eurocodes, in a first phase under EEC and in the second phase under CEN guide.
In EC1 works he has been Convener of the Project Team on Snow Loads and member of the PT on Traffic Loads on road bridges. In Italy is involved in preparing codes on Actions.

Summary

One of the main problems of the ENV phase of the EC1 code on Actions, is the difficulty to conciliate, in each country, its NAD with the national code and the EC1 itself, being the designer free to employ one of this codes.
Italian code has already been updated and made as similar as possible to the EC1, so that it is no admitted alternative use of the European Code.
In the following, since it seems to be interesting in the author's opinion, it will be illustrated the normative criteria adopted in Italy.

= ❖ =

The analysis of the new Italian national code on Actions seems to suit very well the scientific purpose of the IABSE Delft Colloquium on Action on Structures, since it outlines a new way to take a first step towards the European harmonization of Eurocodes.
It is well known that during the ENV phase of Eurocodes, the national Authorities of each member State, charged to prepare the national codes, allow, in theory, the alternative use of each EC, completed by the NAD and anyway supported by the national code, that is to be considered the principal rule to follow. This way of proceeding has been observed in almost all European countries for the application of the first EC and, in particular, for the ENV-1992-1 "Concrete Structures" and ENV-1993-1 "Steel Structures".
Nevertheless this philosophy, which makes the designer able to select the code to be adopted in designing a structure, seems not to be applicable to the Eurocode on Action on Structures, since it seems not possible to let the Engineer select between code provisions which will lead to different loads to be applied on the studied construction.
It is important to underline the fact that codes on actions do not give rough load values, but furnish the design criteria, which enclose a great number of considerations, so that trough an adequate, more or less complex, design procedure the design loads are fixed.

It is so explained the reason for which it is not possible to charge the Engineer to decide the action values to be applied on structures.
On the other hand it is completely reasonable that an Engineer could design his structures not necessary with reference to the provisions of the EC2, for concrete structures, or EC3, for steel ones. The results of such different design procedures might perform adequate and equivalent safety levels.
The author of this paper put in evidence the problem in the TC/250/SC1 meeting held in Paris the 28th - 29th April 1992 by proposing a motion about the elimination of the ENV phase for the Eurocode on Actions, introducing directly the EN phase.
The purpose was discussed but it was decided not to introduce such a variation, since the committee retained not to exclude the temporary phase of the EC1, typical of the ENV.
Now, however, the problem is again actual and it have to be faced in each country.

On the other hand it has to be remembered the favourable fact, regarding the adoption of the ENV-1991-2, that the snow and wind load maps provided by EC1 and the national code ones of each country were the same. In fact it is well known that during the EC1 studies, since it was impossible to elaborate in a few months the new European maps of snow and wind loads, it was decided to go back to the national load maps of each member state and to introduce these ones into the EC1.
In collecting national load maps it was requested, to each National Competent Authority to furnish sounded data elaborated with homogeneous criteria, such as the return period. Each country, among which Italy, participated often updating and improving the national map.

In Italy designing codes and therefore even the code which defines the action on structures, are mandatory and published on the Official Journal of Italy, so that they achieve an extremely important role and it is not possible to derogate from them in any case.

Italian Authorities, charged to prepare the national codes, once verified the inopportunity to make the designers use alternatively European or national codes on actions, since mandatory codes exist, and the availability of the ENV-1991-2 which encloses national wind and snow maps, decided to introduce a new national code on action which could substitute the previous and quite obsolete one. The new code should have been taken into account the new wind and snow load maps, updated during the elaboration requested by CEN when they were introduced in EC1.
Thus the new Italian code on action on structures, with particular reference to buildings, has been arranged following the general EC1 criteria and philosophy in fixing the parameters to be used in loads determination. Only few secondary modifications have been made in order to simplify the application and the practical use of the code and to better suit specific national situations.

The new code, recently published on the Official Journal of Italy, is now mandatory so that, without the NAD's publication, it is fulfilled the desired objective that is to allow the only use of a code as similar as possible to the ENV-1991-2 one, excluding any alternative possibility.

It has seemed useful, in the author's opinion, to illustrate the criteria adopted in Italy to achieve, in practice, the application of the ENV-1991-2.

THE INSERTION OF THE EUROCODES INTO THE NATIONAL CONTEXTS:

temporary assessment of the ENVs insertion and perspectives on the ENs insertion

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SUMMARY

The aim of this paper is to give an assessment of the insertion of the first ENV Eurocodes into the national contexts and to suggest what their insertion might be in the next stage, once the ENV are converted into ENs. The word *insertion* encompasses all the actions aiming at transposing the Eurocodes into a national standard that is widely *applied* to the verification of structural designs and used as a *reference* in the public or private documents dealing with the design rules.

The problems raised by the insertion show how complex the subject is. What is at stake in a good integration of the European standard is assessed (see 1).

The statutory measures provided for by the CEN in order to insert the European standards and the dispensations from these measures that may be applicable to the Eurocodes, given their special statute within the European normalisation, are recalled (see 2).

The additional measures taken to insert the Eurocodes as ENV are detailed. A provisionary assessment of the insertion of the first ENV Eurocodes is given (see 3).

The application of the Eurocodes to the verification of designs is most often prescribed by bodies that are separate from those that apply them (i.e. national authorities, insurance companies, clients). Drawing a lesson from the experimentation of the insertion of the first ENVs, a strategy of insertion, focussing on the satisfaction of the bodies that usually prescribe the application of the structural design rules at the national levels, is recommended. The aim is to introduce *basic adaptations* into the EN so that the documents usually referring to the national rules for structural design may refer to the national standards transposing the Eurocodes. The object of the basic adaptations should be to make the national standard transposing the EN compatible with the safety and/or quality policies adopted at the national level. The basic adaptations should be introduced either into the EN, as *particular national conditions*, unless they are already in the ENV, or in the *National Application Document* (NAD) as *basic transposition measures* (see 4.1).

The basic adaptations are identified and the theoretical elements used to make them are detailed (see 4.2).

An outline is given of a possible way of reaching the major aim of the insertion : to make possible the reference to the Eurocodes both in the national structural design rules and in the construction works contracts (see 5).

Finally, the provisions for the transposition of the EN Eurocodes that are recommended would consist in taking up the adopted measures for the transposition of the ENVs while complementing them :

- the principle of the NAD would be kept
- the numerical values with an unreliable calibration only would be boxed
- guidelines, appended to the CEN Foreword, entitled "Basis of transposition" would make pratical recommendations on how to make the basic adaptations that are not already integrated as particular national conditions
- the theoretical elements necessary to implement the basic adaptations should be detailed into Basis of Design (sections 1 and 2) unless they are there already.

1. The problems of the insertion and the issues at stake

1.1. The scope of the insertion

The insertion of the Eurocodes into the national contexts covers all the measures that are supposed to be taken in order to transpose the European standard into a national standard, ensuring a high level of use of the national standard and allowing the withdrawal of the competing rules in the future. The term *use* encompasses the *application* of the national standards to the verification of structural designs together with the *reference* to the national standard within the documents that usually deals with structural design rules.

1.2. The problems of the insertion

The insertion of the Eurocodes raises problems that are at the interface of structural reliability theories, practices of design verification, construction works safety policies, construction works economy, quality policies adopted by the intervening parties.

The multiple aspects to be taken into account makes it a difficult issue. This state of things is accounted for by the fact that the rules are markedly regulatory and that, under the requirements of EU policy national prescriptions and insurance companies, the public clients especially public ones, credit these rules with a contractual nature.

Note : See Directive 93-37/CEE, article II, 10.

For any party intervening in the construction, the insertion of Eurocodes entails issues at stake. This is due to the fact that the insertion process implies the observance of implicit levels of accepted risks. Indeed such levels result from compromises, that have been reached previously, between the wish for the greatest possible safety and the efforts to be made by the intervening parties to meet needs that undisputedly have priority.

To make the insertion easy and efficient, one must deal with border relations between European and national authorities in a field where states and contracting bodies have strong and infinitely variable prerogatives.

Last, the great variety of the insertion contexts at the institutional, professional and economic levels adds to the difficulties.

1.3. What is at stake

The very quality of the insertion will greatly influence the issue of the Eurocodes and the possibility to withdraw the competing rules. In several Member States, particularly where the rules of structural design have the status of a law, regulation or contractual specification, the level and quality of the insertion should be an essential element of success, at least as important as the scientific and technical values of the EN or its user-friendliness.

2. The statutory provisions for the insertion

The statutory provisions for the insertion of the Eurocodes include general provisions appearing in the "Common rules for standards works" of CEN/CENELEC on the one hand, exceptional or derogatory measures stated into the "Agreement between the Commission and the CEN concerning the works on the Eurocodes" on the other hand.

2.1. The provisions of the CEN rules

They are precisely defined. They deal with the introduction of *particular national conditions* and *national deviations* and with the withdrawal of national standards that are equivalent to the EN.

2.1.1. National particular conditions

National particular conditions may appear within the course of the standard or in annexes. They must refer to technical realities and not to the concerned country.

2.1.2. National deviations

In case modifications or exceptions to the EN cannot be taken into account by the means of particular national conditions, national deviations may be introduced into the national standard transposing the EN. A distinction have to be made between national deviations of type A and type B depending on whether they are related to regulatory or technical matters. Although national deviations are not part of the EN, the CEN is careful to control their introduction by the means of authorisation procedures and acceptability criteria.

2.1.3. The withdrawal of equivalent national standards

The date of the withdrawal of national standards that are equivalent to the CEN (DOW) is stated by the Technical Board (CEN/BT). The duration of coexistence of both standards has to be no longer than six months. The CEN/BT may allow the prolongation of the withdrawal date.

2.2. The provisions of the "Commission/CEN Agreement"

The CEN rules have been stated in order to develop product standards. So derogatory measures have been introduced into the Agreement between the Commission and the CEN concerning the works on the Eurocodes to take into account the specificities of the structural design rules. They concern the stages of development of the Eurocodes, the conditions of withdrawal of the equivalent national "rules" (and not only equivalent national "standards").

2.2.1. The stages of development of the Eurocodes

The Eurocodes are to be developped in two stages as European pre-standard (or ENV) then as European standard (EN). The ENV period of two years is used to experiment the Eurocodes or make practical applications of them.

2.2.2. The withdrawal of equivalent national rules

The transformation of an ENV into EN and its adoption by the CEN will not automatically lead to the withdrawal of the national rules already existing in the fields concerned. The time limits for the coexistence of Eurocode-standards or parts of them with the corresponding national standards will be determined in each case in common agreement between the Commission, the Member States and the CEN.

3. The insertion of the Eurocodes as ENVs

3.1. The provisions for the insertion of the Eurocodes as ENVs

The provisions taken for the insertion of the Eurocodes as ENVs include the statutory provisions (see section 2) and particular provisions for the ENV stages : the appending of a National Application Document (NAD) to the ENV, the boxing of numerical values, the distinction between Principles and Application Rules and the possibility to introduce alternate Application Rules.

3.1.1. The alternate Application Rules

In the clauses of the Eurocodes, the Principles are distinguished from of the Application Rules. There may be an alternative to an Application Rule provided that it is shown that the alternative rule accords with the relevant Principles and has at least the same reliability.

3.1.2. The boxed values

The numerical values that are supposed to be related to safety elements are boxed.

3.1.3. The NAD

The transposition of an ENV into an experimental national standard consists in including a National Application Document to the translation of the ENV. The scope of the NAD is threefold :

- to adapt the boxed values eventually.
- to refer to the accompanying compatible national standards and detail the conditions of application of the temporary reference standards
- to define the national directives for the application of the ENV.

3.2. Provisional assessment of the insertion

3.2.1. Transposition

The provisions described in 3.1 have made possible the transposition of the Eurocodes in the Member States whose legal structures and administrative organisation did not obstruct it or did not take most of its impact away from the experimentation.

It results from the assessment of the transposition of the first set of Eurocodes (i.e. ENV 1991-1 - Basis of Design and parts 1-1 of the ENV 1992 to 8) that these measures have proved pertinent on the whole. Yet they have proved insufficient and hardly convenient on some points :

- one cannot easily adapt the levels of requirement for safety in case the safety policy of the national authorities or the client are a part of a quality policy.

  Note : In Norway, the national design rules take into account the levels of design supervision, and the level of execution control.

- the same goes in case safety and economic requirements are competing.

  Note : See economic impacts of the detailings for concrete building specified within the ENV 1992-Part 1.1.

- the declared scope of the boxed values is to focus the bodies in charge of the transpositions on the reliability elements ; in fact this provision is ambiguous since numerical values with an uncertain or controversial calibration are boxed.

  Note : See properties of particulate material ENV 1991-4 - Table 7.1.

- moreover the provision leads to believe that the adaptation of the levels of requirements for safety can be made in modifying numerical values only (see 4.2.2).

Note : The requirements for the prevention of hazards due to human activities along the construction process or in the course of the working life of the construction works are represented in the Eurocodes as Assumptions.

- besides, the choice of the boxed values may prove arbitrary or insufficiently detailed (e.g. detailings).

However all technical imperfections have been got round ultimately.

3.2.2. Experimentation

The experimentation of the Eurocodes takes various forms :

- general studies (e.g. calibration of safety elements, impact of the application of the European rules on the cost of construction works)
- comparisons (e.g. dimensioning of an existing structure by the means of the Eurocodes alone or associated to the NAD)
- proposals of the application of the Eurocodes for international projects outside the European area
- application of the Eurocodes to the justification of designs of new construction works in the European area
- insertion of an Eurocode or parts of it into the corpus of the national design rules in case gaps have to be filled or outdated rules have to be updated.

The attempts to apply the Eurocodes to the justification of new construction works design have seldom been successful. The obstacles that could not be overcome were invariably due to the reluctance of bodies that refer to the structural design rules (clients fearing an increase in construction costs or insurance companies unable to stand back for lack of experience).

4. Perspectives on the insertion of the Eurocodes as ENs

What is at stake (see 1.3) justifies that provisions should be taken by the CEN to make the insertion of the Eurocodes as EN possible and efficient the insertion. These provisions should aim at explicit objectives and take part in a strategy.

4.1. The strategy of insertion

4.1.1. Objective of the insertion

The objective should be to maximise the level of effective applications of the national standard transposing the Eurocodes in comparison with the potential applications.

4.1.2. The way and means of the insertion

The way used to reach the objective should be indirect. One should manage to make it possible to refer to the national standards transposing the Eurocodes in the documents usually prescribing the application of the national design rules (i.e. regulations on structural safety, insurance policies, technical specifications of contracts).

Three means should be considered :

- the introduction of the concept of *basic adaptation* measures
- the introduction into the EN of particular national conditions implementing basic adaptations
- the development within the EN of the theoretical elements likely to facilitate the implementation of the basic adaptations at the transposition stage.

4.1.3. The basic adaptations

The bodies that prescribe the application of structural design rules are a specific population (regulatory authorities, clients, insurance companies, technical controllers) quite distinct from the bodies that apply them (designers, manufacturers of structural elements, contractors). The concern of these prescribers is to implement the technical aspects of a safety policy of construc-tion works (e.g. regulatory authorities, insurance companies) or a quality policy where reliability is considered as one aspect of quality (clients).

The basic condition set for the acceptance of design structural rules is the compatibility of the rules with the safety and/or quality policies adopted by these bodies.

Note : The scope of the ECs is :

- to translate objectives of an implicit safety policy in terms of requirements for safety
- to specify the formal rules (i.e. Principles and Applications Rules) and the associated conditions to be fulfilled (i.e. Assumptions) to assess the conformity of a structural design to specified requirements for safety.

The scope of construction works safety (resp. quality) policies is :

- to identify the risks of structural failures (resp. of non-quality) and to define the levels of admitted risks (resp. to state the quality of the construction works)
- to translate the admitted risks (resp. stated quality) in terms of requirements for safety (resp. for quality)
- to specify the requirements calling for an attestation of conformity.

It results from the parallel that a national standard transposing the ECs may be made compatible with a safety/quality policy provided that :

i) - the requirements for safety that are considered in the ECs correspond to the objectives, or the requirements, of the safety (resp. quality) policy,

ii) - the legal, regulatory, and contractual conditions be taken into account.

Transposing the Eurocodes into a national standard that should meet the compatibility requirements above mentioned implies that the following *basic adaptations* should be made to the EN :

a) adjusting the level of the requirements for safety that are considered within the EN

b) differentiating the levels of the requirements for safety
c) compensating the levels of the requirements related to interchangeable safety measures
d) detailing the "Assumptions" whose content is vague and adding other legal, regulatory or contractual conditions.

4.2. Basis of transposition

4.2.1. Implementation of the basic adaptations

The National Application Document (NAD) should be maintained.

The basic adaptations (see 4.1.3 a) to d)) should be implemented into the NAD unless they are already integrated into the NE, as particular national conditions, at the ENV level or during the conversion of the ENV into EN.

Note : See an example of particular national conditions relating to the differentiation of a level of require-ment for safety : ENV 1998-1.2 - 3.7 Importance factor.
See an example of particular national conditions relating to a compensation between levels of requirements for interchangeable safety measures : ENV 1996-1.1 - 2.3.3.2 Partial safety factors for materials.

The theoretical elements necessary to implement the basic adaptations would be introduced into the format of the Eurocodes unless they are already there (i.e. 1991-1 Basis of Design - Sections 1 and 2).

Note : In the detailed review of the basic adaptations (see 4.2.2 to 4.2.5) the words in italic correspond to the new theoretical elements that, in our view, should be introduced in Basis of Design.

A guidelines, entitled "Basis of transposition" and appended to the CEN foreword of the Eurocodes should make recommendations on how to implement the basic adaptations into the NAD thanks to theoretical elements.

4.2.2. The adaptation of the levels of requirements for safety

The *risk analysis* inherent in the format of the Eurocodes implies the reference to several *categories of requirements for safety*. The *safety elements* that determine the levels of the requirements for safety are specific for every category of requirement. They are represented most often by numerical values. But in case of *requirements for the prevention of hazards* they are Assumptions and in case of *requirements against the effects of hazards* they are variables.

Note : For example :

- the requirements for the prevention of errors or imprecisions of execution are usually represented by an Assumption
- the requirements for the characteristics of the fire protection materials are represented by variables.

To make the adaptation of the levels of requirements for safety possible, one must be able to modify any safety element of any category of requirement for safety.

4.2.3. Differentiation of the levels of requirements for safety

The differentiation of levels of requirements for safety leads generally to introduce a *level of requirement classification*. Most often, the classification is the result of the application (in the sense of the Sets theory) of a *criteria classification* to a *safety element classification*.

Note : Examples of safety elements classifications :

- design fire time t_i
- importance factor γ_I introduced in ENV 1998-1.1.

The criteria classifications may relate to :

- the risk level
- the level of management measures that are assumed to be taken to prevent hazards due to human activities during the construction stage or the use of the construction.

Note : For example : supervision of design, control of execution, control of use, procedures of maintenance.

The safety elements classifications are normative, whereas criteria classification are given as a rough guide. The definition of the latter is within the competency either national authorities (in case they have a regulatory character) or the clients. They are supposed to be specified either in the transposed national standard, or in the technical specifications of contracts.

4.2.4. Compensation between levels of requirements for interchangeable safety measures

The safety measures aiming to prevent hazards, protect the structure against the effects of hazards and reinforce the design characteristics are interchangeable. The specified requirements for these categories of measures lend themselves to operations of compensation for a given level of admitted risk.

The mechanism of compensation is used to implement a safety strategy or also to take the quality policies of the intervening parties into account.

The mechanism implies that one may assess and testify the conformity of the design to the various categories of specified requirements for safety, whether those requirements are related to measures that come before or after the design verification.

Note : The γ_M values to be taken into account in the verification of mansory structure give a representative example of compensation between requirements for the execution control, for the materials control and the design dimensioning (see ENV 1996-1.1 - Table 2.3).

4.2.5. The introduction of particular conditions

The Application Rules of the Eurocodes are conditional. As the conditions that are taken into account in the Eurocodes correspond generally to those that are ordinarily fulfilled in Europe, they may remain implicit. Only the conditions to which the verification rules are sensible are explicited in the standard as Assumptions.

A design verified by the means of the Eurocodes shall comply with the specified requirements for safety provided that the Principles are observed, the Application Rules are verified and the Assumptions are fulfilled.

If, for geographical, institutional, economical contractual reasons, a condition differs fairly from the corresponding condition implicitly or explicitly considered in the Eurocodes, the conditions have to be changed explicitly and the rules must be modified. One may sometimes have to consider an alternate rule.

Note : See the alternate rule for snow load shape coefficients for specific climatic regions - ENV 1991 - 2.3 Annex B).

The device consisting in differentiating numerical values of the safety elements may avoid such complications.

Note : The obstacles raised to the access of any part of the structure modify the *conditions of survey* of the structure. In the Eurocodes, this change of conditions is translated in terms of increased *design working life*.

A reinforcement of the traffic regulation on a road (or a river), that modifies the usual *conditions of circulation* on a bridge (or on a river), may lead to reduce *the characteristic value* of the traffic loads (or traffic impacts).

5. The reference to the Eurocodes in national rules and contracts

In table 1 the adaptations to be brought in the EN to transpose the Eurocodes into a national standard that specifies the national structural design rules are detailed.

In table 2 are detailed the adaptations to be brought in the national standard transposing the Eurocodes, to specify the structural design rules to be taken into account in a contract.

Figure 1 draws a diagram of the mechanism that makes the reference to the Eurocodes in the contracts possible.

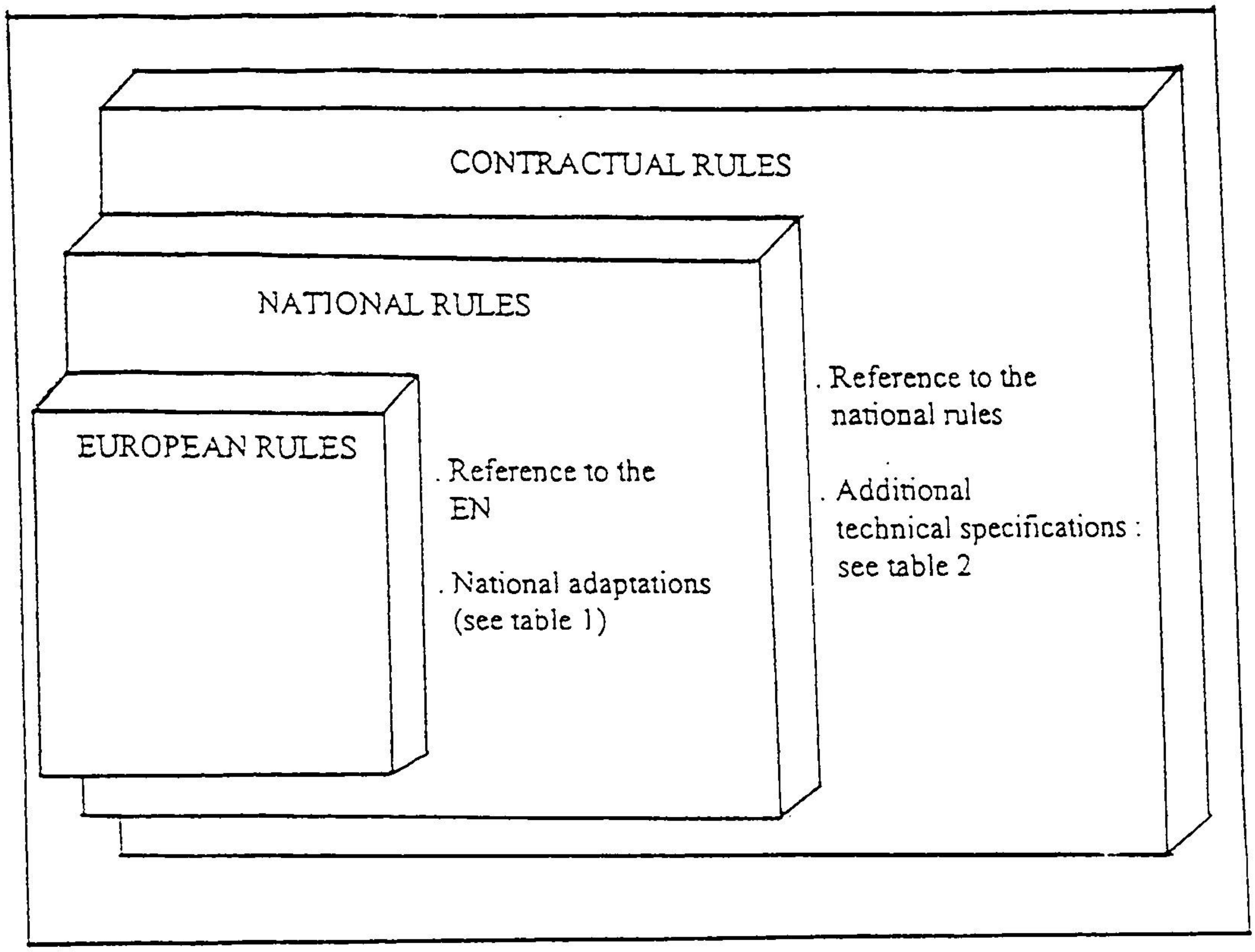
Table 1 - Specifying the national structural design rules by referring to the Eurocodes: the adaptations to be made to the EN, that the national structural design rules are supposed to refer to

a) To adapt or differentiate the levels of requirements for safety and to make possible the compensation of the levels of requirements for interchangeable safety measures (see 4.2.2 to 4.2.4)
b) To adapt or complete the Assumptions in order to take into account the climatic, institutional, economical, etc... conditions that differ significantly from the corresponding conditions explicitly or implicitly considered in the Eurocodes (see 4.2.5)
c) To detail the requirements for safety that are held to be "fundamental" (see ENV 1991-1, 2.1) at the national level but nevertheless are not considered in the Eurocodes. To specify the corresponding verification rules to be referred to.

Table 2 - Specifying the structural design rules, in a contract, thanks to the reference to the Eurocodes : the adaptations to be made in the national structural design rules that are supposed to refer to the Eurocodes

a) To detail the alternate Application Rules (see 3.1.2)
b) To detail the additional conditions to be fulfilled for the design to comply with the project specifications
c) To detail the requirements for safety that are specified in addition to the "fundamental" requirements (e.g. additional serviceability requirements). To specify the corresponding verification rules to be referred to.

Fig 1 : The specification of national and contractual structural design rules by reference to the Eurocodes.



Conference Review and Looking Ahead

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David Lazenby, born 1937, received the postgraduate DIC at Imperial College, London. He is a Fellow of the City and Guilds, London. His work in civil and structural engineering has ranged from U.K. to the Mediterranean and Africa.
In standardisation, he is a member of the Construction Board of BSI, and took over the chairmanship of the Eurocodes Committee in 1993.

Many years were spent in preparation of the preliminary reports forming the basis of the first Eurocodes in the 1970's and 1980's, under the direct authority of the European Commission. In 1990 the work was taken into the European Standardisation body CEN, and the head committee CEN/TC 250 was established to direct the whole enterprise.

As chairman of CEN/TC 250, I can report that the Eurocode programme is making very good progress, with a total suite of some 50 Eurocode parts to be published. Of these, more than half have already been published as ENV (provisional/experimental) stage documents, and work is in progress on all the others. The first four ENV's are now starting conversion to the final EN form, including EC1-Part 1, "Basis of Design".

The whole Eurocode programme has recently received a major boost, as the Standing Committee of the member states (SCC) has authorised work on the conversions of all the Eurocode parts as soon as practicable. We can therefore look to practical completion of the full suite of Eurocodes within a few years, and the absorption of them into the national systems.

This conference should provide valuable input to the detailed work on EC1, as well as hopefully reflecting the industry's views on the nature of codes in general. The conclusions of the conference may therefore help us to look ahead to the important matters to be addressed as we approach the implementations of the EN Eurocodes, as well as to the future of design codes generally in our industry. As the nature of the industry develops, and the methods and demands on it change, we must take care that the codes we produce reflect this ongoing evolution.

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THE BACKGROUND DOCUMENT FOR SNOW LOADS

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Luca Sanpaolesi is involved in Structural Engineering studies, with particular regard to Actions. He has participated, since the beginning , to the development of the Eurocodes, in a first phase under EEC and in the second phase under CEN guide.
In EC1 works has been Convener of the Project Team on Snow Loads and member of the PT on Traffic Loads on road bridges. In Italy is involved in preparing codes on actions.

Summary

The Background Document treats about the most important, both technical and scientific arguments, regarding snow loads, showing the reasons of the choices, which has been made by the Project Team during the elaboration of the new Code for Snow Loads and also the open problems, proposing future research programs.

1. Introduction

The chapter dealing with rules on snow loads forms part of the Eurocode 1 - Basis of Design and Action on Structures. In 1990 a specific Project Team (PT) was formed and charged to carry out a research programme in order to produce this chapter. The PT was made up of: Luca Sanpaolesi (*University of Pisa - Italy*), Manfred Gränzer (*Landestelle für Bautechnik - Germany*), Haig Gulvanessian (*Building Research Establishment - United Kingdom*), Joel Raoul (*SETRA - France*), Rune Sandvick (*NBR, Norway*), Ulrich Stiefel (*Gruner AG, Switzerland*). In addition the following contributed to the research: John Tory (*Building Research Establishment - United Kingdom*), Diana Currie (*Building Research Establishment - United Kingdom*), Riccardo Del Corso (*University of Pisa - Italy*).
At the end of the works, in 1993, the PT decided to explain, in a volume, the fundamental principles which inspired the Code ENV-1991-2-3 itself. This volume has been written in 1994 and 1995, it is titled "New European Code for Snow Loads - Background Document" and a copy of it is now available for each participant to this Colloquium.
In carrying out its work, the PT studied and discussed many specific issues relevant to the various aspect involved in defining snow loads on constructions. The scientific criteria followed by the PT in defining the Code has been based on present state of the art, rather than specific reviews of existing codes. Nevertheless, during the actual drafting, particular attention has been provided for the ISO 4355 (1981), not to introduce its contents into the New Code,

but only to verify the research results with existing ones. On the contrary new draft of ISO 4355, dated 1993, has not been taken into account, being too much complicated to serve at an easy and practical definition of Snow Loads.
In this spite the aims envisioned by the PT in drafting the Background Document can be divided into three:

- illustrate the underlying rationale for and the choices made in EC1 - Snow Loads;
- provide information regarding the basic studies to the NCA;
- furnish broader guidelines and explanations to designers.

In the following the Background Document will be shortly illustrated with special regard to some of the most relevant problems.

2. Ground Snow Loads

2.1. Climatic data

The first problem encountered in studying snow loads regards climatic conditions and the need for quantitative definition of the ground snow loads. The problem is quite complex and depends on several factors, such as region's climate, altitude and topographic features, many of which causes consequences in the determination of snow loads which lead to the need of more detailed studies in the field.
The soundest basis for assessing characteristic snow loads are long term records of snow loads measured at a large number of stations. Such a solid basis is difficult to achieve because of the scarcity, both in frequency and in geographical density, of the available data, which have often been collected not with engineering objectives. Another problem consists in obtaining homogeneous measurements taken all over the European territory, existing different measurement techniques, such as weighing snow cover or evaluating the water equivalent values starting from the measure of the snow cover depth. The major frequency, up to date, of records giving snow depth, instead of weigh values, has lead to the need to describe, with empirical formulae, since no physical models exist which would permit this calculation, the correlation between snow depth and snow density, taking into account all the factor which affect the deposition of snow, such as wind, temperature, rain falling onto the snow and the nature of the snow layer. The snow cover, in fact, can be considered, for some regions, to be the result of multiple snow events, in climates where the snow accumulates over a relatively long period of time; on the other hand the snow cover can also be considered as the result of single snow events, in climates where the snow tends to melt completely between successive weather systems.

2.2. Statistical analysis of snow loads

Data records have to be treated with statistical procedures aiming at fixing a load value for design purposes.
Statistical analysis is first applied to the record made at a single station alone. The daily registered snow load values combine to give a record of the whole winter season. The values of particular interest to be found in records of daily registered values are the absolutely yearly maximum. These extreme values, one for each winter, have a statistical distribution, which may be approximated to one of the well-known extreme value distribution functions.

The reliability of statistical analysis depends on the length of maximum value records. It has been proved, thanks to a German investigation based on 94-years snow depth record, that the design value derived from samples of a floating period of 30 consecutive winters are not yet stable, but still influenced by exceptional years. Consequently, in the purpose for CEN Code, in which snow loads are given with a mean recurrence interval of 50 years, a record length of 40 to 50 years have been suggested for the statistical analysis of the collected data.

As already mentioned, the statistical analysis of the data consists in checking on distribution types to find the best fit to the sample data. It has been found that the choice of the probability function is influenced by the climatic condition of each site; for example, the Gumbel distribution seems to be preferable for regions whose maximum snow cover is usually build up trough accumulation of several snowfalls, while the lognormal distribution better suits regions where maximum amount of snow is caused by a single snow event.

Compared to the imposed or wind loads, snow loads may have a notably higher coefficient of variation. The smallest coefficient are found in mountainous regions where snow falls quite regularly and accumulate during the winter. In many areas, especially in coastal areas and in the southern part of Europe, snowfalls do not occur every year. Taking into account these zero values, if their number is quite important, in the statistical analysis should lead to unrealistic results. In this cases the analysis should be restricted to the non-zero values only, by operating an adjustment of the return period.

Another problem which has been encountered is represented by the "exceptional snow falls". These values are so high that clearly do not fit the distribution calculated when they are discounted. A study, carried out in France, has shown the great influence that these values would produce on the distribution function's parameter if taken into account (see fig. 1).

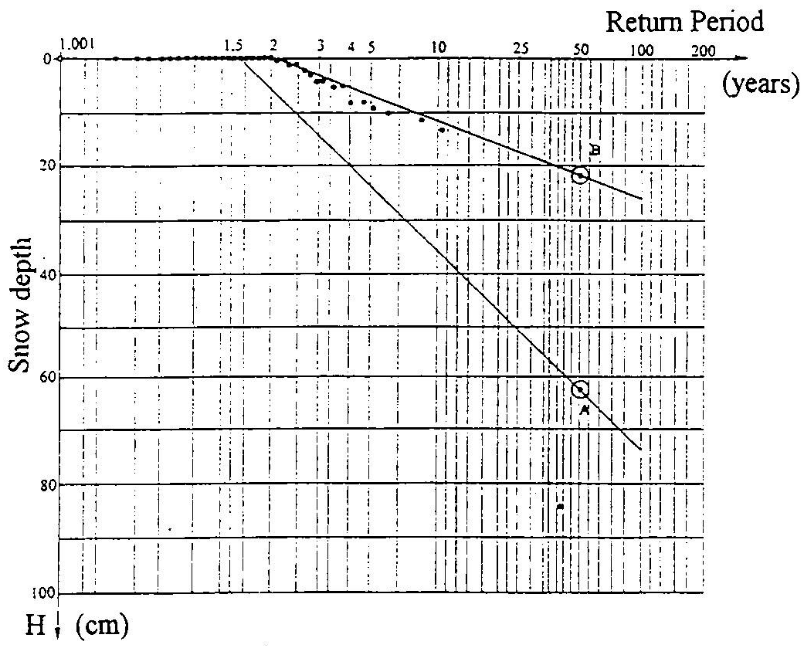

Fig. 1. Snow at Perpignan.

The purpose of the PT is to dealt with exceptional snow falls separately in order to determine the accidental value for the snowfall. A still open problem is the drawing of an European map where are defined all the areas where the exceptional snow falls have to be considered.

2.3. *Characteristic snow load and return period*

The characteristic snow load on the ground is based on annual probability of exceedance of 0.02 (1/50), that correspond to a mean recurrence interval of 50 years as recommended in the "Basis of Design". The choice of a 50 years return period, as already mentioned above, avoids inappropriate extrapolation from a data sample which generally cover several decades. It is to notice that it does not mean that it will necessary take 50 years after construction of a building for the characteristic snow load to be exceeded. From this point of view it is explained how much dangerous it is to reduce the design snow load taken from an extreme value distribution for a return period directly equal to the reduced expected lifetime of a temporary structure.

2.4. *Regionalization*

All the procedure and the problems encountered in what described above dealt with the analysis of records snow measurements at the single station, in order to find the characteristic ground snow load valid for each station. Now a procedure must be found to arrive at a geographic representation of the results, covering a whole region starting from the point values obtained at observation places.

The merely mathematical approach to this problem, trough one of the several existing methods, would give a continuos best fitting geographic distribution of the characteristic snow loads. Such an automatic procedure would completely ignores the knowledge and the experience of meteorologists and would furnish misleading results.

Sample data and the corresponding characteristic value obtained at a single station are influenced by several factors: orography, frontal waves, presence of great lakes, distance to the sea (macroscale effects); slope and contour of terrain, canopy and crop density (mesoscale effects); surface roughness presence of obstruction (microscale effects). All these parameters have to be taken into account for the extrapolation of a snow load map covering whole regions, making distinction between various homogeneous areas, in other words to carry out a regionalization.

It has been shown that very important parameters for local snow load variation are mainly: altitude, air temperature, orientation to solar radiation and wind exposure. In particular, it is often possible to arrive at a quite simple relationship between snow load and altitude alone, determining the "Altitude functions".

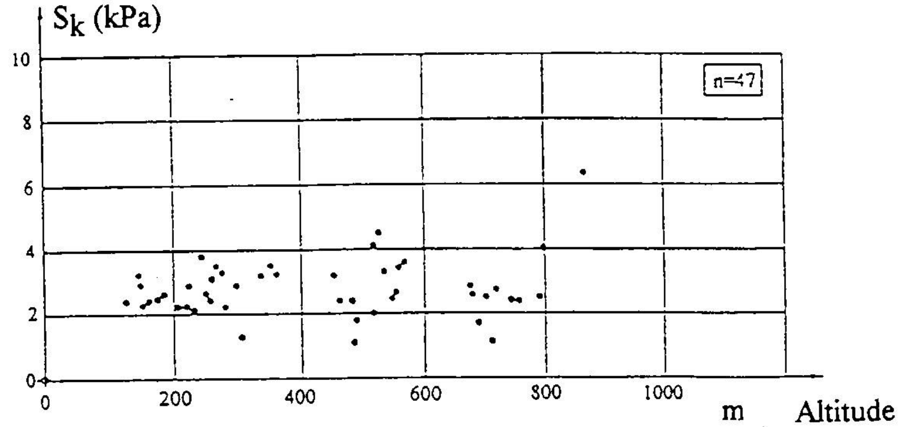

Fig. 2. Snow load; 50 year return period - County: Hedmark. Poor correlation between snow load and altitude.

Large European countries have used this method of zoning in their national codes. This simple procedure is not suitable for all European regions, as shown in a Norwegian study: there are areas where snow load does not increase with altitude following a more or less complex law (see figure 2).
The PT has always aimed at the definition of general rules, applicable in all CEN member states, in order to achieve a homogeneous framework for determination of design snow load. In a first phase was attempted to collect existing snow load data from several European countries and to elaborate in a new European snow map. Since the differences in the criteria that each country have followed in measuring, collecting and elaborating that data, it was impossible to proceed in this direction; the PT went back to the national codes re-elaborating these ones to achieve a common level of safety. In this way arose conceptual inconsistencies and not acceptable differences at the borderlines between the countries. This inconvenience had to be accepted during the first phase of works.
What stated above leads to the need of a great research programme in order to determine a new European snow load map, elaborated with common and homogeneous techniques all over the European territory. This research would permit to update records of each country and to standardise and simplify the application of the Code.

3. Snow loads on structures

The roof snow load is normally calculated form the ground snow load by multiplying by conversion factors which account for the roof shape, thermal characteristics, exposure and, depending on the code, other influences that may increase or decrease roof snow loads.
The scientific basis underlying determination of the roof coefficients is rather limited and research work has been carried out especially in cold regions, thus these results are not directly applicable to all of Europe. It has been necessary to develop empirical formulae supported by experience and engineering judgement. In this field the comparison of the adopted criteria and parameters for the determination of snow loads on structures in the CEN Code and in the ISO 4355 one, has been very useful for testing results (see figure 3).

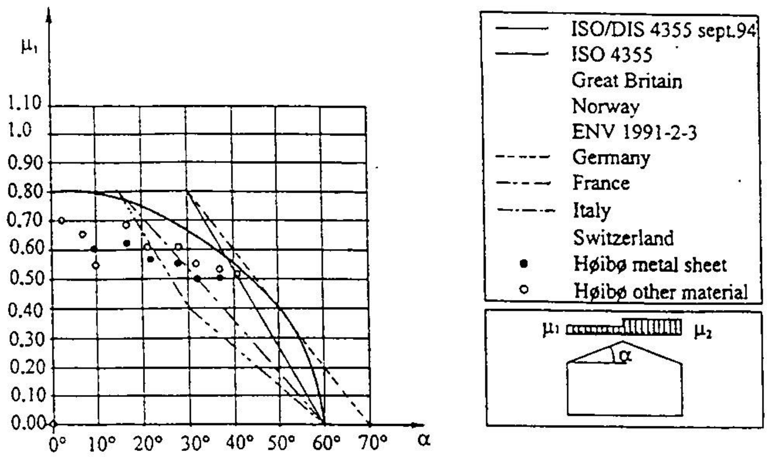

Fig. 3. Shape coefficient μ_1 for duo-pitched roofs.

In determining the conversion factors there are mainly three source of uncertainty: natural uncertainty, statistical and model uncertainty. Against natural uncertainties it can not be dealt with; on the other hand treatment of the uncertainties in the statistical and modelling procedures would follow lines similar to that for determining ground snow loads. It must be said that all the influences that affect ground snow loads determination also affect the roof snow load, to which are added the uncertainties in the other influences related to the roof itself. The statistical uncertainties begin with data sampling due to difficulties of measuring directly the snow load on roof and to the enormous number of different types of roofs. Although the code attempts to standardise such types, the huge of existing roof shapes must be underscored. As for the ground snow loads, the problem of translating height into the load, until new practical techniques of measurement will be set up, also exists for the roof snow load. The probability distribution function, or the probability model for analysis of sampled data has been studied only rarely. It is necessary to develop simple models which permit calculation of the design load, in respect of the fixed levels of safety. Within a reasonable degree of uncertainty, the selection of two different loading types can be proved: a uniform and an unbalanced distribution of the snow layer.
The substantial lack of scientific knowledge on a probabilistic basis has emerged from elaboration of the shape coefficient within the EC1 work. Only further research will be able to reduce such uncertainties and therefore future efforts must be concentrated on this issue. Herebelow are listed some of the specific arguments which could be object of this research:
- specific study about the definition of the values of the shape coefficients for the more frequent typologies of roof;
- probabilistic basis: only with such a prenormative research it will be possible to provide roof snow loads with a defined mean recurrence interval;
- shape coefficient for regions within single snowfalls.

4. Design situations

From the point of view of risk analysis it must be mentioned that the selection of relevant design situations is far more important than trying to develop "precise" partial factors. Therefore it is important to use good engineering judgement in selecting design situations that may occur and for which the design of the structure must be performed with reference to the SLS and ULS.
Special attention have to be used: in evaluating snow load on multi-level roofs with special dimensions such that cut drifts might result; in determining snow distribution model on roofs in those regions where, due to wind conditions, drifting predominates; in fixing snow loads in constructions without walls, for which high values of snow load, superimposed to horizontal wind action or horizontal earthquake acceleration may cause failures.
The combination factors between different action given in "Basis of Design" have been calibrated upon national codes values and general reflections. So far, no systematic calculation checks have been performed for snow loads combined with other actions and no investigation has been made in order to evaluate the modifications which may be required for different geographical regions.
In designing for serviceability, functioning and appearance of construction or its parts and comfort of people must be achieved by checking the structure in appropriate load combinations similar to the ultimate limit states ones. The corresponding representative values are obviously dependent on snow dispersion, but are also strongly influenced by the duration of the snow

cover on the ground, which depends on the region's climate. An important research work should be carried out, in the whole Europe, for the determination of the correction factors to be applied to snow load values, for serviceability checks, distinguishing between short-term value and long-term value. Such a research, performed for Switzerland only until now, would permit to execute serviceability verifications whit special reference to long term effects of great importance, for example, for timber structures.

5. Conclusions

The above illustrated "Background Document for Snow Loads" collects all the studies and the most significant issues encountered by the Project Team during the Code's elaboration. It is addressed to Engineers and to National Authorities charged to prepare national codes, in order to illustrate the underlying rational for and scientific basis of the choices made in Eurocode 1. Until now the still open problems are a lot and they regard all the fields of study.
There is no doubt that the ENV-1991-2-3 represents only a good base for the harmonization of the snow load on structures and are therefore necessary more detailed studies and researches in order to achieve a more faithful standardization.
Similar items arise in other codes on snow loads, such as ISO 4355 (ed. 1981 and ed. 1993), which do not solve the open problems mentioned above, especially for extremely non homogeneous regions such as Europe, extending from North Cape to Sicily, where climate conditions present widely varying features.
The "Background Document", in author's opinion, is an helpful publication and, supported by the research work that is to be developed, will lead to an improvement of the Code from the ENV phase towards the EN one.

References

The Background Document for Snow Loads presents, as already mentioned, an extremely wide bibliography, updated to 1994. Thus, for a complete bibliography list, it is made reference to the document itself.

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EC 1: Wind Actions

EC 1: Actions du vent

EC 1: Windeinwirkungen

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SUMMARY

A description of the confirmed draft of the European Wind Load Code ENV 1991-2-3 "Wind Actions" is presented. The code includes static actions as well as dynamic actions. For those structures which are less sensitive to dynamic effects, a simplified method is presented. Other structures must be calculated with the detailed method. For the decision between simplified and detailed method, criteria are given which are based on calculations with the detailed method. Some examples are presented.

RÉSUMÉ

On présentera la description du projet de la Norme Européenne (Eurocode) ENV 1991-2-3 "actions du vent" qui a été approuvé par le SC 1. La norme contient les actions statiques et dynamiques du vent. Une méthode de calcul simplifiée sera indiquée pour les structures qui ne sont pas susceptibles de vibrations. Les autres structures doivent être calculées en employant un procédé de calcul détaillé. Afin de faciliter la décision entre les deux procédés, on donnera des critères qui sont basés sur la méthode détaillée. En outre, on présentera quelques exemples de calcul.

ZUSAMMENFASSUNG

Es wird eine Beschreibung des vom SC 1 bestätigten Entwurfs des Eurocodes ENV 1991-2-3 "Windeinwirkungen" vorgestellt. Der Code enthält sowohl die statischen als auch die dynamischen Windeinwirkungen. Für solche Strukturen, die nicht schwinganfällig sind, wird eine vereinfachte Berechnungsmethode angegeben. Die anderen Strukturen müssen nach einem detailierten Berechnungsverfahren berechnet werden. Für die Entscheidung, ob das vereinfachte oder die detaillierte Verfahren angewendet werden muß, sind Kriterien angegeben, die auf der detaillierten Methode beruhen. Es werden einige Berechnungsbeispiele angegeben.

1. PRINCIPLES

The draft of the Eurocode "WIND ACTION" [1, 2], has been started from the ISO T 98 "Wind Action" [3] and it has been developed to a code proposal which can be applied to most of the common buildings and structures. To achieve the design aims of a structure account shall be taken of

- turbulent wind acting over part or all of the structure
- static and fluctuating pressures induced by the wake behind the structure
- fluctuating forces induced by the motion of the structure

The wind load is presented either as a wind pressure or a wind force resp. wind moment. The response of structures due to wind action is divided into the following types:

- static response
- stochastic and resonant response due to turbulence and wake effects
- vortex resonance
- galloping
- interference
- divergence and flutter

Structures of an unusual nature, complexty or size i.e. structures or structural parts higher than 200 m, bridges longer than 200 m, suspended bridges and guyed masts are not yet completely covered by this code and may require special engineering study. Some rules for these structures are incorporated during the ENV period.

2. WIND PRESSURE AND WIND FORCES

2.1. Wind pressure on surfaces, $w_{e,i}$

The wind pressure on surfaces given in this code is valid for surfaces which are sufficiently rigid to neglect their resonant vibration caused by the wind. The pressure is described as an

- external pressure $w_e = q_{ref} \cdot c_e\ (z_e) \cdot c_{pe}$ (1)
- internal pressure $w_i = q_{ref} \cdot c_e\ (z_i) \cdot c_{pi}$ (2)

and the net pressure is

$$w_{net} = w_e - w_i \tag{3}$$

where: q_{ref} = reference mean wind velocity pressure = $\rho/2\ v_{ref}^2$ (see 3.1)
$c_e\ (z_e, z_i)$ = exposure coefficient (see 3.2.2) which includes the effects of the wind profile and of the topography
$c_{pe,i}$ = external (e) and internal (i) pressure coefficients derived from a coefficient catalogue
$z_{e,i}$ = reference height defined together with the $c_{pe,i}$-values.

2.2. Wind force, F_w

The global force, F_w, which results from the pressure distribution (without friction forces) shall be obtained from the following expression

$$F_w = q_{ref} \cdot c_e(z_e) \cdot c_d \cdot c_f \cdot A_{ref} \tag{4}$$

where: c_f = force coefficient
A_{ref} = reference area for c_f
c_d = dynamic factor, which takes into account the aerodynamic admission and the resonant gust response and is ≤ 1 for structures which are not sensitive to vibrations
q_{ref}, $c_e\ (z_e)$, z_e defined as before

If not otherwise specfied, the resultant wind force on non circular and nearly symmetric structures, F_w, may be assumed to act with an eccentricity

$$e = b/10 \tag{5}$$

where: b = largest dimension of the cross section

2.3. Friction force, F_{fr}

For structures with large areas swept by the wind (i.e. large free standing roofs), friction forces, F_{fr}, may be important. They shall be obtained from:

$$F_{fr} = q_{ref} \cdot c_e(z_e) \cdot c_{fr} \cdot A_{fr} \tag{6}$$

where: c_{fr} = friction coefficient
A_{fr} = area swept by the wind
q_{ref}, $c_e(z_e)$ defined as before.

3. REFERENCE WIND AND WIND COEFFICIENTS

3.1. Reference wind velocity

The reference wind velocity, v_{ref}, is defined as

- the 10 min mean wind velocity
- at 10 m above ground of terrain category II (see Table 1)
- with an annual probability of exceedence of 0,02 (50 year return period).
  For other annual probabilities of exceedence a calculation formula is given.

3.2. Wind coefficients

3.2.1. Coefficients for the reference wind velocity.

$$v_{ref} = c_{DIR} \cdot c_{TEM} \cdot c_{ALT} \cdot v_{ref,0} \tag{7}$$

where: $v_{ref,0}$ = basic value of the reference wind velocity at 10 m above sea level given in the national wind maps which are presented in an Annex.
c_{DIR} = direction factor, which takes into account the probability of wind speed depending on the wind direction. It is taken as 1,0 unless otherwise specified in the national wind maps.
c_{TEM} = temporary (seasonal) factor which takes into account the probability of wind speed for structures which are
- structures during construction and which may require temporary bracing supports
- structures whose life time is known and less than one year.

Unless otherwise specified in the national wind maps, c_{TEM} is taken as 1,0.
c_{ALT} = altitude factor which takes into account the altitude level of the site and is to be taken as 1,0 unless otherwise specified in the national wind maps.

3.2.2. Coefficients for the mean wind velocity at height z.

The mean wind velocity at height, z, at the site of the structure depends on the roughness of the terrain in the direction from where the wind is blowing and on topographical effects (hills, escarpments etc.). It is given by

$$v_m(z) = c_r(z) \cdot c_t(z) \cdot v_{ref} \tag{8}$$

where: $c_r(z)$ = roughness coefficient at height z
$c_t(z)$ = topography coefficient at height z

The roughness coefficient describes the effect of the terrain roughness and is defined by a logarithmic law (velocity profile). It shall be calculated by

$$c_r(z) = k_r \cdot \ln(z/z_0) \quad \text{for } z_{min} \le z \tag{9}$$
$$c_r(z) = c_r(z_{min}) \quad \text{for } z < z_{min}$$

Four different terrain categories are defined and given in Table 1 together with the parameters

k_r = terrain factor
z_0 = roughness length
z_{min} = minimum height

When there is any doubt about the choice between two categories in the definition of a given area, the worst case should be taken.

	terrain category	k_r	z_0[m]	z_{min}[m]	ϵ
I	Rough open sea, Lakes with at least 5 km fetch upwind and smooth flat country without obstacles	0,17	0,01	2	0,13
II	Farmland with boundary hedges, occasional small farm structures, houses or trees	0,19	0.05	4	0,26
III	Suburban or industrial areas and permanent forests	0,22	0,3	8	0,37
IV	Urban areas in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m	0,24	1	16	0,46

Table 1: Terrain categories and related parameters (The parameter ϵ is used for the calculation of the integral length scale in the detailed procedure for c_d)

If a structure is situated near a change of terrain roughness, a simple procedure is given in the Code.
Where detailed knowledge of the influence of landscape on the wind profile is available, detailed rules to take into account the transition may be adopted. The topography coefficient, c_t (z), accounts for the increase of mean wind velocity over isolated hills and escarpments and is given in the code by a formula and two diagrams. Otherwise it is set to 1,0.

3.2.3. The exposure coefficient, c_e (z) and the dynamic factor, c_d

The exposure coefficient, c_e (z), takes into account the effects of terrain roughness, topography, turbulence and height above ground on the mean wind speed.
It is developed from the gust response factor, G [4], which itself is not used in its classical expression in the Eurocode. Starting from the basic expression for the quasi static design wind pressure, q(z), in the height, z, above ground:

$$q(z) = q_{ref} \cdot c_r^2(z) \cdot c_t^2(z) \cdot G \tag{10}$$

$$G = 1 + 2 \cdot g \cdot I_v(z) \cdot \sqrt{Q_o^2 + R_x^2} \tag{11}$$

where: g = peak factor
$I_v(z)$ = turbulence intensity in the height z above ground
Q_o^2 = background part of the gust response
R_x^2 = resonant part of the gust response
$c_r(z)$, $c_t(z)$, q_{ref} defined as before

and expanding the equation with $(1 + 2 \cdot g \cdot I_v(z))$ we receive the following expression for q(z):

$$q(z) = q_{ref} \cdot c_r^2(z) \cdot c_t^2(z) \cdot (1 + 2 \cdot g \cdot I_v(z)) \frac{1 + 2 \cdot g \cdot I_v(z) \cdot \sqrt{Q_o^2 + Q_x^2}}{1 + 2 \cdot g \cdot I_v(z)}$$

In the first bracket the turbulence intensity $I_v(z)$ is replaced by

$$I_v(z) = \frac{k_r}{c_r(z) \cdot c_t(z)} \tag{12}$$

thus:

$$q(z) = q_{ref} \cdot \left[c_r^2(z) \cdot c_t^2(z) \cdot \left(1 + 2 \cdot g \frac{k_r}{c_r(z) \cdot c_t(z)}\right)\right] \frac{1 + 2 \cdot g \cdot I_v(z) \cdot \sqrt{Q_o^2 + Q_x^2}}{1 + 2 \cdot g \cdot I_v(z)} \tag{13}$$

The expression in the first bracket is called "exposure coefficient", $c_e(z)$:

$$c_e(z) = c_r^2(z) \cdot c_t^2(z) \left(1 + 2 \cdot g \frac{k_r}{c_r(z) \cdot c_z(z)}\right) \tag{14}$$

and the quotient is called "dynamic factor", c_d:

$$c_d = \frac{1 + 2 \cdot g \cdot I_v(z) \cdot \sqrt{Q_o^2 + R_x^2}}{1 + 2 \cdot g \cdot I_v(z)} \tag{15}$$

The dynamic factor, c_d, is described in chapter 5.2 in more detail. The peak factor, g, can be approximated by g = 3.5, thus

$$c_e(z) = c_r^2(z) \cdot c_t^2(z) \left[1 + \frac{7 \cdot k_r}{c_r(z) \cdot c_t(z)}\right] \tag{16}$$

For the most common cases, $c_t(z)=1$, the exposure coefficient is illustrated in Figure 1.

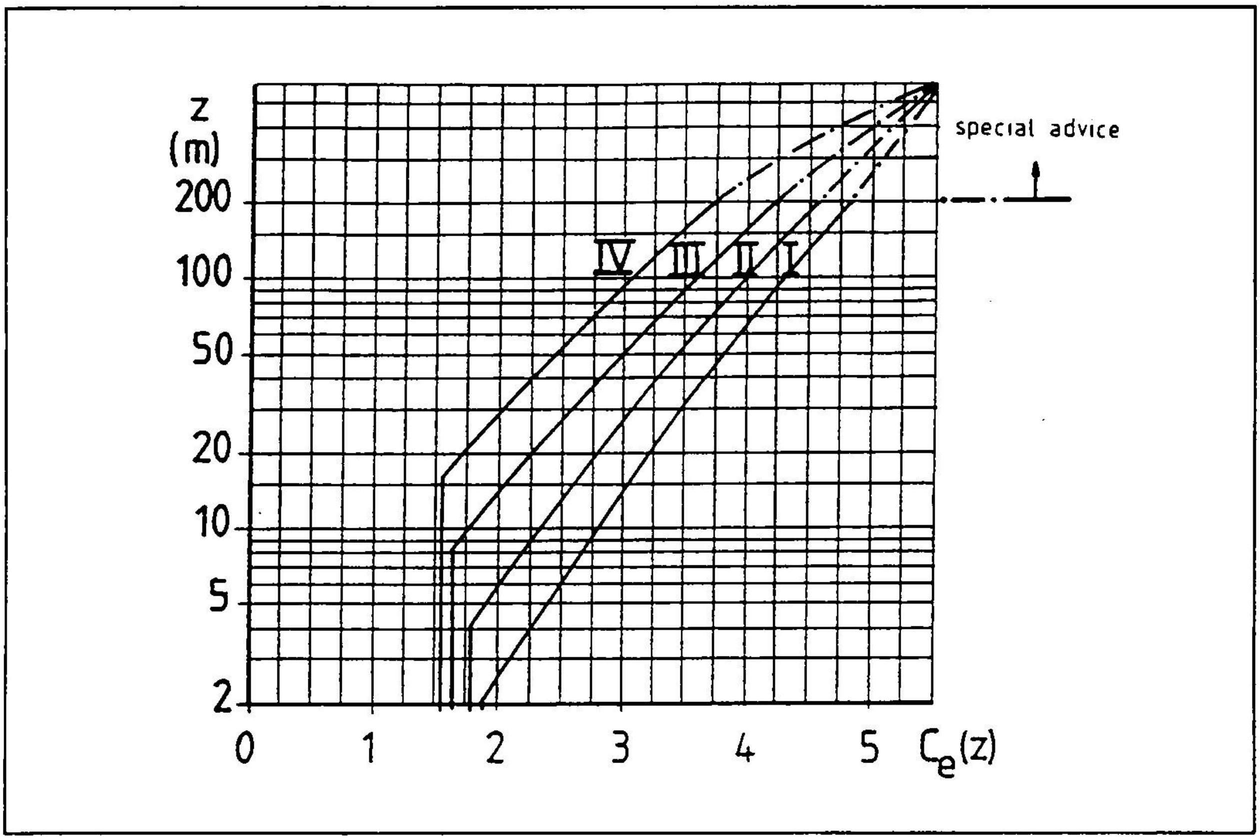


Figure 1: Exposure coefficient $c_e(z)$ as a function of height z above ground and terrain roughness category for c_t = 1.

4. AERODYNAMIC COEFFICIENTS

The Eurocode presents aerodynamic coefficients for the following structures, structural elements and components:

- buildings, including building walls, different types of roofs, internal pressure
- canopy roofs and multispan roofs
- free standing boundary walls, fences and signboards
- structural elements with rectangular, sharp edged and regular polygonal section
- circular cylinders and spheres
- lattice structures and scaffoldings
- bridges
- flags
- slenderness effect

Each coefficient is referred to a reference area and a reference height, which are defined for that particular coefficient. In the following, some explanations are given for some coefficients without presenting the whole coefficient catalogue.

4.1. Buildings, roofs and walls

The wind load for buildings, roofs and walls are presented by pressure coefficients for internal and external pressure [6, 7]. In order to take into account the reduction of the mean value of wind pressure by the integrating effect of a larger loaded area (size effect), the external pressure of the loaded area is given by the following rule:

$$c_{pe} = c_{pe,1} \qquad \text{for } A \leq 1\ m^2$$
$$c_{pe} = c_{pe,1} + (c_{pe,10} - c_{pe,1}) \log_{10} A \qquad \text{for } 1\ m^2 \leq A \leq 10\ m^2$$
$$c_{pe} = c_{pe,10} \qquad \text{for } A \geq 10\ m^2$$

where:

$c_{pe,1}$ = standard value of the external pressure coefficient corresponding to an area of 1 m^2
$c_{pe,10}$ = standard value of the external pressure coefficient corresponding to an area of 10 m^2

The values for $c_{pe,10}$ and $c_{pe,1}$ are given for orthogonal wind directions and represent the highest values obtained in the range of wind direction $\pm$ 45° either side.
More detailed information about pressure coefficients for special wind directions may be obtained from the background paper, which is also available.
The internal pressure for buildings is described depending on the ratio of openings in the walls. The values are based on numerous experimental investigations on model and full-scale buildings and on theoretical considerations. The advantage of the presentation in form of the opening ratios is the fact, that the knowledge of the absolute value of the openings must not be known. The presented values for c_{pi} are valid for buildings without partition walls but under specific conditions they can also be applied to buildings with partitions walls if assumptions for the internal openings (i.e. opened doors) can be made. Other critical cases has to be considered which are mentioned in the Eurocode.
The pressure coefficients for free standing walls, fences and signboards include the influence of return corners, the solidity and shelter effects. The wind load for sign boards are described by a force coefficient combined with a slenderness factor.

4.2. Structural elements, circular cylinders and spheres

The wind forces on structural elements as rectangular sharp edged and polygonal sections and circular cylinders are presented by aerodynamic force coefficients depending on the aspect ratio of the cross section (rectangular sections), the corner radii (rectangular and polygonal sections), Reynolds number and surface roughness (circular and polygonal sections and spheres). For circular sections pressure distributions are given for three Reynolds number ranges: subcritical, critical and transcritical range.
The effect of finite slenderness is included by a slenderness factor, where the slenderness is defined for the different application. An indication is given for cylinders and spheres near a plane surface.

4.3. Lattice structures and scaffoldings

Aerodynamic force coefficients for lattice structures based on model tests with full-scale Reynolds numbers, solidity ratios and slenderness are given for plane and spatial elements with members of sharp-edged and circular cross-sections.
The values for scaffoldings are restricted to the worst case of wind direction.

4.4. Bridges

The description of the wind load for bridges is derived in two parts:

(i) For those bridges which are less sensitive to wind, a global boxed[1] value of wind pressure is defined.
(ii) In general, aerodynamic force coefficients are given in alongwind, crosswind and longitudinal direction depending on the aspect ratio of the bridge deck and for different bridge deck types (two groups). A slenderness factor is included into the formula.

The reference area is described in detail, except of the description of the reference area due to traffic. This reference area depends on typical traffic situations and must be defined in the design codes for railway and road bridges.

4.5. Flags and friction force

For free flags a formula for determining the force coefficient is presented. It is based on tests with full-scale flags and describes the wind load including a dynamic factor caused by the fluttering of the flags.

[1] A boxed value means: Each country may define its own value referred to the special situation in that country.

For large areas swept by the wind (i.e. large free standing roofs or long free standing boundary walls) the friction force coefficient is given for three different surface roughnesses.

4.6. Slenderness reduction factor, ψ_λ

The influence of the slenderness is taken into account by a slenderness reduction factor, ψ_λ. It is presented versus the effective slenderness, λ, and the solidity ratio, φ. The effective slenderness, λ, is defined in a table for the different boundary conditions. Worth to mention is, that for vertical structures placed on the ground the wind boundary layer causes flow disturbances at the support and reduces the aerodynamic correlation on the structure. The effective slenderness for those structures is defined by l/d and not by 2 l/d (mirror analogon).

5. DYNAMIC FACTOR FOR GUST RESPONSE

As shown in chapter 3.2.3 the dynamic factor, c_d, takes into account the reduction effect due to the lack of correlation of pressures over surfaces and the magnification effects due to the frequency content of turbulence close to the fundamental frequency of the structure.
In order to evaluate the dynamic factor, c_d, two procedures can be applied:

(i) simple procedure
(ii) detailed procedure.

The simple procedure has been developed for buildings , chimneys and bridges which are less sensitive to dynamic response. The dynamic factor for those structures is less or near 1.
Based on the detailed procedure and with approximations of natural frequencies and damping, criteria have been developed for the field of application of the simplified procedure, which provides conservative results.
In the following chapter the field of application is presented.

5.1. Field of application

In Fig. 2a - c the field of application of the simplified procedure is given for buildings (concrete, steel and mixed material). In order to avoid an abrupt change from one to the other procedure, the c_d-value has been included as a parameter for the range of $0{,}9 \le c_d \le 1{,}2$. Most of the common buildings may be calculated with the simplified procedure. Only few extreme buildings must be handled with the detailed procedure. In general it is allowed to use the detailed procedure for all buildings, but it is recommended to do so if $c_d > 1$ or/and the structural data are not close to the data indicated in Figure 2 to 4.
The calculation with the detailed procedure for roadway and railway bridges provides $c_d < 1$. Therefore the simplified procedure may be applied for those bridges of span $l \le 200$ m. The dynamic factor c_d can be taken from Figure 3.
Figure 4a - d shows the field of application for chimneys. The criterion in Fig. 4 is related to gust wind response. The vortex shedding phenomenon which is important for chimneys is indicated, too, and is described in chapter 6 in more detail.

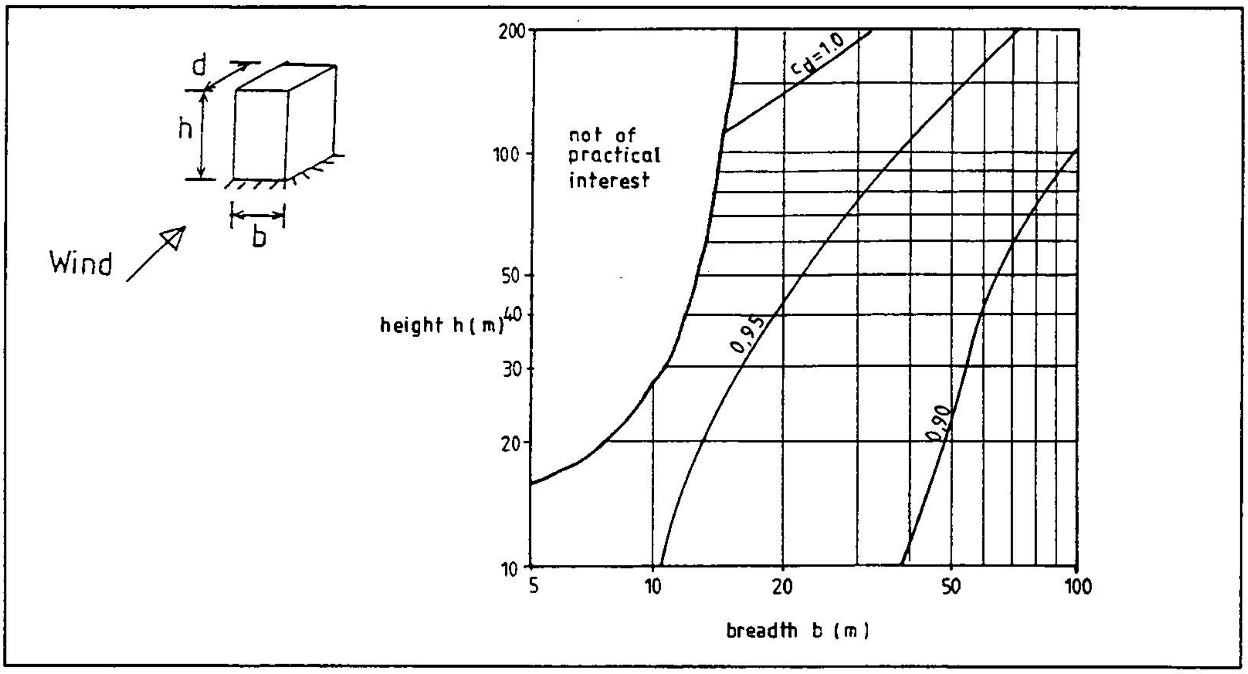


Figure 2a: Concrete structure, $\delta = 0{,}045\, n_1 + 0{,}05$

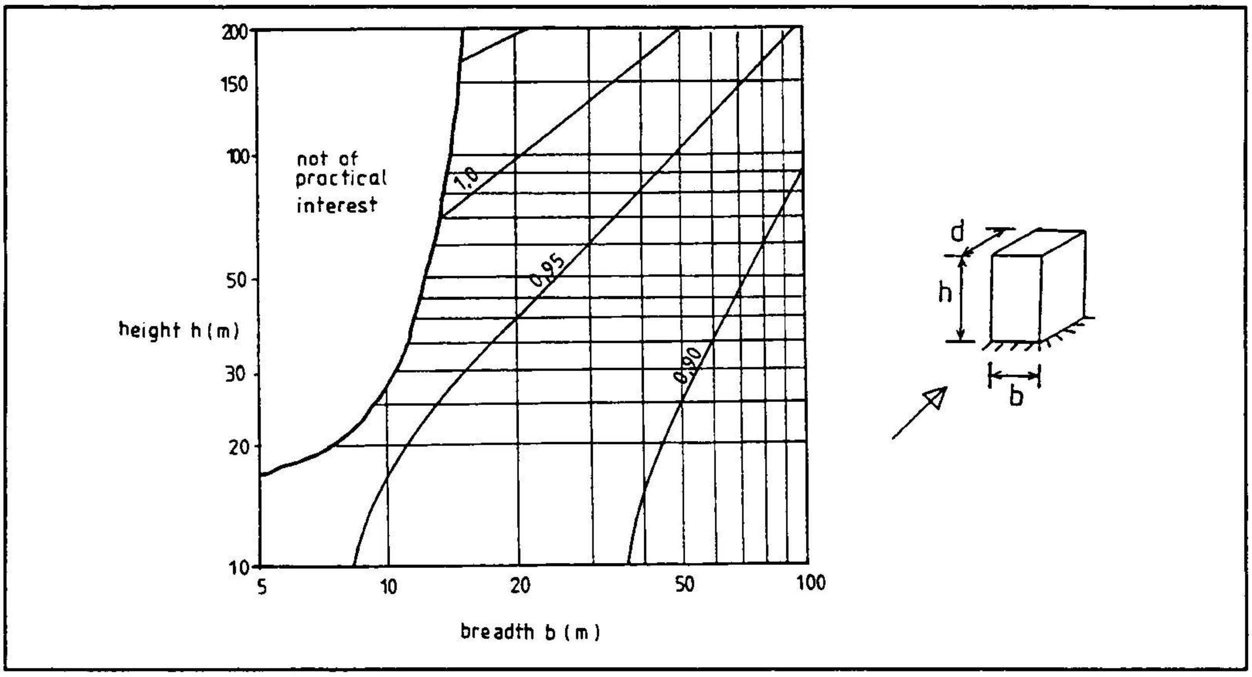


Figure 2b: Mixed structure steel-concrete, $\delta = 0{,}08$ $n_1 > 0{,}08$

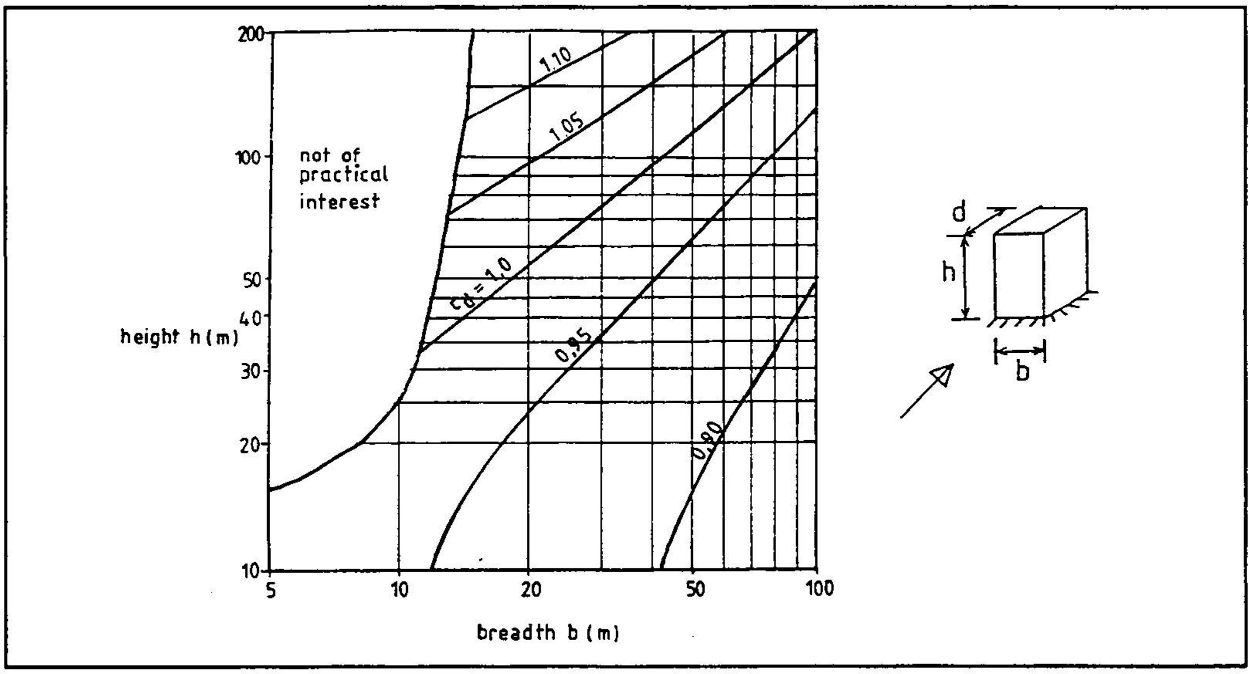


Figure 2c: Steel structure, $\delta = 0{,}045$ $n_1 > 0{,}05$

Figure 2: Field of application for the simplified procedure for buildings. Approximation for the natural frequency: $n_1 = 46/h$ where $[h] = m$

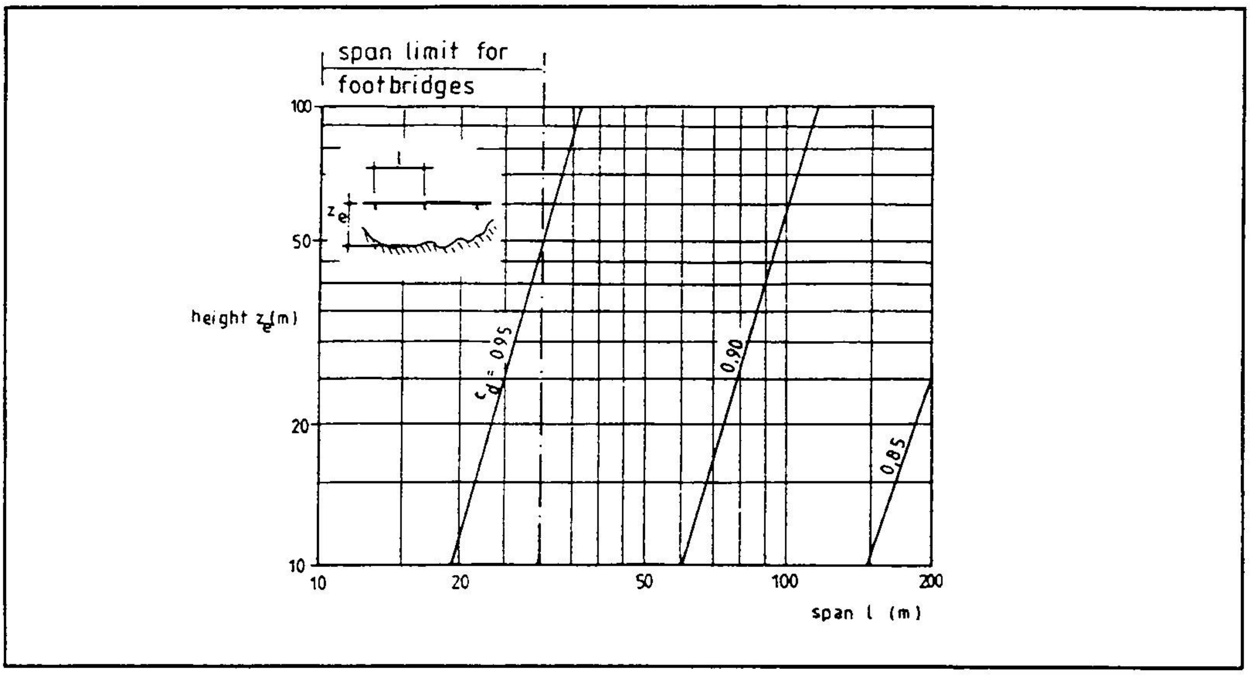


Figure 3: Dynamic factor, c_d, for roadway, railway and footbridges

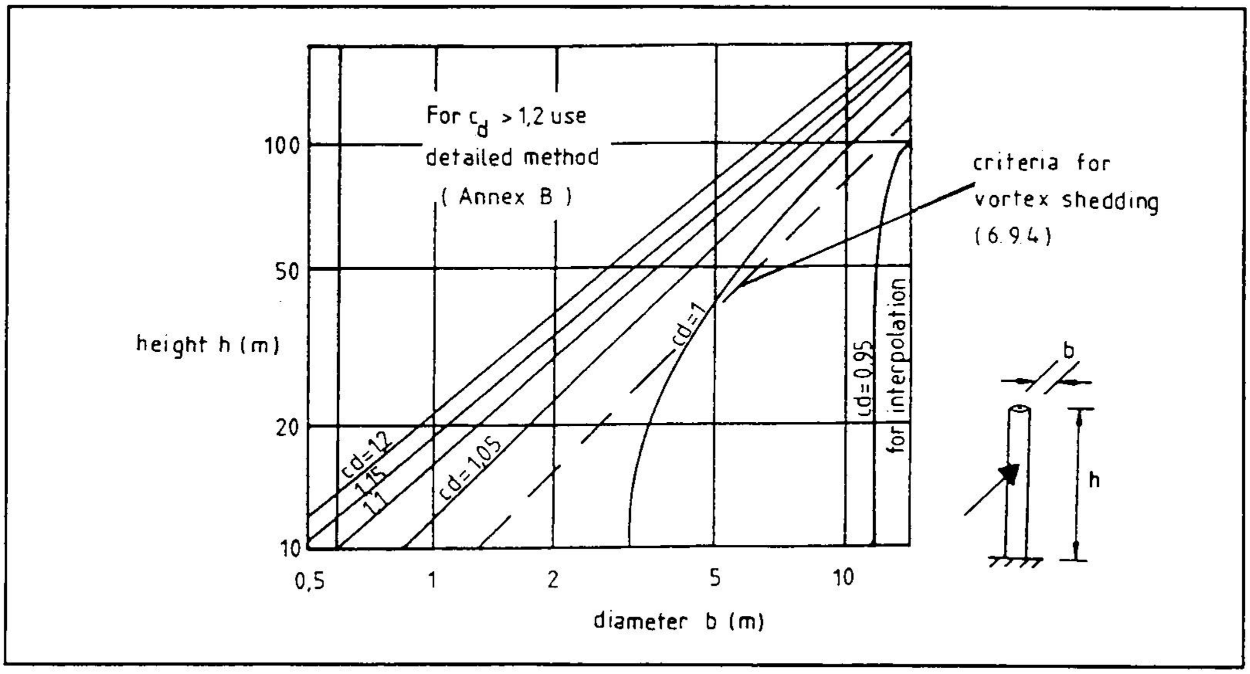


Figure 4a: Field of application for chimneys, values for unlined welded chimneys

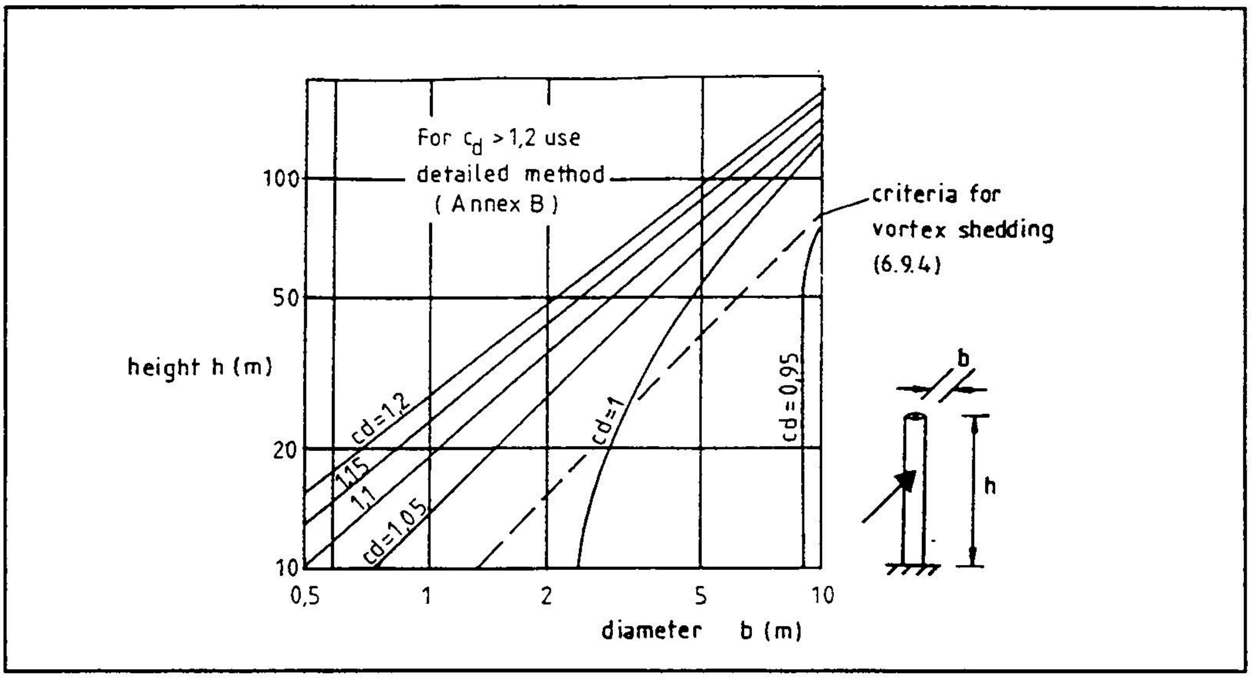


Figure 4b: Field of application for chimneys, values for lined steel chimneys

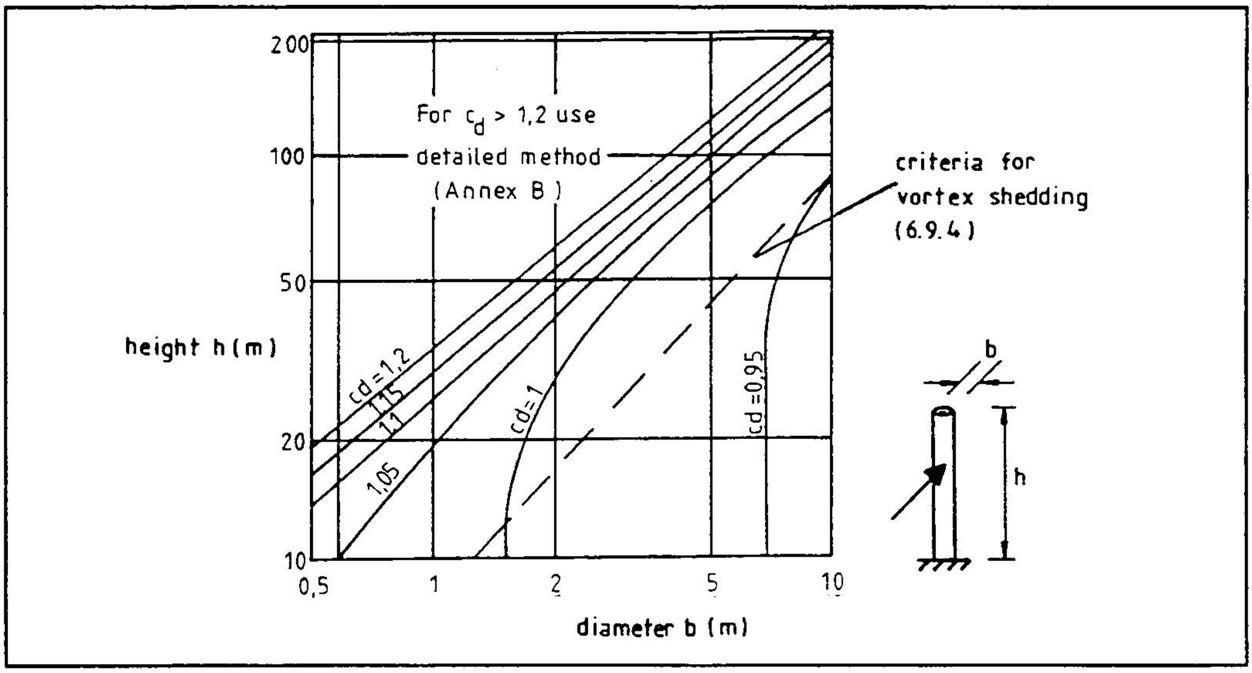


Figure 4c: Field of application for chimneys, values for brick lined steel chimneys

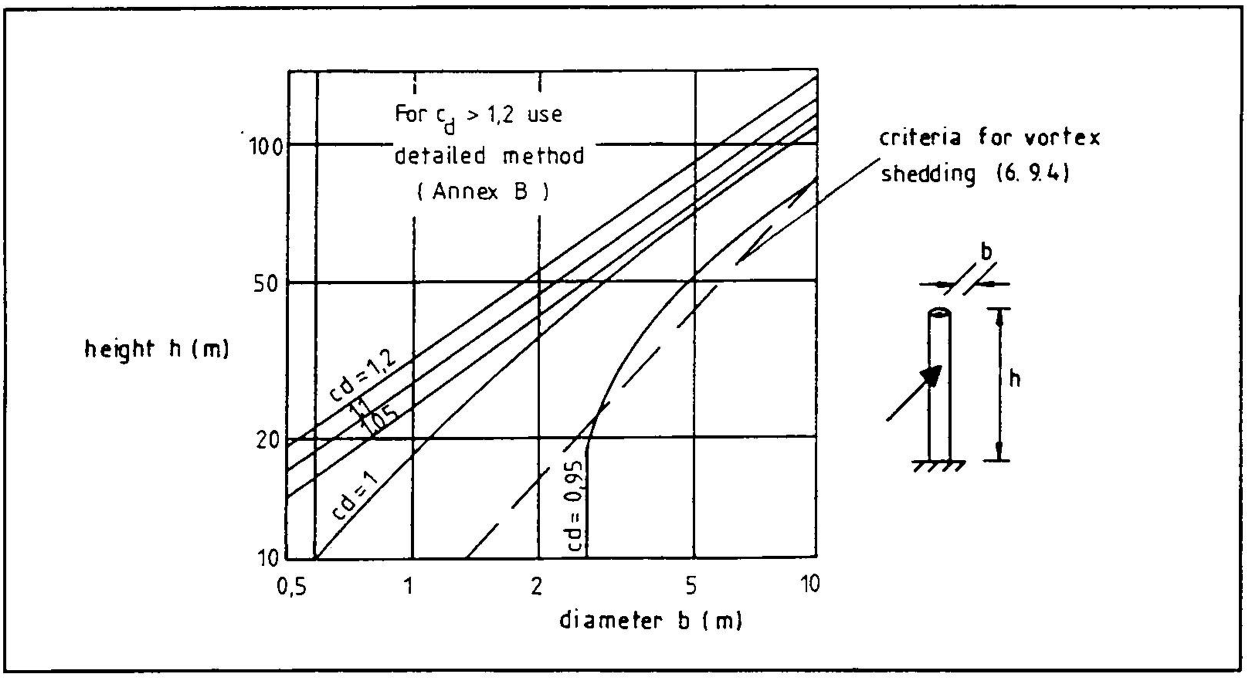


Figure 4d: Field of application for chimneys, values for reinforced concrete chimneys

In Figure 4a to 4d the following values have been used (δ may be used for the vortex resonance calculation):

$$\eta = \frac{\epsilon_1}{h_{eff}^2} b \sqrt{\frac{w_s}{w_z}}$$

	Material	δ	ϵ_1
a)	concrete	$0{,}05 \cdot n_1 > 0{,}025$	700
b)	steel or aluminium with brick liner	0,07	1000
c)	Lined steel or aluminium	0,025	1000
d)	Unlined welded steel or aluminium	0,015	1000

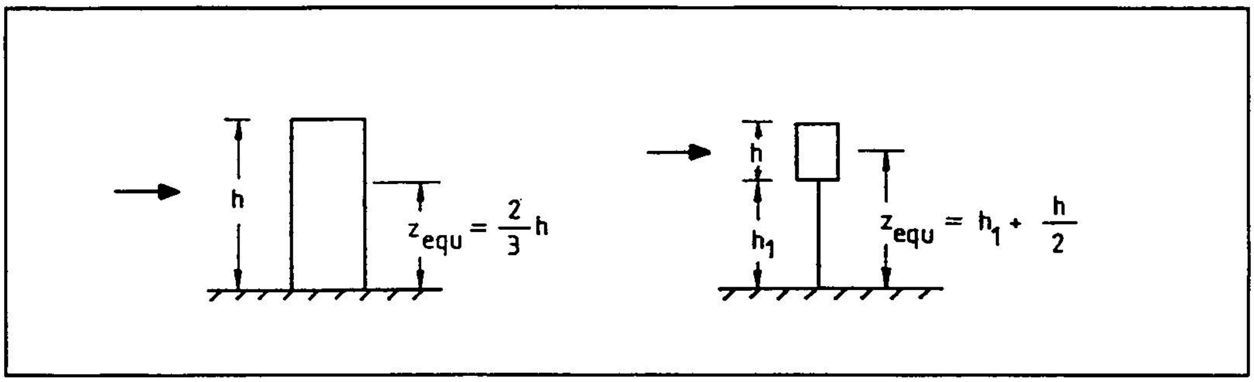
b = diameter w_s = weight of the structural part
h_{eff} = effective height w_z = total weight

6.2. The detailed procedure for c_d

The dynamic factor, c_d, is defined by

$$c_d = \frac{1 + 2 \cdot g \cdot I_v (z_{equ}) \; \overline{Q_0^2 + R_x^2}}{1 + 7 \cdot I_v (z_{equ})} \qquad (17)$$

where: z_{equ} = equivalent height of the structure



The quantities

g	=	peak factor
$I_v (z_{equ})$	=	turbulence intensity
Q_0	=	background part of the gust response
R_x	=	resonant part of the gust response

are presented by mathematically expressions which allow a numerical calculation with computers. For a quick check and for illustration the parameters are presented in diagrams, too.
In order to evaluate the serviceability of the structure in respect to alongwind vibration an expression is given to calculate the displacements and accelerations.
Finally, interference factors are presented for high-rise buildings in tandem or grouped arrangement effected by wake buffeting.

6. VORTEX SHEDDING

Slender structures such as chimneys, observation towers, component elements of open frames and trusses, bridges and in some cases high rise buildings shall be designed to resist the dynamic effect of vortex shedding. The shedding of vortices from unstiffened cylindrical shells may in addition excite ovalling oscillations. The field of application is given by the criteria in Figure 5 and 6 which implies the limit of

$$v_{crit} \leq 1{,}25\ v_m \qquad (18)$$

where v_m = design wind speed (see chapter 3.2.2).

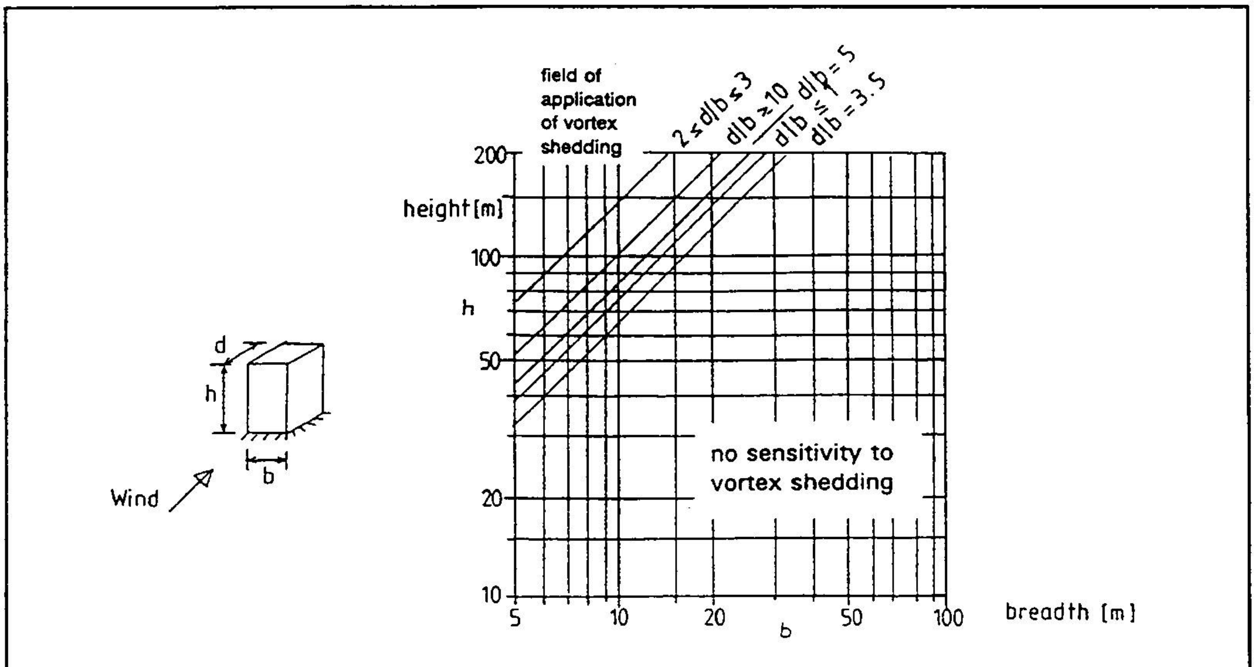
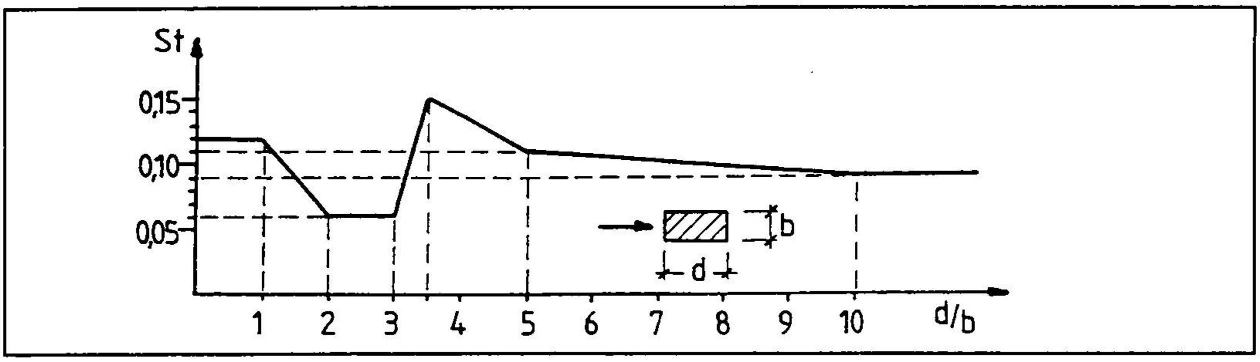


Figure 5: Criteria for the field of application of vortex shedding for buildings

Note: The effect of the d/b ratio is based on the Strouhal number of a rectangular cross section, as shown in the following figure:



Strouhal, St, number for rectangular cross sections with sharp corners

	Criteria satisfied provided:			
Elongated structures like chimneys, posts, towers (h = height, b = diameter	h/b< 8			
Bridges	l ≤ 200m			
Types of supports for horizontal forces		$\frac{d}{b} \le 5$	$5<\frac{d}{b}<10$	$\frac{d}{b} \ge 10$
	l/b	8	linear interpolation	14
		16		29
		24		44
		32		58

Figure 6: Criteria for the field of application of vortex shedding galloping interference galloping and flutter for elongated structures

In order to calculate the critical wind speed

$$v_{crit} = (b \cdot n_i)/St \tag{19}$$

Strouhal numbers St are given for

- rectangular sections
- circular cylinders
- sharp edged sections
- bridge decks

and the natural frequency n_i may be estimated by using the formulae given in a special Annex (see chapter 8).
For the calculation of the maximum vortex resonance amplitude the correlation length model is presented. This model has been developed including the existing knowledge of that phenomena and to present a stable solution for structures in natural wind. The model has been checked and verified with a lot of full scale measurements during the past 18 years [5]. The maximum amplitude at the top of the chimney is given by equ. (20).

$$\frac{\max y_F}{b} = K_w \cdot K \cdot c_{lat} \frac{1}{St^2} \frac{1}{Sc} \qquad (20)$$

where: b = reference width of the cross section
K_w = effective correlation length factor $0,1 \leq K_w \leq 0,6$
K = mode shape factor $0,1 \leq K \leq 0,14$
c_{lat} = aerodynamic force coefficient, given for the cross sections as listed above
St = Strouhal number
Sc = Scruton number = $2\ m\delta/(\rho b^2)$

The most important point of the correlation length model is the calculation of the effective correlation length factor, K_w which includes the locking-in effect and the type of response (random or harmonic). For large amplitudes ($\max y_F/b > 0,1$) K_w must be calculated by an iterative procedure. Simple formulae are presented for common cases.
The correlation length model cannot only be applied to cantilevered or simple supported structures but also to more complicated structures, like spatial lattice structures, frame structures or guyed masts. The handling for those systems is described in the Eurocode.
Vortex-excited resonance vibrations may cause fatigue problems. Therefore a calculation rule is presented to estimate the stress and the number of stress cycles.

7. AEROELASTIC INSTABILITIES AND INTERFERENCE EFFECTS

The following phenomena are described

- galloping instability
- interference galloping
- bridge flutter
- divergence

Criteria and calculation procedures are presented for the onset velocity resp. divergence velocity for galloping, interference galloping and divergence. Bridge flutter stability should be calculated by solving the flutter equation or with model tests and is not presented in detail. Simplified rules availabe in literature may be used provided that they have been agreed with the relevant authorities.

8. DYNAMIC CHARACTERISTICS

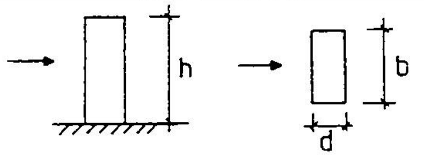
For the calculation of the dynamic effects the natural frequency, the damping, the mode shape and the equivalent mass of the structure must be known. In most cases it is sufficient to have a good approximation of that values. In a special chapter these dynamic characteristics of the most important structures are described. For more complicated structures a modal analysis is recommended.

9. FINAL REMARKS

The new draft of the Eurocode "Wind Action" presents calculation procedures of wind pressure and wind forces for the most common buildings and structures for static loads as well as for dynamic effects. It was the aim to draft a code with includes modern calculation procedures and verified aerodynamic coefficients which produces realistic load values. The most important influcence parameters have been left separated and are not combined together so that the structure of the code enables to add or to reduce values as available and to follow an increase of knowledge and experiences in the field of wind action.
For the description of the wind characteristics a format is given and the regional values are presented in different national wind maps. Every country may introduce simplifications or more sophisticated descriptions of wind parameters, if available.
For structures which are less sensitive to dynamic effects, the calculation procedure becomes simple and is restricted to only a few calculation steps.
All design codes of the Eurocode will refer to this wind action code, so that the wind load will be calculated in an identical manner for all buildings and structures.
During the following ENV period it may happen, that some supplements will be included if the design codes request for it.

10. EXAMPLES

10.1 Steel building of different height



h	=	10 + 200 m
b	=	20 m
d	=	10 m

The building is situated in urban area (category 2) with a reference wind speed of v_{ref} = 27,5 m/s. The force coefficient is set to constant, c_t = 1,3. From the criterion of Fig. 2 the simplified method may be applied up to a height of 50 m. In Fig. 7 the calculated wind force per m², F_w/A_{ref} is plotted against the building height h for the simplified and the detailed method. For h > 50 m the result of simplified method is above the result obtained by the detailed method, while for h > 50 m the detailed method has to be applied because of increasing dynamic response.

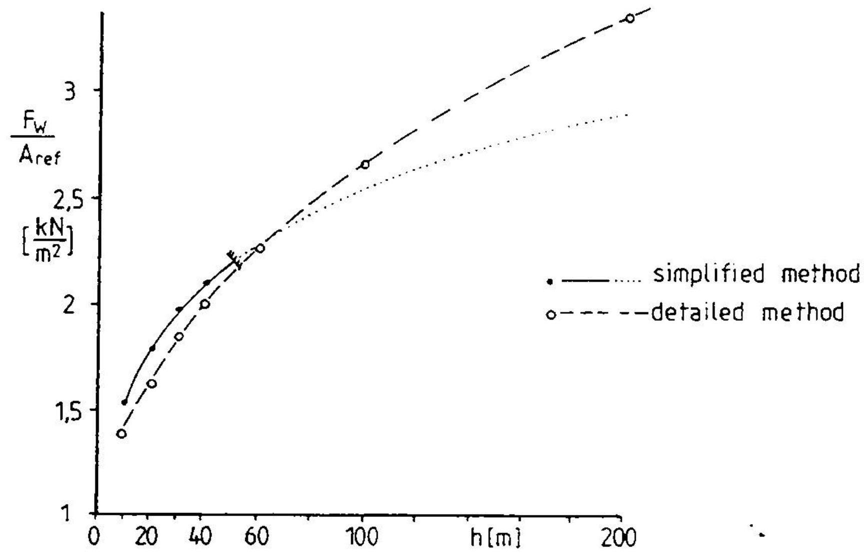
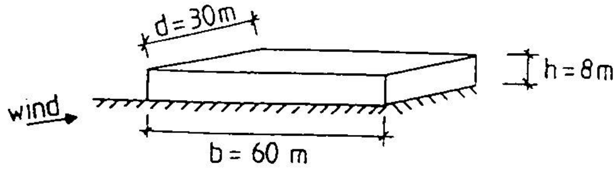


Figure 7: Calculated wind force per m² for the buildings of example 5.1. Comparision of simplified method with detailed method.

10.2 Large low rise steel hall

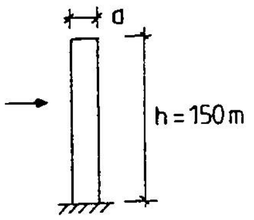


under the same site condition as in example 11.1 we get for the

simplified method: F_w/A_{ref} = 1,40 kN/m²
detailed method: F_w/A_{ref} = 1,19 kN/m²

i.e. the simplified method presents nearly 18 % higher wind loads. The reason for this fact in the neglected size effect in the simplified method.

10.3 Concrete tower



v_{ref} = 27,5 m/s

roughness category 2

10.3.1 Small slenderness, d = 13 m:

$$\lambda = h/d = 11{,}5 < 12$$

From the criteria for towers and stacks this structures may be calculated with the simplified method. Both calculations, simplified and detailed method come to the same result

$$\frac{F_w}{A_{ref}} = 1{,}47 \text{ kN/m}^2$$

The reason for the good agreement is that the size effect as well as the dynamic effect may be negligible.

10.3.2 Large slenderness, d = 5 m

$$\lambda = h/d = 30 > 12$$

This structure has to be calculated with the detailed method. The along wind force is

$$\frac{F_w}{A_{ref}} = 2{,}23 \text{ kN/m}^2$$

and is 52 % above the result which would be received with the simplified method.

11. REFERENCES

[1] ENV 1991-2-3:Eurocode 1, Basis of Design and Action on Structures, part 2.3: Wind Actions. Approved proposal, Dec. 1993

[2] Project Team PT 5 (Editor: H. Ruscheweyh): Background document to ENV 1991-2-3, Lehrstuhl für Stahlbau, RWTH Aachen, April 1994

[3] ISO/TC 98/SC 3: Wind Action on Structures, 1990

[4] G. Solari: Tubulence modeling for gust loading, J. Struct. Eng., ASCE, 113, 7, 1550-1569 (1987)

[5] H. Ruscheweyh, G. Sedlacek: Crosswind vibration of steel stacks - Critical comparison between some recently proposed codes. J. of Wind Eng. and Industrial Aerodyn., Vol. 30 (1988) pp. 173-183

[6] N.J. Cook: The designer's guide to wind loading of building structures, part 2, BRE, Butterworth London, 1985

[7] ECCS TC 12: Recommendations for calculating the effects of wind on buildings and structures, No. 52, 2nd edition, 1987, ECCS Secret., Bruxelles, Av. des Ombrages, 32/36 bte 20

ENV 1991 Part 3 : The main models of traffic loads on road bridges Background studies

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SUMMARY

Part 3 of Eurocode 1 defines the traffic load models to be used for the design of road bridges, footbridges and railway bridges, for serviceability, ultimate resistance and fatigue verifications. For road bridges, several load models are proposed for serviceability and ultimate resistance verifications : one of them (Load Model 1 or LM1) is the main system. The background studies related with the calibration of this main system are presented.

Introduction

The work for the definition of traffic loads on road bridges and footbridges started in 1987. A working group was appointed by the EEC, convened by Ir. Gl. H. Mathieu. At the end of 1991, the work for the development of what is now included in ENV 1991 Part3 was transferred to the CEN and allocated to a project-team (SC1/PT6), the composition of which is given in Table 1.

Members	Associated experts
Ir. Gl. H. Mathieu (Convenor)	Pr. Calgaro
Ir. Gulvanessian (SC1 Technical secretary)	Dr. Ir. Croce
Pr. Bruls	Dr. Ir. Jacob
Dr. Flint	Ir. Gilland
Pr. Sanpaolesi	Dr. Ir. Merzenich
Pr. Sedlacek	Ir. Prat

Table 1 - SC1/PT6 for road bridges

For road bridges, Part 3 of ENV1991 defines :

- Four load models for serviceability and ultimate resistance verifications (noted LM1 to LM4) : LM1, completed by LM2 for some specific local verifications, is the main model, LM3 includes a set of abnormal vehicles and LM4 represents the load due to a crowd on a bridge deck. These two last models are used, for a particular project, only if required by the client.
- Five models for fatigue verifications (noted FLM1 to FLM5) [6]

The present contribution gives information on the background studies wich led to the definition of LM1 and LM2.

1. Basic data on road traffic

When studies started, the available road traffic data consisted of :

- old data collected from 1977 to 1982 in France, Germany, Great Britain, Italy and The Netherlands ;
- recent data mostly collected in 1986 and 1987 in several countries. Four countries (France, Germany, Italy and Spain) had full records of traffic, including all the needed information about the axle weights of heavy vehicles, about the spacing between axles and between vehicles, and about the length of the vehicles.

Most of the recent data were recorded on the "slow lane" only (supporting the heaviest traffic) of motorways or main roads. The duration of the records varied from a few hours to more than 800 hours.

1.1 Traffic composition

The observed flow of heavy vehicles varied from 1000 to 8000 vehicles per day on the slow lane of motorways, from 600 to 1500 per day on the other roads. On the fast lanes of motorways or on secondary roads, this medium flow dropped to 100 or 200 vehicles per day.

The distribution of the distance between lorries appeared to follow a "gamma" type law with a mode between 20 and 100 m, a mean value varying from 300 to more than 1000 m and a large variation ratio (2 to 4). The most frequent types of vehicles were the double-axle vehicles and the articulated vehicles. The number of axles per vehicle, depending on the constructor, varied widely, but the histograms of their spacing showed three persistent modes with peak values particularly constant :

- 1,30 m for double and triple axles with a very small standard deviation,
- 3,20 m for the tractor axles of the articulated lorries, with a small standard deviation,
- 5,40 m for other spacings but with a widely scattered distribution.

1.1.2 Axle and vehicle weights

The axle weight was very scattered, with an average value around 60 kN, but the maximum weight corresponding to a mean return period of one day was not very different from a location to another one. Table 2 shows the range of the maximum weight per axle type, corresponding to a mean return period of one day.

	Single axles	Tandems	Tridems
Range of the maximum weight per day (kN)	130 to 210	240 to 340	220 to 390

Table 2

Even the maximum total weight of vehicles corresponding to a mean return period of one day was not very different from one location to another one, mostly in a range 400 - 650 kN. And all statistical distributions had two modes : the first one around 150 kN and the second one around 400 kN.

Finally, and in spite of some variations in the result of the measurements from one country to another one (these variations resulted mostly from the choice in traffic samples), the road traffic parameters turned out to be rather homogeneous, especially for the maximum daily values of the axle and vehicle weights.

2. Procedure for the definition of LM1

Preliminary studies were performed to compare different results obtained from the various load models of the existing European standards. They all demonstrated qualities and failings, and it was therefore decided to build an original main load model such that :

- its effects reproduce accurately the total utmost effects (local and global) due to the actions of the real traffic for various shapes and dimensions of influence areas representative of the bridge construction in Europe, including the dynamic magnification ;
- its effects could not vary too much if the system only partially applies on the relevant influence surfaces, so that the unfavourable location (loading arrangement) can be easily determined both transversally and longitudinally ;
- its application rules should be as simple and unambiguous as possible.

From a global point of view, the procedure adopted for the development of LM 1 consisted of the two following major steps :
1. Definition and assessment of « target values » for various effects ;
2. Reseach of the best fitted model, able to reproduce the « target values » with accuracy, by the use of operational research.

The definition and the assessment of target values needed several choices :
- Traffic samples and pavement roughness,
- Traffic situations,
- Set of influence areas,
- A level of probability for the assessment of characteristic values,
- Extrapolation methods.

2.1 Trafic samples

The first idea was to mix all the traffic records in order to get an "European sample", but some extrapolation methods, based on mathematical simulations of traffic, needed samples of homogeneous traffic. Considering that the traffic recorded on the A6 motorway near the French city Auxerre was already an "European" traffic, it was decided that all the statistical manipulations would be done only with this traffic and that other traffics might be taken into account to bring, possibly, some corrections.

The Auxerre traffic is rather heavy for one loaded lane, but it is not the heaviest observed traffic : for example, the traffic on the slow lane of the Brohltal bridge in Germany was the most "aggressive", and the recorded daily maximum axle weight was equal to 210 kN on the Ring of Paris while it was equal to 195 kN on the slow lane of A6 motorway (these values are in conformity with the technical capacities of industrial tyres). Taking account of the method of measurement (by piezo-electric cables and a weigh-in-motion system) it was agreed to consider that the real traffic records included an inherent dynamic effect characterised by a magnification factor of 1,10.

Many numerical simulations considering the dynamic behaviour of the vehicles and of the bridges were performed, which were based on some hypotheses about the roughness quality of the carriageway. The corresponding studies are detailed in [6]. Target values of the extrapolated traffic effects were determined for a set of influence areas (see 2.3) on the basis of numerous dynamic calculations.

2.2 Traffic situations

Traffic records give information only on usual traffic conditions. But it is clear that the most critical situations appear with disturbed traffic conditions. Therefore, it has been necessary to define and to combine realistic traffic scenarios (arrangements of vehicles, traffic types) such as free flowing traffic, condensed traffic, traffic jam, special situations due to social demonstrations ("snail" operations), etc. The assessment of the target values, previously mentioned, to be used for the calibration of LM1 needed the selection of traffic situations (or scenarios) on the various lanes of a bridge, i.e. combinations of basic traffics depending on the location and the number of lanes.

The studies for the definition of traffic situations were rather complex and it is only possible to give a general overview in the present contribution. The considered basic traffics were :

- flowing traffic, as recorded on the slow and fast lanes of A6 Motorway, or simulated (random distribution of lorries and cars) on the basis of traffic records and possible manipulations on the number of lorries ;
- congested traffic, that is simulated flowing traffic, moving at very low speed (5 to 10 kph);
- jam situation taking into account vehicles with a conventional distance of 5 m between them.

Several contributors proposed various traffic situations (hazard scenarios), combining flowing and congested traffic and corresponding to deterministic[1] processes. They are summarized hereafter.

2.2.1 Hazard scenarios with flowing traffic

On the *first* (most heavily loaded) *lane*, all contributors proposed to consider the extrapolated traffic corresponding to A6 motorway slow lane, as recorded or simulated with a percentage of lorries of 25%, and speeds about 80-100 km/h.
On the *second lane*, the propositions were more varied : same traffic as on the first lane, traffic on A6 slow lane but corresponding to the daily maximum (no extrapolation), or traffic on A6 fast lanes with 10% of lorries.
On the *third and fourth lanes*, it was mainly proposed to take into account the recorded traffic on A6 fast lanes corresponding to the daily mean loads, or with limited percentages of lorries.

2.2.2 Hazard scenarios with congested traffic or jam situations

On the *first lane*, all propositions were based on A6 slow lane traffic without cars, corresponding to a congested traffic or a jam situation as previously defined.
On the *second lane*, it was envisaged A6 flowing traffic as recorded on the slow lane (extrapolated to 1000 years or daily maximum) or simulated jam situations both on first and second lanes with a distance of 5 m between lorries.
On the *third and fourth lanes*, it was generally proposed to take the daily maximum and the daily mean of A6 slow lane, respectively.

Considering that all points of view were pertinent, the target values defined by each contributor of the Project Team were used to some extent for the calibration of the load model.

2.3 Set of influence lines

Numerous influence areas (for beams as for slabs) were used for the calibration work of the main loading system. The main (cylindrical) influence areas used to assess the effects of the loading pattern are represented in Table 3.

[1] This method was the only one that could be used considering the time constraint of the Project-Team, and was considered as sufficient at the considered step, being conscious that adaptations should be fitted for bridges with traffic lighter than A6 traffic.

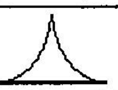
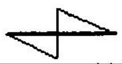
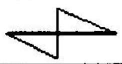
Nr.	Representation	Description of the influence line
I1		Maximum bending moment at mid-span of a simply supported beam.
I2		Maximum bending moment at mid-span of a double fixed beam with an inertia that strongly varies between mid-span and the ends.
I3		Maximum bending moment on support of the former double fixed beam.
I4		Minimum shear force at mid-span of a simply supported beam.
I5		Maximum shear force at mid-span of a simply supported beam.
I6		Total load.
I7		Minimum bending moment at mid-span of the first of the two spans of a continuous beam (the second span only is loaded).
I8		Maximum bending moment at mid-span of the first span of the former continuous beam.
I9		Bending moment on central support of the former continuous beam.

Table 3 : Desciption of the influence lines

For all these influence lines, 9 span lengths were considered : 5, 10, 20, 30, 50, 75, 100, 150 and 200 m. In fact, influence lines I1 and I9 turned out to be the most important lines for the calibration of the load model.

2.4 Level of probability

The target values of the traffic effects were determined such that the probability of exceeding in 50 years was less than 5% (or, in other words, such that they correspond to a mean return period of $\frac{50}{0{,}05} = 1000$ years). This choice was based on the following considerations :

- The chosen probability had to be small enough so that combinations with LM1 or with exceptional convoys (LM3), as dominant variable actions, could be based on the same reliability format.
- The probability for several exceedances of irreversible serviceability limit states during the period of reference should also to be strongly limited.
- It is rational to think that the loads will increase in future and the difference between return period values of 1000 years and of 200 years is small because the distribution of the traffic utmost effects is weakly scattered.

Note that there is no discrepancy in the reliability level between the design effects of LM1 and of LM3 : in other words, the effects of the lightest exceptional convoys are covered by those of LM1. Note also that the approach adopted for road traffic loads started from assessments of load effects and not, as for climatic loads, from a natural parameter representing partially the action.

3. Calibration of LM1

3.1 Principles

The aim of the works was to build a model that would include the dynamic magnification with simultaneous concentrated and distributed loads, so that it covered all traffic scenarios and that both general and local verifications could be simultaneously performed. The minimum intensity of the distributed load was set to 2,5 kN/m^2 on the basis of existing national standards. The calibration tests later confirmed this value.

These calibration tests were carried out at the SETRA with methods of the operational research. Noting :

- S_{1i}, the target values of the selected effects for the various span lengths and the various influence lines or areas[2] ;
- S_{2i}, the corresponding values deriving from the load model under calibration ;
- d_i the gap between S_{1i} and S_{2i} defined by : $d_i = \left|\frac{S_{1i}}{S_{2i}} - 1\right|$;

the following functions were considred :

$$d_{max} = \text{Max}\left|\frac{S_{1i}}{S_{2i}} - 1\right| \qquad d_m = \frac{(\Sigma d_i)}{n}$$

The optimisation method consisted of finding, for various models depending on various parameters, a function S_2 calibrated on the basis of the following criteria, considered separately :

- d_m is minimum,
- d_{max} is minimum as well,
- d_{max} and d_m are minimum and $\frac{S_{1i}}{S_{2i}} \geq 1$ or 0,95

3.2 Calibration procedure and major results

The calibration of LM1 was performed step by step, by the successive consideration of a single loaded lane, two loaded lanes, and finally four loaded lanes. From a general point of view, it appeared that, in the longitudinal direction :

- The best fitted model was composed of both concentrated and uniformly distributed loads;
- It was possible, for the assessment of general effects, to have only a single concentrated load in each lane, but its magnitude made impossible the definition of realistic rules for local verifications.
- The introduction of more than 2 concentrated loads was superfluous because it did not really increase the accuracy of the results.
- The intensity of the uniformly distibuted load should be a decreasing function of the loaded length, noted L.

Table 4 shows LM1 as it resulted from the first calibrations.

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[2] Index i = 1 to n is an identification index of the values obtained for the 9 span lengths, the 8 influence lines and the 3 loading systems (1 lane, 2 lanes, 4 lanes) ; thus n = 9 x 8 x 3 = 216 values.

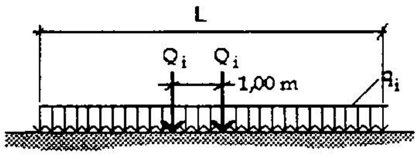
	Loaded lane(s)	Q_i (kN)	q_i (kN/m)
	1	$Q_1 = 185$	$q_1 = 29{,}3 + \dfrac{375{,}6}{L}$
	2	$Q_2 = 100$ kN	$q_2 = 0{,}487\ q_1$
	3+4	$Q_3 + Q_4 = 150$ kN	$q_3 + q_4 = 0{,}56\ q_1$

Table 4

This solution was progressively modified for the purpose of simpler application. The accuracy of the calibration was slightly decreased in order to obtain a load model of very simple use and unambiguous application. Consequently, it was agreed to give a constant magnitude to the uniformly distributed load.

3.3 The final Load Model 1

Further studies about the influence lines and areas with a length smaller than 5 m led to increase the intensity of the concentrated loads on the first and second lanes, to correlatively decrease the intensity of the distributed load on the same lanes and to remove the concentrated loads after the third lane. Besides, the distance between concentrated loads in lanes 1 to 3 was increased up to 1,20 m. This value seemed to fit better the real spacing between two axles of lorries (the aim being to define realistic local verification rules), although the concentrated loads were not initially supposed to represent the axles of actual vehicles. The final model is represented in figure 1.

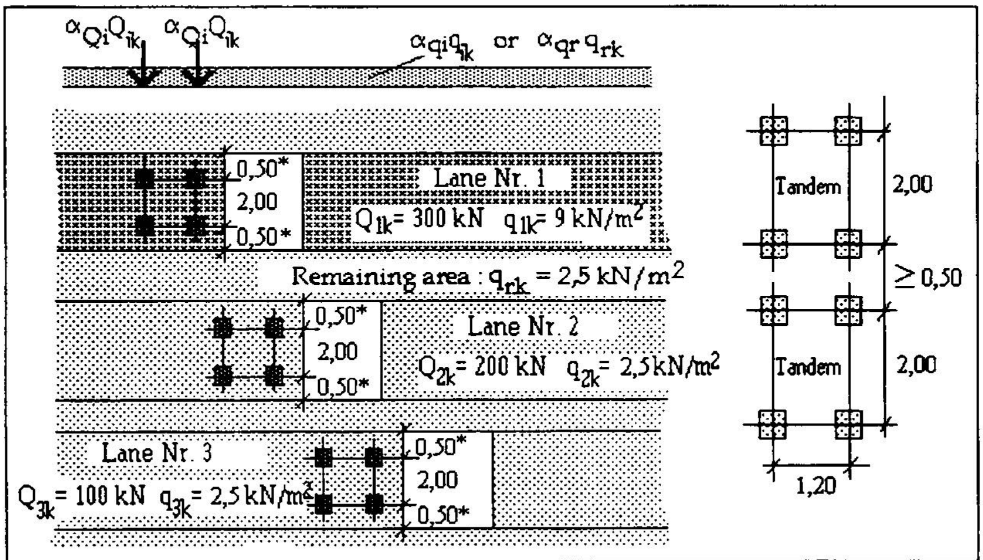


Fig. 1 : Description of LM1

As formerly explained, the calibration of LM1 was performed with the effects of a rather "heavy" traffic (Auxerre traffic). For bridges intended to carry a lighter traffic, special loading classes may be defined in the various NADs by using adjustment factors on concentrated loads (factors α_{Qi}) and on distributed loads (factors α_{qi} or α_{qr}).

3.4 Choice of the wheel contact area shape

On the basis of a detailed study performed by Mr. Prat [5] on local loads transmitted to the carriageway by heavy vehicle wheels, the contact area of the concentrated loads is a square of 400 x 400 mm (fig. 2), corresponding to realistic wheel contact areas of wide tyres.

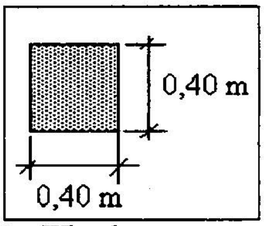


Fig.2 : Wheel contact area

Two main tyre achitectures are met : diagonal carcass, for aircraft and farm tractors, and radial carcass, for almost all road vehicles. In the case of lorries, the use of simple wheels is widely developed : radial tyres deform by crushing only in the longitudinal direction. Therefore, the footprint of such a tyre is a rectangle of constant width and all studies showed that the contact area of tyres with the road pavement is rather square (in all cases, the transverse dimension is less than, or equal to, the longitudinal dimension). With the adopted dimensions, the contact pressures on lane 1 are equal to 150/0,16 = 937,5 kPa (for adjustment factors equal to 1), which corresponds to the dynamic pressure of a tyre on the road pavement (equal to the inflation pressure plus the structural reaction of the tyre).

3.5 Additional considerations

3.5.1 Definition of the loadable width of a bridge deck

For the sake of simplicity, it was agreed that the loadable width of a bridge deck was equal to the net width of the carriageway, measured between the kerbs where they are higher than 100 mm or between the inner limits of relevant road restraint systems.

3.5.2 Definition of the lanes

Considering that, in the most critical traffic situations, vehicles can be driven close to each other, and considering also transient situations (e.g. for maintenance and/or repair), it was agreed that the loads resulting from the calibration tests should apply on strips of 3 m width. So are defined the so-called "*notional lanes*" of 3 m width, that are independent of the marker strips on the road surface. These notional lanes can be located anywhere on the drivable surface : their ***maximum*** number is thus the integer part of the division by 3 of the carriageway width (where this width is more than 6 m).

3.5.3 Definition of Load Model 2

Some calculations showed that the tandem systems of LM1 did not cover all local effects of vehicles of various configurations. Therefore, for some local verifications (in particular in case of orthotropic slabs), it was agreed to complete this model with a

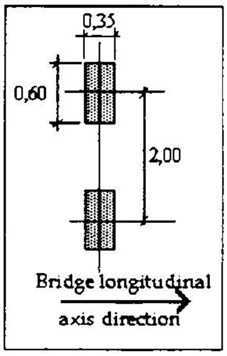


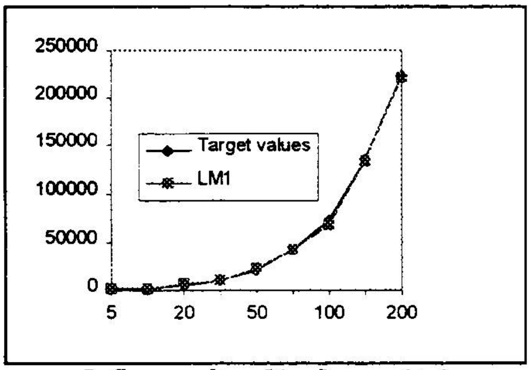
Fig. 3 : LM2

loading system (LM2) which also allows to take into account other contact areas than the ones corresponding to wide tyres (in case of double wheels) and to correct the effects of LM1 for very short influence lines. It consists of a single axle corresponding to a load of 400 kN to which can be applied an adjustment factor β_Q depending on the class of the expected traffic for a particular project (fig. 3).

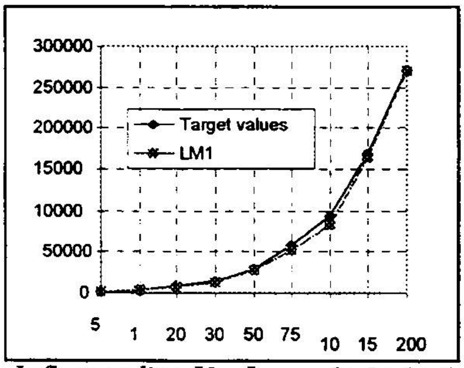
4. Comparisons between the effects of LM1 and the corresponding target values

Some typical curves are given hereafter to show the quality of the adjustment between the target values and the effects of LM1.

4.1 Influence line I1

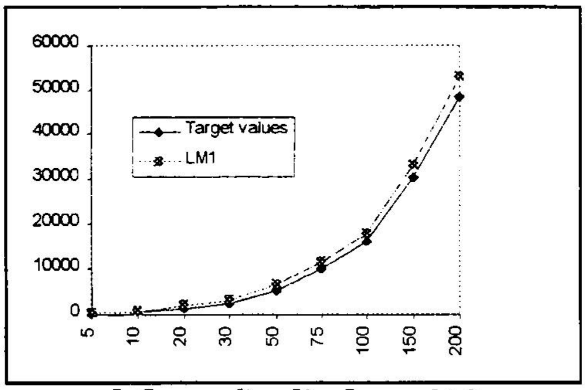


Influence line I1 - Lanes 1+2

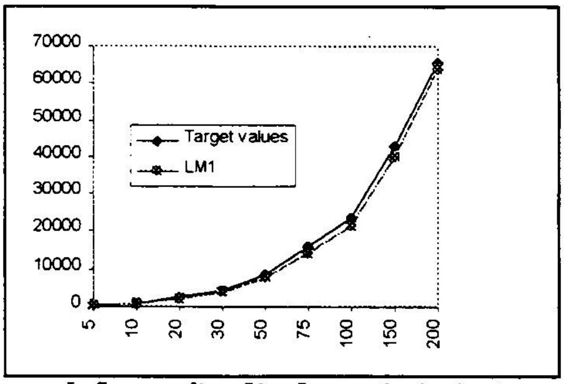


Influence line I1 - Lanes 1+2+3+4

4.2 Influence line I2

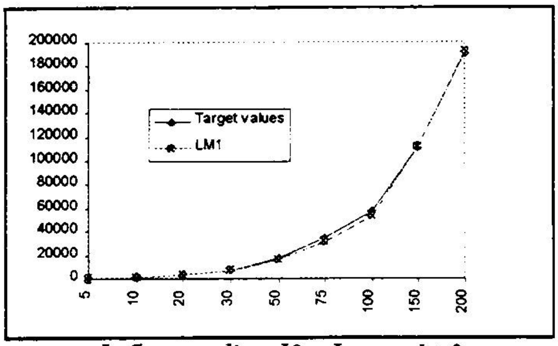


Influence line I2 - Lanes 1+2

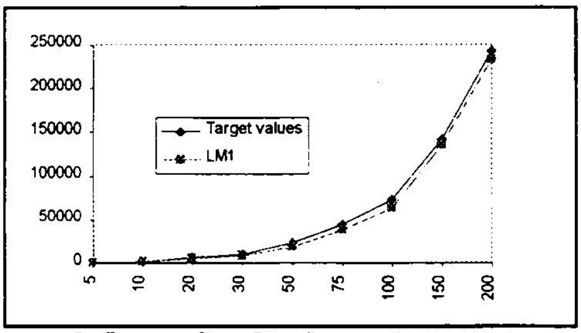


Influence line I2 - Lanes 1+2+3+4

4.3 Influence line I9



Influence line I9 - Lanes 1+2



Influence line I9 - Lanes 1+2+3+4

The major differences are obtained with influence line I2, which is a very particular influence line because the ratio between the moment of inertia of end cross-sections and cross-section at mid-span is very high. The values obtained from the model are bigger than the calibration values for one and two loaded lanes, but not for four loaded lanes. For loaded lanes 1+2, the deviation varies from 27 % for L = 50 m to 9 % for L = 200 m.

5. Representative values of traffic loads and groups of loads

5.1 Representative values of traffic loads

The various representative values of traffic loads are :
- the ***characteristic values*** (which were calibrated as previously described) for the ultimate limit states ;
- the ***infrequent values***, corresponding to a mean return period of one year ;
- the ***frequent values***, corresponding to a mean return period of one week.

The ***quasi-permanent values*** of the actions due to the road traffic are in general practically zero. In case of bridges that support a heavy and continuous traffic, a non null quasi-permanent value of the uniformly distributed load of the main loading system might be considered, with a likely uniform or unique distribution in the transverse direction.

Infrequent values had been required by drafters of EC2.2. They were intended for verifications concerning some serviceability limit states that correspond to an imperfect reversibility of the effects, and were assessed on the basis of a mean return period of 1 year. The infrequent models derive from the characteristic models by means of a reduction factor ψ'_1 equal to 0,8. This means consequently that the traffic effects corresponding to a return period of 1000 years are only 20% higher than those corresponding to a return period of 1 year. For more details concerning the calibration of frequent and infrequent values, see [6].

5.2 Definition of groups of loads

On a bridge deck, various kinds of loads, represented by specific models, may be more or less simultaneously applied : road traffic loads, pedestrian loads, abnormal vehicles, crowd, etc. These loads are multi-component and give rise to vertical and horizontal forces.

In Part 3 of ENV 1991, various models are defined to represent all kinds of loads : vertical forces due to vehicles or pedestrians, braking and acceleration forces. In order to facilitate the work of designers, and in accordance with ENV 1991-1 « Basis of design », groups of loads are defined, each of them being considered as one variable action in combinations. The major group of loads is group Nr. 1, which includes the vertical forces due to LM1 and vertical forces due to a load on footways and cycle tracks with a reduced value of 2,5 kN/m^2. For this group :

- the infrequent values are obtained as previously explained by applying to the characteristic values of LM1 and LM2 a ψ'_1 factor equal to 0,80 ;
- the frequent values are obtained by applying a ψ_1 factor equal to 0,75 to the concentrated loads of LM1 and LM2, and equal to 0,4 to the uniformly distributed load.

In the ultimate limit state combinations, the partial safety factor for road traffic actions is equal to 1,35 (as for permanent actions).

6. Possible evolution of traffic load models

Most of the background studies was scientifically performed. Based on real traffic records, the calibration of the main loading system resorted to probabilistic techniques and to the methods of operational research. Only one step was deterministic : the choice of traffic scenarios on the various lanes of a bridge deck.

In the future, if more refined calibrations are undertaken especially for the EN stage, it would be probably more satisfactory to reconsider the problem of traffic scenarios on the basis of a probabilistic approach, as far as statistical bases can be found for this.

REFERENCES

[1] Final Report to the Commission of the European Communities on contract Nr. PRS/89/7750/MI 15, concerning development of Models of Traffic Loading and Rules for the Specification of Bridge Loads. Prepared by H. Mathieu, J.A. Calgaro, M. Prat - October 1989.

This report includes :

- Final Report of Subgroup 1 (10 December 1988) : « Definition of a set of reference bridges and influence areas and lines » - MMrs Calgaro, Eggermont, König, Malakatas, Prat, Sedlacek.
- Final Report of Subgroup 2 (March 1898) : « Traffic data of the European countries » - MMrs. Jacob, Bruls, Sedlacek.
- Final Report of Subgroup 3 (8 May 1989) : « Definition and treatment of abnormal loads » - MMrs. De Buck, Demey, Eggermont, Hayter, Kanellaidis, Mehue, Merzenich.
- Report of Subgroup 6 (April 1989) : « Draft clauses for secondary components of the action of traffic » - MMrs. Gilland, Vaaben, Pfohl, O'Connor, Mehue.

[2] Final Report to the Commission of the European Communities on contract Nr. PRS/90/7750/RN/46 Concerning develomment of Models of Traffic Loading and Rules for the Specification of Bridge Loads. Prepared by H. Mathieu, J.A. Calgaro, M. Prat.

This report includes :

- Final Report of Subgroup 5 (9 October 1991) : « Definition of dynamic impact factors » - MMrs. Astudillo, Bruls, Cantieni, Drosner, Eymard, Flint, Hoffmeister, Jacob, Merzenich, Nicotera, Petrangeli, Sedlacek.
- Final Report of Subgroup 6 (November 1990) : « Secondary components of the action of traffic » - MMrs. Gilland, Vaaben, Pfohl, O'Connor, Mehue.
- Final Report of Subgroup 7 (October 1991) : « Fatigue rules » - MMrs. Bruls, Flint, Jacob, König, Sanpaolesi, Sedlacek.
- Final Report of Subgroup 8 (August 1991) : « Methods for the prediction of vehicle loads and load effects on bridges » - MMrs. Jacob, Bruls, Flint, Maillard, Merzenich.
- Final Report of Subgroup 9 : « Reliability aspects » - MMrs. Flint, Sedlacek, Jacob, Bruls, Sanpaolesi.
- Report on local loads (27 November 1989) - M. Prat

[3] Prat M. (1990) - Comparaison de différents règlements de charges. Annales ITBTP n° 482 - Paris.

[4] Mathieu H., Calgaro J.A., Prat M. (1993) - Etat d'avancement des travaux de l'Eurocode 1 (volume 3 : charges de trafic sur les ponts routiers). Annales ITBTP n° 514 - Paris.

[5] Prat M., Jacob B. (1992) - Local load effects on road bridges - Third International Symposium on heavy vehicle weights and dimensions - Cambridge.

[6] Bruls A., Croce P., Sanpaolesi L., Sedlacek G. (1996) - « ENV 1991 - Part 3 : Traffic loads on bridges - Calibration of load models for road bridges » - IABSE Colloquium - Basis of Design and Actions on Structures, 27-29 March 1996.

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Railway traffic actions and combinations with other variable actions

Einwirkungen aus Eisenbahnverkehr und ihre Kombinationen mit Einwirkungen, die nicht bahnspezifisch sind

Actions de circulation ferroviaire et combinaisons avec les actions autres que de circulation ferroviaire

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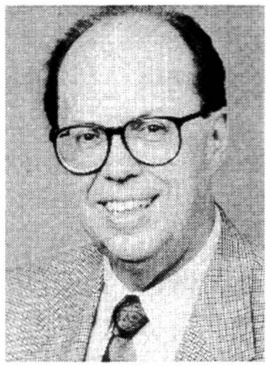

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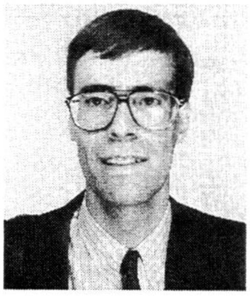

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SOMMAIRE

Cet article présente la partie 6 "actions ferroviaires et autres actions spécifiques aux ponts-rail" de l'ENV 1.3. Il décrit les charges élémentaires et groupes de charges, leurs combinaisons, ainsi que certains "backgrounds" liés aux états-limites de service ferroviaires.

ZUSAMMENFASSUNG

Der vorliegende Artikel stellt den Teil 6 "Einwirkungen aus Eisenbahnverkehr und andere spezifische Einwirkungen für Eisenbahnbrücken" der ENV 1.3 vor. Er beschreibt die elementaren Lasten und die Lastgruppen, ihre Kombinationen, sowie gewisse "Backgrounds" zu den Gebrauchsgrenzzuständen.

SUMMARY

This article presents part 6 "Railway actions and other actions specific to railway bridges" contained in ENV 1.3. It describes elementary loads, groups of loads and their combinations, together with the background to the serviceability limit states.

1. Introduction

This article contains a presentation of the current thoughts of European railway companies about the contents of ENV 1991.3 standard "Traffic loads on bridges" - Part 6 which is called "rail traffic actions and other actions specifically for railway bridges" (EC 1.3, Part 6). This provisional draft for the Eurocode has been made by 6 railway experts from a number of European railway undertakings, who incidentally are members of the Bridge Sub-committee of the International Union of Railways (UIC).

ENV 1991-3 (EC1-3 part 6) needs to be completed and tested before it is put to the vote as an EN. Groups of loads and combinations of actions, among other things, are still being discussed by European railway administrations. Therefore, the following should be considered as initial thoughts which is slightly different from the ENV prescriptions and application rules, taking into account the early observations raised by BANVERKET (Sweden), DB-AG (Germany) and S.N.C.F. (France).

This part of the Eurocode is essential for the railway administrations which will be involved in the future European High-Speed Rail System.

As a matter of fact, in conjunction with the Eurocode development, Engineers have been given a two-year timescale to produce a common technical response to the E.U. directive on interoperability of the European High-Speed Rail System. The purpose of this directive is to enable the operation of any type of train, whether existing or yet to be developed, for speeds of 250 km/h and above, on the totality of the network.

To achieve this goal, an organization was set up consisting of representatives from UIC (railway companies) and UNIFE (railway manufacturers) and is supported by editorial groups in charge of technical specifications on interoperability (STI). This specification will be mandatory for designers and suppliers of high speed sub-systems (infrastructure, rolling stock, power supply systems, command/control systems). The organisation is further supported by a coordination group addressing interfaces between these various sub-systems. So far as bridges are concerned, STI will refer to the Eurocode.

The interoperability parameters applicable to bridges are the following :

* for railway actions : vertical loads, horizontal static loads (in particular, braking and traction forces, slip-stream effects, load combinations) (see § 6.3 to 6.7).
* for traffic safety criteria : permissible girder vertical accelerations, twists, rotations and horizontal deformations (see § 3.1.2 in appendix G).

It should be noted that the operating comfort and durability of structures are not assumed as essential interoperability requirements for bridges.

The parameters relating to rolling stock which have an impact on bridge interoperability are as follows : axle load, axle spacing, operating speed, vertical suspension characteristics (or transfer function) braking and traction forces, train aerodynamic drag coefficient. The range of characteristics required for future high-speed trains will have to be validated by the coordination and interface group.

2. Vertical load models

To design railway bridges, various vertical load models must be taken into account, as follows :

Vertical load models	UIC 71 + SW/0	SW/2	Unloaded train	Actual trains for dynamic calculation	Train types for fatigue
Approach	Deterministic (characteristic = nominal values).	Deterministic (characteristic = nominal values).	Deterministic (characteristic = nominal values).	Actual trains, especially present high speed trains all over the world.	Collection of trains representative of traffic.
Description	Normal traffic : see load model below.	Heavy load traffic : see load model below.	a 12.5kN/m uniformly distributed force.	For instance, AVE, ETR, EUROSTAR, ICE,TGV SHINKANSEN.	* 12 train types, * standard traffic mix, * heavy traffic mix.
Static assessment	Yes, multiplied by a factor α if specified.	If specified.	Yes.	No.	No
Dynamic assessment	Dynamic effect is taken into account by a multiplying factor Φ, within a field of application.	Dynamic effect is taken into account by a multiplying factor Φ, within a field of application.	No.	To be used for dynamic calculation, outside of the field of application of the dynamic factor Φ.	No.
Fatigue assessment	Normal traffic mix is taken into account by UIC 71 (including the dynamic factor Φ) multiplied by a factor λ.	No.	No.	No.	If specified : standard traffic mix, or, heavy traffic mix with 250kN axles, or, special traffic mix as a combination of train types.

Table 1

(1) European Rail Research Institute (ERRI), Union Internationale des Chemins de fer (UIC).

Fig. 1: UIC71 LOAD MODEL :

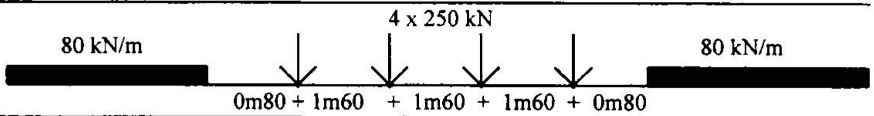


Fig. 2: SW/0 AND SW/2 LOAD MODELS :

Load model	q_{vk} (kN/m)	a (m)	c (m)
SW/0	133	15,0	5,3
SW/2	150	25,0	7,0

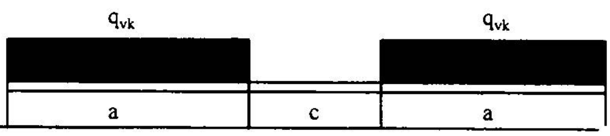


Fig. 3: TGV AXLE LOADS (as an example) :

V = 350 km/h
L = 237,60 m
q = 21,5 kN/m
ΣQ = 5100 kN

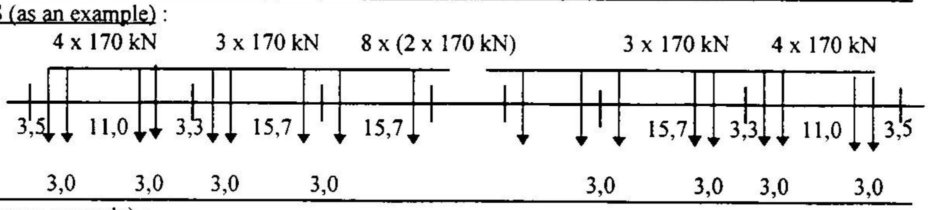
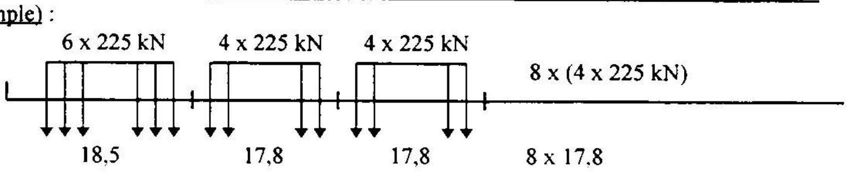


Fig. 4: TRAIN TYPE N°7 (as an example) :

ΣQ = 10350 kN
V = 120 km/h
L = 196,50
q = 52,7 kN/m



3. Groups of loads, combinations of actions and specific railway assessments

3.1 Horizontal forces

Together with vertical loads, some horizontal forces due to rail traffic must be taken into account :

- **traction and braking** forces (see ENV 1991-3, §6.5.3 and §6.5.4) act at the top of the rails in the longitudinal direction of the track; when the track is continuous at one or both ends of the bridge, only a proportion of these forces is transferred through the deck to the bearings, the remainder of the forces are transmitted through the track where it is resisted behind the abutments. This is called « interaction between the track and the bridge due to traction and braking » ;
- **centrifugal** forces (see ENV 1991-3, §6.5.1) are considered fully transmitted through the deck to the bearings ;
- **nosing** forces having generally only local effects.

3.2 Groups of loads and rail traffic action

The simultaneous effects of the various vertical and horizontal forces due to the railway traffic is taken into account by considering the groups of loads, as follows (boxed values) :

Groups of loads			Vertical forces			Horizontal forces		
Nb of loaded tracks	**Group n°**	**Loaded track n°**	**LM71= UIC71+ SW/0**	**SW/2**	**Unloaded train**	**Traction and braking**	**Centrifugal force**	**Nosing force**
One track	**11**	**T1**	[1,0]	[0]	[0]	[1,0]	[0,5]	[0]
	12	**T1**	[1,0]	[0]	[0]	[0,5]	[1,0]	[0]
	13	**T1**	[1,0]	[0]	[0]	[1,0]	[0,5]	[1,0]
	14	**T1**	[0]	[1,0]	[0]	[1,0]	[0,5]	[0]
	15	**T1**	[0]	[0]	[1,0]	[0]	[1,0]	[0]
Two tracks	**21**	**T1**	[1,0]	[0]	[0]	[1,0]	[0,5]	[0]
		T2	[1,0]	[0]	[0]	[1,0]	[0,5]	[0]
	22	**T1**	[1,0]	[0]	[0]	[0,5]	[1,0]	[0]
		T2	[1,0]	[0]	[0]	[0,5]	[1,0]	[0]
	23	**T1**	[1,0]	[0]	[0]	[1,0]	[0,5]	[1,0]
		T2	[1,0]	[0]	[0]	[1,0]	[0,5]	[1,0]
	24	**T1**	[1,0]	[0]	[0]	[0,75]	[0,5]	[0]
		T2	[0]	[1,0]	[0]	[0,75]	[0,5]	[0]
Three tracks or more	**31**	**T1**	[0,75]	[0]	[0]	[0,5]	[0,75]	[0]
		T2	[0,75]	[0]	[0]	[0,5]	[0,75]	[0]
		T3	[0,75]	[0]	[0]	[0]	[0,75]	[0]
	32	**T1**	[0,75]	[0]	[0]	[1,0]	[0,375]	[0]
		T2	[0,75]	[0]	[0]	[1,0]	[0,375]	[0]
		T3	[0,75]	[0]	[0]	[0]	[0,375]	[0]
	33	**T1**	[0,75]	[0]	[0]	[1,0]	[0,375]	[1,0]
		T2	[0,75]	[0]	[0]	[1,0]	[0,375]	[1,0]
		T3	[0,75]	[0]	[0]	[0]	[0,375]	[1,0]

table 2

to be considered in designing a structure supporting one track.
to be considered in designing a structure supporting two tracks ; that means all the groups from 11 to 24.
to be considered in designing a structure supporting three tracks or more ; that means all the groups from 11to33.

The multicomponent action due to railway traffic from the groups of loads above should be chosen in order to determine the most unfavourable effect for each assessment. Embankment loading can be added, when relevant.

3.3 Other variable actions

Some other actions must be taken into account. For instance :

- aerodynamic effects (slipstream due to railway traffic, see ENV 1991 -3, §6.6), should be considered as a separate variable action,
- non public footpaths loads (see ENV 1991-3, §6.3.6.1)),
- wind forces (see ENV 1991 -2-4),
- temperature effects (see ENV 1991-2-5), including interaction between track and deck of bridges (see ENV 1991 -3, §6.5.4).

3.4 Representative values of the rail traffic action

Each traffic action, as defined in ENV 1991-3, part 6, must be considered as a characteristic value, for combination with non-traffic actions.
The other representative values are defined by multiplying by factors Ψ_1' (infrequent values), Ψ_1 (frequent values) and Ψ_2 (quasi-permanent) : see table 3 below (boxed values).

3.5 Combinations of actions

In order to use « Basis of Design » format, see combination factors Ψ_0 and partial safety factors γ_Q in table 3 below (boxed values).

Variable actions		γ_Q	ψ_0	ψ'_1	ψ_1	ψ_2
Main traffic action (Groups of loads)	Gr. n° 11	[1,45]	[0,80]	[1,00]	[0,80]	[0]
	Gr. n° 12	[1,45]	[0,80]	[1,00]	[0,80]	[0]
	Gr. n° 13	[1,45]	[0,80]	[1,00]	[0,80]	[0]
	Gr. n° 14	[1,20]	[0,80]	[1,00]	[0,80]	[0]
	Gr. n° 15	[1,00] (3)	[0,80]	[1,00]	[0,80]	[0]
	Gr. n° 21	[1,45]	[0,80]	[1,00]	[0,60]	[0]
	Gr. n° 22	[1,45]	[0,80]	[1,00]	[0,60]	[0]
	Gr. n° 23	[1,45]	[0,80]	[1,00]	[0,60]	[0]
	Gr. n° 24	[1,20]	[0,80]	[1,00]	[0,60]	[0]
	Gr. n° 31	[1,45]	[0,80]	[1,00]	[0,40]	[0]
	Gr. n° 32	[1,45]	[0,80]	[1,00]	[0,40]	[0]
	Gr. n° 33	[1,45]	[0,80]	[1,00]	[0,40]	[0]
Other traffic actions	aerodynamic effects	[1,50]	[0,80]	[1,00]	[0,50]	[0]
	non public footpaths	[1,50]	[0,80]	[0,80]	[0,50]	[0]
Wind forces	F_{wk} or F_{wn} (1)	[1,50]	[0,60]	[0,60]	[0,50]	[0]
	F_w^{**} (1)	[1,50]	[1,00]	[0]	[0]	[0]
Temperature effects	T_k (2)	[1,50]	[0,60]	[0,80]	[0,60]	[0,50]

Table 3

(1) Whenever wind action is required to be considered with traffic, the wind action $\psi_0 F_{wk}$ or $\psi_0 F_{wn}$ should be taken as no greater F_w^{**} : see ENV 1991-2.4.

(2) see ENV 1991-2.5.

(3) generally [1,00], to be combined with wind forces, for transversal static equilibrium or lateral internal forces.

3.6 Design situations and combinations of actions

Design situations and combinations of actions are summarized in the following next two pages table 4 (boxed values).

Table 4 (1/2)

SITUATIONS			PERSISTENT AND TRANSIENT											
LIMIT STATES			Ultimate (3) static equilibrium		resistance		Service infrequent		frequent		quasi permanent		fatigue	
COMBINATIONS			fundamental		fundamental		infrequent		frequent		quasi permanent		fatigue	
PERMANENT ACTIONS			G1 G2		G. Max. G. Min.		G. Max. G. Min.		G. Max. G. Min.		G. Max. G. Min.		G. Max. G. Min.	
direct actions	Self weight Earth pressure and weight	Fav. Unfav.	0.9 (2) 1.1 (2)		1 1.35		1 1		1 1		1 1		1 1	
	Movable loads	Fav. Unfav.	0.9 (2) 1.1x1.3 (2)		1 1.35x1.3		1 1.3		1 1.3		1 1.3		1 1.3	
indirect actions	Settlements	Fav. Unfav.	0 1.1 (2)		0 1.35		0 1		0 1		0 1		0 1	
	Prestressing, shrinkage and creep	Fav. Unfav.	/ /		1 1.35		1 1		1 1		1 1		1 1	
Variable actions			d.	a.	d.	a.	d.	a.	d.	a.	d.	a.	d.	a.
Traffic actions	Gr11		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	0.8	/	/	/	/	/
	Gr12		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	0.8	/	/	/	/	/
	Gr13		1.45	1.45x1	1.45	1.45x1	1	1	0.8	/	/	/	/	/
	Gr14		1.2	/	1.2	/	1	/	0.8	/	/	/	/	/
	Gr15		1	1x1	1	1x1	1	1	/	/	/	/	/	/
	Gr21		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	0.6	/	/	/	/	/
	Gr22		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	0.6	/	/	/	/	/
	Gr23		1.45	1.45x1.0	1.45	1.45x1.0	1	1	0.6	/	/	/	/	/
	Gr24		1.35	/	1.35	/	1	/	0.6	/	/	/	/	/
	Gr31		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	0.4	/	/	/	/	/
	Gr32		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	0.4	/	/	/	/	/
	Gr33		1.45	1.45x1	1.45	1.45x1	1	1	0.4	/	/	/	/	/
	Embankment loads		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	(1)	/	/	/	/	/
	Other traffic actions (actual trains, specific actions)		1.45	1.45x0.8	1.45	1.45x0.8	1	0.8	0.8	/	/	/	/	/
	Fatigue traffic actions		/	/	/	/	/	/	/	/	/	/	1	/
	Other traffic actions		1.5	1.5x0.8	1.5	1.5x0.8	1	0.8	0.5	/	/	/	/	/
Other variable actions	Natural actions Wind		1.5	1.5x0.6	1.5	1.5x0.6	1	0.6	0.5	/	/	/	/	/
	Thermal		1.5	1.5x0.6	1.5	1.5x0.6	1	0.6	0.6	0.5	/	0.5	/	0.6
	Seismic actions		/		/				/		/		/	
	Accidental actions		/		/		/		/		/		/	

(1) 0.8 / 0.6 / 0.4 for 1, 2 or 3 tracks.
(2) 0,85 and 1,15 instead of 0,9 and 1,1 when people safety is involved.
(3) General equilibrium of earthworks is not included in this table.
« d. » = dominant
« a. » = accompanying

Table 4 (2/2)

SITUATIONS			ACCIDENTAL				SEISMIC					
LIMIT STATES			**Ultimate** **static equilibrium**		**resistance**		**Ultimate** **static equilibrium**		**resistance**		**Service**	
COMBINATIONS			accidental		accidental		seismic		seismic		infrequent	
PERMANENT ACTIONS			G1 G2		G. Max. G. Min.		G. Max. G. Min.		G. Max. G. Min.		G. Max. G. Min.	
Direct actions	Self weight Earth pressure and weight	Fav.	(1)		[1]		[1]		[1]		[1]	
		Unfav.	(1)		[1]		[1]		[1]		[1]	
	Movable loads	Fav.	[1]		[1]		[1]		[1]		[1]	
		Unfav.	[1.3]		[1.3]		[1.3]		[1.3]		[1.3]	
Indirect actions	Settlements	Fav.	[0]		[0]		[0]		[0]		[0]	
		Unfav.	(1)		[1]		[1]		[1]		[1]	
	Prestressing, shrinkage and creep	Fav.	/		[1]		/		[1]		[1]	
		Unfav.	/		[1]		/		[1]		[1]	
Variable actions			d.	a.	d.	a.	d.	a.	d.	a.	d.	a.
Traffic actions	Gr11		[0,8]	/	[0,8]	/						
	Gr12		[0,8]	/	[0,8]	/						
	Gr13		[0,8]	/	[0,8]	/						
	Gr14		[0,8]	/	[0,8]	/						
	Gr15		/	/	/	/						
	Gr21		[0,6]	/	[0,6]	/						
	Gr22		[0,6]	/	[0,6]	/						
	Gr23		[0,6]	/	[0,6]	/						
	Gr24		[0,6]	/	[0,6]	/						
	Gr31		[0,4]	/	[0,4]	/						
	Gr32		[0,4]	/	[0,4]	/	**R E**	**S E**	**R**	**V**	**E**	**D**
	Gr33		[0,4]	/	[0,4]	/						
	Embankment loads		(1)	/	(1)	/						
	Other traffic actions (actual trains, specific actions)		[0,8]	/	[0,8]	/						
	Fatigue traffic actions		/	/	/	/						
	Other traffic actions		[0.5]	/	[0,5]	/						
Other variables actions	Natural actions Wind		[0.5]	/	[0,5]	/						
	Temperature		[0,6]	[0,5]	[0,6]	[0,5]						
	Seismic actions		/		/		[1]		[1]		[1]	
	Accidental actions		[1]		[1]		/		/		/	

(1) 0.8 / 0.6 / 0.4 for 1, 2 or 3 tracks.
« d. » = dominant
« a. » = accompanying

3.7 Specific railway assessments

Besides assessments related to structures and materials, there are some specific railway criteria to be checked (see ENV 1991-3, annex G3).

	Criteria	Safety of traffic due to bridge		Durability of bridge		Safety of traffic due to track			Comfort
Situations	Limit states of bridge	Static equilibrium	Resistance	Durability of bridge	Fatigue of bridge	Track geometry	Stress in rail	Ballast compacity	Deflection
Normal traffic (persistent and transient situations)	ULS static equilibrium	X							
	ULS resistance		X						
	SLS infrequent			X		X	X	X	X
	other SLS			X					
	Fatigue LS				X				
Earthquake	ULS static equilibrium	X							
	ULS resistance		X						
	SLS infrequent			X		X	X	X	
Accidental situations (derail- -ments and collisions)	ULS static equilibrium	X							
	ULS resistance		X						
	SLS infrequent			X		X	X	X	

4 Current research topics

Up to now, research which was conducted with a view to setting up common directives specific to the development of a European High Speed Rail System, has highlighted two main problems raised concerning track safety, they are the dynamic behaviour of bridge girders under traffic action, and the problems related to the interaction between continuous track and structure.

4.1 Dynamic behaviour under traffic actions (background)

Safety and comfort are two major requirements determining the deformability limits of rail bridges.

The safety of train operations is conditional upon the strict observance of certain criteria concerning the permanent way. It is first important to make sure that the wheel/rail contact is still maintained despite the oscillations of the structure and the train dynamic trajectory. As a result, the ascending vertical acceleration onto axles and the track twist due to the girder torsional movements have to be restricted. Secondly, it is also necessary to check that the girders' dynamic oscillations do not cause a reduction in track stability or loss of track geometry (in the case of the ballasted track, this can give a lateral stability defect).

To prevent any discomfort when a train is crossing a bridge, passengers should not be subject to excessive levels of vertical acceleration. These accelerations are generated, on the one hand, by bridge oscillations and, on the other, by the damping from vehicle body suspensions.

The deformability criteria that should be assumed for specific checks on the serviceability limit state of the railway bridges are shown in appendix G 3. The limitations on the natural frequency are shown in item 6.4.3. Such limitations should guarantee that the dynamic stresses due to actual trains at speeds smaller than or equal to 220 km/h, remain smaller than the stresses calculated with the UIC loading scheme, including the dynamic factor. In the 70's UIC developed a dynamic increment factor from a statistical survey on existing bridges' stiffnesses. Therefore, new design bridges should not be made more flexible than existing bridges. At very high speeds (in excess of 220 km/h), this check has to be supplemented by dynamic calculations under actual traffic as shown in appendix H, in order to cover any resonance or excessive vibration of the girder.

The calculations and measurements made by the various UIC members on the permanent way, girders and vehicles have led to the determination of the following limits for high speeds : the vertical acceleration of the girder is limited to 0,35 g (wheel/rail contact criteria and ballast loosening), twist to 0,4 mm/m (wheel/rail contact criterion), rotations at girder ends to a level usually comprised between 0,5 10^{-3} and 10^{-3} rd under an actual train (rail breakage criterion -due to excessive tensile strength or to track buckling resulting from excessive compressive load- and ballast looseness criterion), the vertical accelerations on vehicle bodies to a level between 0,1 and 0,2 g (depending on the level of comfort required).

Detailed investigations into the dynamic behaviour of structures should be made by calculations appropriate to the structures to solve the equation of the dynamic movement of bending beams using the finite element method. This equation is the following :

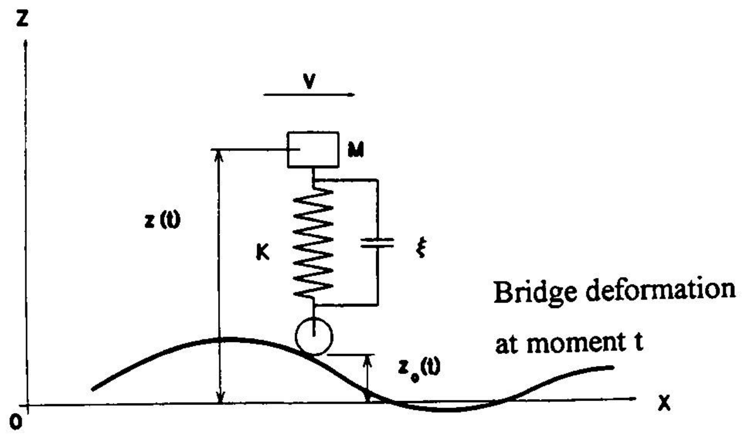
$$m(x)\frac{d^2y(t,x)}{dt^2} + c\frac{dy(t,x)}{dt} + \frac{d^2}{dx^2}EI(x)\frac{d^2y(t,x)}{dx^2} = p(t,x)$$

The finite element method consists of determining successive vibration modes on the structure and then in calculating the structural response by model superimposition, with the selection of train speeds likely to result in resonance situations (so called "critical" speeds) .

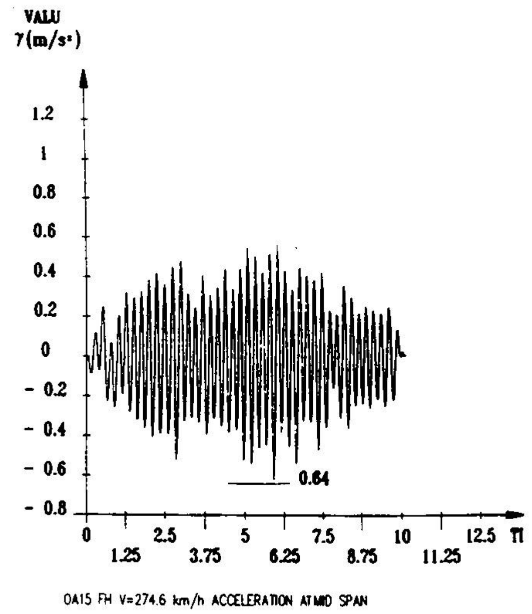
The study on the train dynamic behaviour is carried out on the basis of calculations determining the Za(t) displacement at the level of a bogie and the vertical acceleration onto the body including bogie suspension characteristics which are obtained by the integration of the z(t) differential equation where :

$$\frac{d2z}{dt2} + 2\xi\omega n \left(\frac{dz}{dt} - \frac{dza}{dt}\right) + \omega_n^2 (z(t) - za(t)) = 0$$

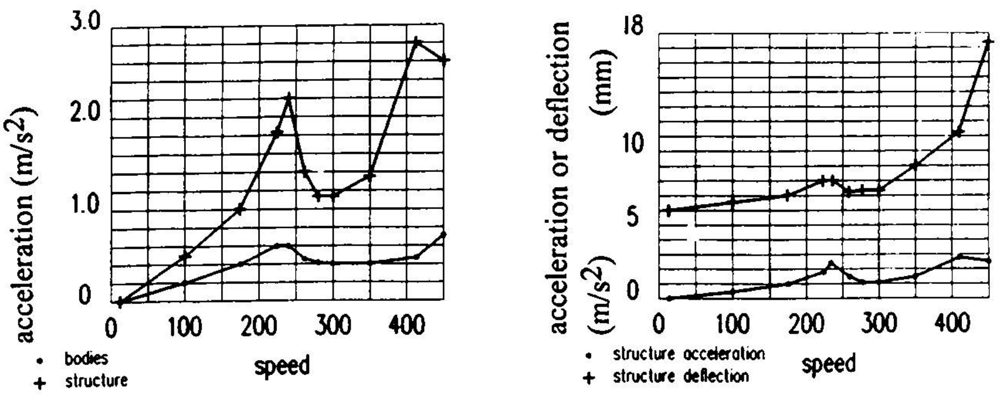
ωn: is vibration of the body/bogie assembly.
and ξ: is the damping/critical damping ratio.



Trajectory of bodies - modelling



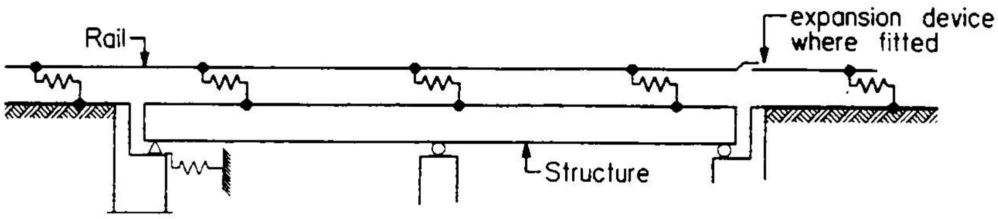
Midspan of continuous bridge acceleration at resonance



Example of complete analysis of vibratory behaviour

4.2 Track structure interaction (background)

When a track is continuous at least at one end of the bridge, the longitudinal forces generated by the track are distributed as a result of the interaction between track and structure. The longitudinal force components transmitted to each element (bridge and track) depend on track resistance to longitudinal displacement in relation to the adjacent structure or substructure, and on the girder resistance to longitudinal displacement, hence on the stiffness of bearings (bearing devices, piers, foundations). The additional forces exerted on the track will have to be withstood by the track ; the force components affecting the bridge will have to be taken into consideration for the design of the structure.



Model of structure for interaction

The loading cases likely to generate additional horizontal forces are essentially : thermal expansion, horizontal traction and braking loads, angular rotation of the structure at the bearings.

For each of these determinant factors, item 6.5.4 gives values for the design of structures under the interaction effects from a ballasted continuous track and takes into account the variations of permissible stress increment factors in long-welded rails. These interaction effects are essentially : the girder maximum expansion lengths, the permissible girder longitudinal displacement under braking and traction forces, permissible bending rotations at the level of bearings, the bearing reactions due to thermal loads, the bearing reactions due to braking and traction.

However, it should be noted that the design assumptions of the ENV relating to the interaction only reflect the case of ballasted structures with either isostatic girder and a fixed bearing at one end, or continuous girder and fixed end or intermediate bearing, and with track equipped with UIC 60 rails, providing for a standard track behaviour law on its bearing and that they are only valid for certain temperature ranges of the rail and of the structure.

The other cases (different track equipment, direct fastened track, sequence of isostatic or continuous girders, etc.) are subject to specific requirements in each railway. A UIC committee of experts has been set up to conduct modellings, tests and measurements, so as to achieve a joint specification by the end of 1997, this deadline being both applicable to EC1 and STI.