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Rubrik: Eurocode 1: Basis of design and actions on structures

EC 1: Basis of Design and Actions on Structures

EC 1: Principes de dimensionnement et actions sur les structures porteuses

EC 1: Grundlagen für Entwurf, Bemessung und Konstruktion und Einwirkungen auf Tragwerke

John B. MENZIES
Partner
Andrews, Kent & Stone
London, UK

John Menzies, born 1937, is a graduate of the University of Birmingham, England. For many years he undertook research and development in structural engineering at the UK Building Research Establishment. He is now a consulting engineer.

Paul LÜCHINGER
Dr. Eng.
Wenaweser+Wolfensberger AG
Zurich, Switzerland

Paul Lüchinger, born 1943 is a graduate of the Swiss Federal Institute of Technology (ETHZ), Zurich. After a long involvement in research and development in structural engineering, he is now a consulting engineer mainly engaged with the design of buildings and bridges.

Haig GULVANESSIAN
Civil Eng.
Build. Res. Establ.
Garston, UK

Haig Gulvanessian, born 1941, is a graduate of the Universities of London and Wales. Before appointed to his present post he was at the Departments of Transport and Environment UK, where he was concerned with the design of bridges elevated roadways, and buildings.

SUMMARY

The Eurocode «Basis of Design and Actions on Structures» is to be a comprehensive code of practice providing information on basis of design and on all actions that it is necessary to consider in the design of structures. This paper briefly describes the background to the preparation of the Eurocode and outlines its proposed framework and contents. The progress achieved in its development is summarised and the plans for completing the work are described.

RESUME

L'Eurocode «Basis of Design and Actions on Structures» sera une norme complète concernant les principes de dimensionnement et toutes les actions qui doivent être considérées lors de la conception d'une structure. Cet article décrit brièvement le cheminement du développement de l'Eurocode et présente le cadre général proposé ainsi que son contenu. Il résume également les progrès obtenus pendant son développement et la planification pour compléter le travail en cours.

ZUSAMMENFASSUNG

Der Eurocode 1 «Grundlagen für Entwurf, Bemessung und Konstruktion und Einwirkungen auf Tragwerke» wird eine umfassende Norm für die Praxis mit Angaben zu den Grundlagen für die Tragwerksplanung sowie Informationen zu all denjenigen Einwirkungen, die beim Entwurf und der Bemessung von Tragwerken beachtet werden müssen. In kurzer Form werden die Ausgangslage und Vorbereitungen dargelegt, sowie der vorgeschlagene Rahmen und dessen Inhalt dargestellt. Der heute erreichte Stand in der Entwicklung wird zusammengefasst und das weitere Vorgehen bis zum Abschluss der Arbeiten wird beschrieben.

1. INTRODUCTION

1.1 Scope

1.1.1 When complete, this Eurocode, which is generally referred to as Eurocode 1, will serve two purposes. First it will provide details of the basis of design for building and civil engineering structures. Secondly it will contain information on all actions which it is necessary to consider in the design of such structures. It will be a comprehensive code relating to a wide range of structures including buildings, bridges, towers, masts, silos, tanks, and chimneys. It will not, however, specifically cover exceptional structures such as nuclear reactors and dams, although the rules given for particular actions may be applicable to the design of these structures.

1.1.2 Eurocode 1 is the first in the series of nine Eurocodes which will, in due course, present common rules for the design of structures made of the major materials. The design of any particular structure on the basis of the Eurocodes is made by the use of the relevant Parts of Eurocode 1 together with the appropriate other Eurocode which gives the design rules for the specific structural material. The Eurocodes are not concerned specifically with the appraisal of existing structures.

1.2 Background

1.2.1 The preparation of the Eurocodes for the design of structures was initiated by the Commission of the European Communities in 1976 who established a Steering Committee made up of national delegations from the Member States to oversee the work. The work which was undertaken by drafting groups of experts under contract to the Commission, did not initially include development of rules for actions. It did include the preparation of a draft code on common unified rules for all types of construction and common safety requirements. These rules were not operational but were provided as a basis for preparing the operational Eurocodes. The rules were published in 1984 [1].

1.2.2 In 1984 the Steering Committee agreed to a proposal that an enquiry on national codes and standards concerned with actions be undertaken by the Building Research Establishment (BRE) amongst Members States. The report of the enquiry concluded that the preparation of a Eurocode for Actions on Structures was feasible. With the agreement of the Steering Committee a small Task Group was established to advise on the steps necessary. The Task Group was supported by national bodies including the Building Research Establishment (BRE), Centre Scientifique et Technique du Batiment (CSTN), Institut für Bautechnic (IfBt), and the Danish Building Research Institute (SBI).

1.2.3 An outline for a comprehensive Eurocode for Actions was proposed, together with suggestions for the first stages of the work, based on preparatory studies by Task Group members. The proposal was accepted by the Steering Committee in 1985.

1.2.4 An inherent feature of the proposed Eurocode 1 was that it would be suitable for structural design based on the limit state concept using the partial safety factor format.

1.2.5 Priority was first given in developing Eurocode 1 to the most important actions related to the design of building structures. The aim was to have the Parts covering these actions - gravity loads, imposed loads, snow loads, wind loads and actions due to fire - available for use when, or as soon as possible after, publication for experimental use by the European Committee for Standardisation (CEN) of ENV Eurocode 2: Part 1: Concrete Structure and ENV Eurocode 3: Part 1: Steel Structures.

1.2.6 The scope of the work was extended to include traffic loads on bridges (road and rail) and loads in silos and tanks leading in 1990 to completion of draft Eurocode documents [2] covering;

- General rules (for buildings)
- Densities of building materials and stored materials
- Permanent actions due to gravity
- Imposed loads on floors and roofs
- Snow loads
- Wind loads: static actions
- Actions on structures exposed to fire
- Loads in silos and tanks
- Railway loads (in relation to bridges)

For traffic loads on road bridges reports were presented to the Commission of technical studies and giving proposals for drafting [3].

1.2.7 The transfer of the technical work of preparation of the Eurocodes from the Commission of the European Communities to the European Committee for Standardisation (CEN) took place in 1990 [4]. Whilst establishing the programme of mandates of the Commission for the continuation of the work, the opportunity was taken to reorganise the comprehensive framework of Eurocode 1 into Parts.

Part 1: Basis of design

Fundamental requirements, limit state concept, general definitions and classifications concerning actions, material properties, geometrical data, load arrangements and load cases, common design requirements, durability aspects.

Part 2: Gravity and imposed loads, snow, wind and fire loads

General basis for determining actions for use in the structural design of Buildings, Bridges and Civil Engineering Works and specific rules for actions on Buildings arising from gravity, imposed loads on floors and roofs, snow, wind and fire.

Part 2A: Thermal actions

Data and rules for the determination of temperatures in components and structures for use in the structural design of Buildings, Bridges and Civil Engineering Works.

Part 2B: Construction loads and deformations imposed during execution

General basis for determining actions arising in the execution of Buildings, Bridges and Civil Engineering Works and for taking them into account in structural design.

Part 2C: Accidental Actions

General basis for determining accidental actions arising from impact, explosions and seismic events and rules for taking them into account in the structural design of Buildings, Bridges and Civil Engineering Works. (This Part will refer for seismic events to Eurocode 8: Design of Structures in Seismic Regions.)

Part 2D: Water and Wave loads

Basis for determining actions arising from flow of water and waves for use in the structural design of Buildings, Bridges and Civil Engineering Works.

Part 2E: Soil and water pressure

General basis for determining actions from soil and water pressure for use in the structural design of Buildings and Civil Engineering Works

Part 3: Traffic loads on bridges

Basis for determining, for use in the structural design of road bridges and mainline railway bridges, the actions arising from traffic and pedestrian loads.

Part 4: Loads in silos and tanks

General basis for determining, for use in structural design, the actions in Silos and Tanks arising from the storage of bulk materials.

Part 5: Actions induced by Cranes and Machinery

General basis for determining, for use in structural design, the actions on Buildings, Bridges and Civil Engineering Works arising from the operation of cranes and machinery.

Part 10: Actions on structures exposed to fire

Mechanical actions and standard fire exposure. Supplement to Part 2 rules for the actions to be considered in fire exposure.

3. ORGANISATION AND THE PREPARATION WORK

3.1 CEN Sub-Committee TC250\SC1

3.1.1 When the mandate to elaborate the Eurocodes was given to CEN in 1990, Technical Committee TC250 was established with Sub-committees, one responsible for each Eurocode [4]. Sub-committee TC250/SC1 undertook the task of preparation of the Eurocode 1: Basis of design and Actions on Structures. The Swiss Association for Standardisation (SNV) was appointed as the Secretariat for Subcommittee SC1. The work is being undertaken by the Swiss Society of Engineers and Architects (SIA) on behalf of SNV.

3.1.2 The scope of the work of Subcommittee TC250/SC1 was agreed by TC250 as follows:

'To prepare and maintain European Standards in the field of structural design rules for building and civil engineering works covering general rules for determining actions for use in design, and special and additional rules for actions arising from gravity, imposed loads, snow, ice, wind, thermal actions, currents and waves, soil and water pressure, and traffic and pedestrian loads on bridges; execution loads and deformations; actions from storage of bulk materials in silos and tanks, actions induced by cranes and machinery, accidental actions and actions on structures exposed to fire'.

3.1.3 The primary responsibility of Subcommittee TC250/SC1 is the elaboration of the rules for actions in Parts 2-5 and 10. Since basis of design concerns the specification of actions, the assessment of design resistance and design verification, the preparation of Part 1: Basis of design is being undertaken under the guidance of the Co-ordination Group thus allowing all Subcommittees the opportunity of comment during the formative stages of the draft.

3.1.4 At the inaugural meeting of Subcommittee TC250/SC1 in Zurich in December 1990, the programme of work in accordance with the mandates was agreed and Project Teams were established.

3.1.5 The Project Teams are made up of experts and are preparing drafts, with the assistance of the Subcommittee's technical secretaries and in consultation with the national technical contacts, for approval by Subcommittee SC1 following CEN procedures. This procedure, which is adopted by all the TC250 Subcommittees, should provide the most rapid progress to agreement on harmonisation of actions for use in the design of structures.

3.2 Programme of Work

3.2.1 Essentially the first phase of work - the initial programme - is to advance the results achieved by the previous Task Group [3, 4] to produce European prestandards (ENV).

3.2.2 The initial programme of work under the aegis of CEN was established with target dates as follows:

	Target Dates		
	Approval of draft by Project Team	Approval by SCI as ENV	Publication as ENV
Part 1	April 1993	October 1993	April 1994
Part 2	January 1993	July 1993	January 1994
Part 3	April 1993	October 1993	April 1994
Part 4	December 1992	June 1993	December 1993
Part 10	October 1992	April 1993	October 1993

3.2.3 The preparation of the remaining parts of the Eurocode for Actions - Parts 2A, 2B, 2C, 2D, 2E, and 5 - will not commence until 1993 at the earliest when specific mandates covering them are expected to be issued by the Commission. In the meantime preparatory work is in hand by the Secretariat.

3.2.3 The development of the Parts covered by the initial programme, the main technical aspects and the progress of preparation is described in the companion papers to this overview [5].

4. CONCLUSIONS

4.1 Scope

4.1.1 Eurocode 1 is a comprehensive code of practice providing information on basis of design and all actions which it is necessary to consider in the design of structures.

4.1.2 Eurocode 1 is being prepared specifically for use with Eurocodes 2 to 9. It anticipates design verification based on the limit state concept using the partial safety factor format.

4.2 Preparation

4.2.1 The initial programme of work to prepare Eurocode 1 is well advanced. The publication of European Prestandards (ENV) covering Basis of design, Gravity, Imposed loads, Snow, Wind and Fire loads, Traffic loads on bridges, and Bulk materials' loads in silos and tanks is targeted for 1993 and 1994.

5. REFERENCES

1. Eurocode No. 1: Common unified rules for different types of construction and material. Commission of the European Communities. 1984. EUR 8847.

2. Eurocode for Actions on Structures, Documents (1), (2) and (3). Draft: June 1990.

3. Concerning development of models of traffic loading and rules for the specification of bridge loads. Final report to the Commission of the European Communities - ref PRS/90/7750/RN/46 - November 1991.

4. BREITSCHAFT G., The conceptual approach of the Structural Eurocodes. IABSE International Conference 'Structural Eurocodes', September 1992.

5. Eurocode 1: Basis of design and actions on structures. Papers on Parts 1, 2, 3, 4 and 10. IABSE International Conference 'Structural Eurocodes', September 1992.

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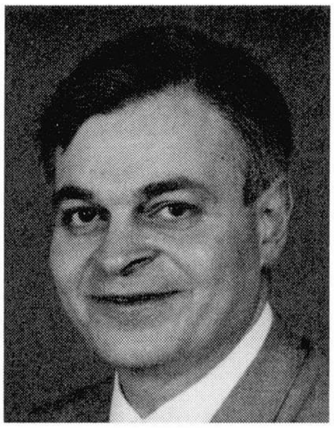
EC 1: Gravity Loads and Densities of Building and Stored Materials

EC 1: Charges de gravité et densités des matériaux permanents et entreposés

EC 1: Schwere Lasten und Dichte von Baustoffen und Lagergütern

Haig GULVANESSIAN
Civil Eng.
Build. Res. Establ.
Garston, UK



Haig Gulvanessian, born 1941, is a graduate of the Universities of London and Wales. Before being appointed to his present post he was at the Departments of Transport and Environment UK, where he was concerned with the design of bridges, elevated roadways, and buildings.

SUMMARY

This paper describes the background against which Eurocode 1: Part 2.2: Gravity Loads and Densities of Building and Stored Materials is being drafted. It identifies some of the problems in achieving a fully harmonised code and discusses future development of the Code as other CEN standards become available.

RESUME

Cet article décrit l'esprit dans lequel l'Eurocode 1: partie 2.2 «Gravity Loads and Densities of Building and Stored Materials» a été écrit. Il souligne les difficultés à réaliser un code adapté harmonieusement aux besoins de tous les pays et présente quelques suggestions pour remédier au problème. L'article aborde aussi les développements futurs du code au fur et à mesure que d'autres codes CEN entreront en vigueur.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die Grundlagen, auf denen der Entwurf zu Eurocode 1, Teil 2.2, über Eigengewichtsannahmen beruht. Er streicht einige der Probleme heraus, die sich aus der angestrebten Harmonisierung ergeben und diskutiert die zukünftige Entwicklung im Zuge der Einführung weiterer CEN-Normen.

SUMMARY

This paper describes the background against which Eurocode 1 : Part 2.2 : Gravity Loads and Densities of Building and Stored Materials is being drafted. It identifies some of the problems in achieving a fully harmonised code and discusses future development of the Code as other CEN standards become available.

INTRODUCTION

In developing this part of Eurocode 1, consideration was given to the contents of the National Codes of the CEN Member States and the International standard ISO 9194[1]

There are however differences in the scopes and specifications of the codes of the CEN Member States relating to Gravity Loads and Densities of Building and Stored Materials. For example National Codes of particular countries provide considerable detail, with much of this detail based on comprehensive supporting Standards; while other countries offer little guidance. Additionally the guidance that is available is at times somewhat contradictory. These differences have imposed restraints and limitations to the content of Eurocode 1 : Part 2.2:

The Project Team drafting this part of Eurocode 1 are the Technical Secretary of CEN/TC/250/SC1, Mr H Gulvanessian, Mr J Nielsen (Denmark) and Mr J Tory (UK)

SCOPE AND FIELD OF APPLICATION

This part of Eurocode 1 applies to the weights of

- materials used in construction;
- individual structural elements;
- parts of structures and of whole structures;
- some fixed non-structural items; and
- materials used in construction

As special cases, it also covers the weight of certain movable light weight partitions, materials for bridge construction, services and earth and soil pressures. The code provides specific advice for the determination of the weight of the following structural elements; floors and walls, claddings and finishes and roofs.

The Code gives,

i) representative values for the Bulk Weight Densities of building materials;

ii) representative values for the Bulk Weight Densities for a range of stored materials relating to building and construction, agriculture, liquids, solid fuel and industry;

iii) the angle of repose for particular stored materials; and

iv) methods for the assessment of the representative values of permanent actions due to gravity.

BASIS OF BULK WEIGHT DENSITY VALUES

There is in general little statistical basis for the load values given in current National and International Codes and no new research has been carried out for this Eurocode. It is not therefore possible to describe the load values included in this Eurocode as either mean or characteristic values since both of these terms imply some understanding of the underlying statistical distribution of the load values. Loads in this part of EC1 are therefore described as representative values. For materials where the bulk weight density has significant variability according to its source a range of values is provided in the Code.

EVALUATION OF ACTIONS DUE TO GRAVITY

Unless more reliable data is available (ie from product standards, the producer or by weighing), the Code recommends that the weights of individual elements (eg. beams or columns) be estimated from their dimensions and the densities of their constituent materials; the weights of parts of structures (eg. whole floors or whole storeys) and of non-structural elements (eg. plant) be determined from the weights of the elements of which they are composed. It recommends that dimensions used should be intended values of geometric properties (in general taken from the drawings).
For situations where more accurate values are required (eg. where a design is likely to be particularly sensitive to variations in dead load) the code recommends that a

representative sample of the materials to be used, at representative moisture contents, be tested.

When the self-weight of a component or element is likely to be significantly influenced by time-dependent effects (eg. moisture, dust accumulation etc.) the code recommends that appropriate allowance should be made.

For certain situations the code recommends that upper and lower values for the permanent actions on structures should also be considered. Account shall also be taken of possible variations in the thickness of finishes; eg. when the thickness depends on the deflection of the structural component to which the finish is applied. Examples of these situations are

- thin concrete members
- when there is uncertainty about the precise value of the dead load; and
- where dimensional alternatives and the exact materials to be used remain open at the design stage.

FUTURE DEVELOPMENT

The draft being developed at the present time will be presented in a 'final' form to CEN/TC/250/SC1 for submission for voting as a prENV by 31 January 1993.

In drafting the Code, a particular problem has been the lack of harmonised specifications and descriptions for many of the building and stored materials. CEN Standards on many of these items are expected to become available in the future and after the ENV stage amendment of this part of the Eurocode can be expected to reflect consideration of such standards as they become available.

CONCLUSIONS

This paper has described the basis of Eurocode 1 : Part 2.2 : Gravity Loads and Densities of Building and Stored Materials, the first steps to produce a fully harmonised Code.

REFERENCES

1. ISO 9194 : 1987 "Basis of Design of Structures - Actions Due to Self-Weight of Structures, Non-Structural Elements and Stored Materials".

EC 1: Imposed Loads on Buildings

EC 1: Charges d'exploitation dans les bâtiments

EC 1: Verkehrslasten im Hochbau

Gerhard SEDLACEK
Prof. Dr.
RWTH Aachen
Aachen, Germany

Gerhard Sedlacek, born in 1939, is Professor for Steel Construction at the RWTH Aachen. He has been convener of the Project Team for the preparation of the chapter «Imposed loads on floors and roofs» in Eurocode 1.

SUMMARY

After recalling the list of contents of Part 2.4 of Eurocode 1 «Imposed Loads on Buildings» the background of the choice of the loading models and the numerical values for loads on roofs and floors is presented. The studies that have been carried out include both probabilistic approaches and comparisons of rules in national codes. They are documented in a Background Document to Part 2.4.

RESUME

Après une présentation du contenu de la partie 2.4 de l'Eurocode 1 «Charges d'exploitation dans les bâtiments», l'article traite des bases et du choix des modèles d'actions ainsi que des valeurs numériques pour les charges sur les toitures et planchers. Les études entreprises concernent aussi bien des approches probabilistes que des comparaisons de normes nationales. Elles sont détaillées dans un document annexe à la partie 2.4.

ZUSAMMENFASSUNG

Nach Darstellung des Inhaltsverzeichnisses des Teiles 2.4 des Eurocode 1 «Verkehrslasten im Hochbau» wird auf den Hintergrund der dort angegebenen Belastungsmodelle und der Zahlenwerte für Lasten auf Decken und Dächern eingegangen. Dabei wird auch auf die Untersuchungen mit probabilistischen Ansätzen und die Vergleiche mit nationalen Normen hingewiesen, die im Hintergrundbericht zu dem Teil 2.4 dokumentiert sind.

1. SCOPE OF THE PART "IMPOSED LOADS ON BUILDINGS"

In the part "Imposed Loads on Buildings" of Eurocode 1 loaded floor and roof areas are divided into four classes according to their use

- areas in dwellings, offices etc.
- garages and vehicles traffic areas
- areas for storage and industrial activities
- roofs.

The standard gives numerical values for the floor and roof loads in buildings including parking and vehicle traffic areas.

For areas for storage and industrial activities only guidance for the determination of numerical values is given.

The list of contents of the part "Imposed Loads on Buildings" can be taken from fig. 1.

Figure 1: List of Contents of Part 2.4 "Imposed Loads on Buildings" of Eurocode 1

2. BACKGROUND OF THE MODELS AND NUMERICAL VALUES [1]

2.1 Areas of dwellings, offices etc. [1],[2]

For areas of dwellings, offices etc. the imposed loads depend on the type of occupancy, see fig. 2.

The loads may be caused by:

- furniture and moveable objects (e.g. light moveable partitions), loads from commodities the contents of containers.

  These loads are at certain points in time subjected to considerable instantaneous changes in their magnitudes, mainly due to change of occupancy or tenant, change of use etc. Between these instantaneous changes the load varies very slowly with time and the magnitudes of the variations are generally small, see fig. 2a.

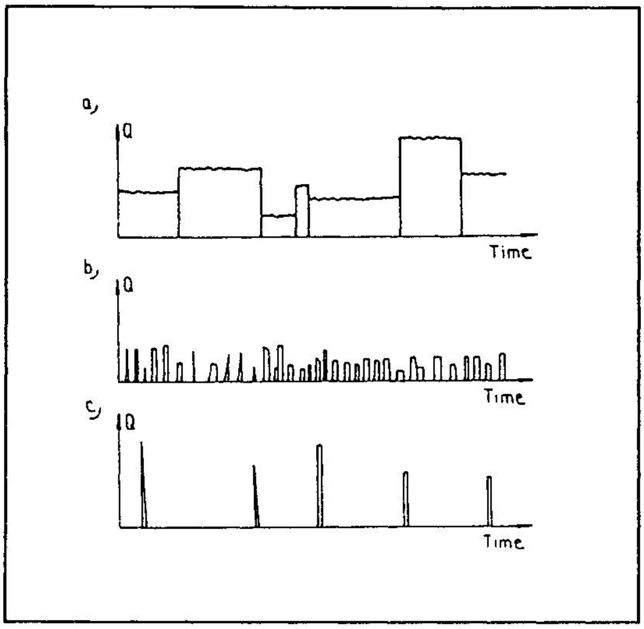


Figure 2: Time variability of the magnitude Q of the load
a) Load caused furniture and heavy equipment
b) Load caused by persons in ordinary load situations
c) Loads in special load situations

- normal use by persons. These loads are often periodical and only present during a relatively small part of the time, e.g. for school rooms only about 1/4 of the day, as illustrated in fig. 2b. The proportion between the load caused by persons and the load caused by furniture depends on the type of locality. E.g. for residential buildings it is small, in theatres and on corridors it is great. In some cases the loads from persons may also cause dynamic effects, e.g. in dancing halls.
- extraordinary use, such as exceptional concentrations of persons or of furniture, or the moving or stacking of commodities which may occur during reorganization or redecoration. These special situations occur during a short or moderate period of time, however sufficiently often during the lifetime of a building to make it necessary to take them into account, fig. 2c.

In an attempt to determine the design values and the characteristic values of imposed loads on a statistical basis the following assumption have been made:

1. In principle for the description of imposed loads it appeared appropriate to consider separately the variation in space and the variation in time.
2. For the variation in space for practical reasons it is normally usual to represent the "per definition" discrete loads by means of an equivalent uniformly distributed load. This uniformly distributed load is dependent on the tributary area, and also on the static system of the component to be designed.
3. The variation in time is taken into account by modelling the load by two components, fig. 3:
   - a quasipermanent (sustained) load, fig. 3a, the magnitude of which represents approximately the time average of the real fluctuating load between the changes of occupancy, including herein also the weight of persons who are normally present. The magnitude of the fluctuations between the changes of occupancy will then be included in the uncertainties of the sustained load.
   - an intermittent load, fig. 3b to represent all kinds of live load not covered by the sustained load, e.g. the loads due to extraordinary use.

The combined sustained and intermittent live load is shown in fig. 3c.

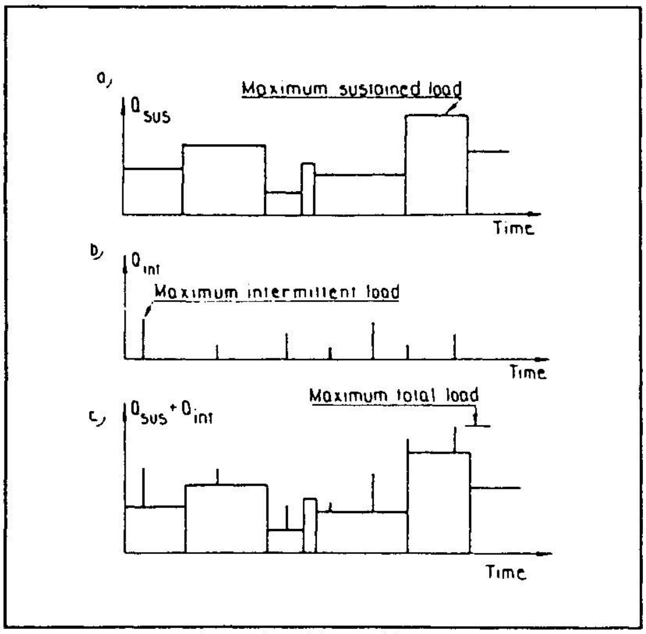


Figure 3: Sustained load (Q_{sus}), intermittent load (Q_{int}) and total load as stochastic process representing the variability

4. To determine the design values a reference period of 50 years and a reliability index β = 3.80 has been adopted and the characteristic values p_K were determined from the design values p_d by

$$p_K = \frac{p_d}{\gamma_Q}$$ where γ_Q = 1.50 was used.

Unfortunately the statistical database for the determination of the characteristic values is rather poor; the numerical load measurements in the literature [4] deal mainly with quasipermanent loads parts in some areas of representative use only, whereas little is known about quasipermanent loads in case of other types of use (e.g. warehouses, archives, libraries, tool sheds) and about short term loads, where estimations are necessary.

Fig. 4 gives some values determined in this way.

Imposed Load	Tributary area [m²]	p_K [kN/m²]	ψ_0	ψ_2
Office building	10	1,90	0,44	0,27
	50	0,95	0,68	0,50
Residental building	10	1,75	0,51	0,23
	50	0,87	0,69	0,32
Commercial building	10	2,10	0,45	0,14
	50	1,00	0,66	0,31
School	10	2,20	0,50	0,23
	50	1,30	0,67	0,37
Hotel	10	2,30	0,54	0,09
	50	0,90	0,72	0,26
Hospital	10	0,80	0,58	0,43
	50	0,55	0,31	0,56

Figure 4: Characteristic values and combination values determined on a statistical basis.

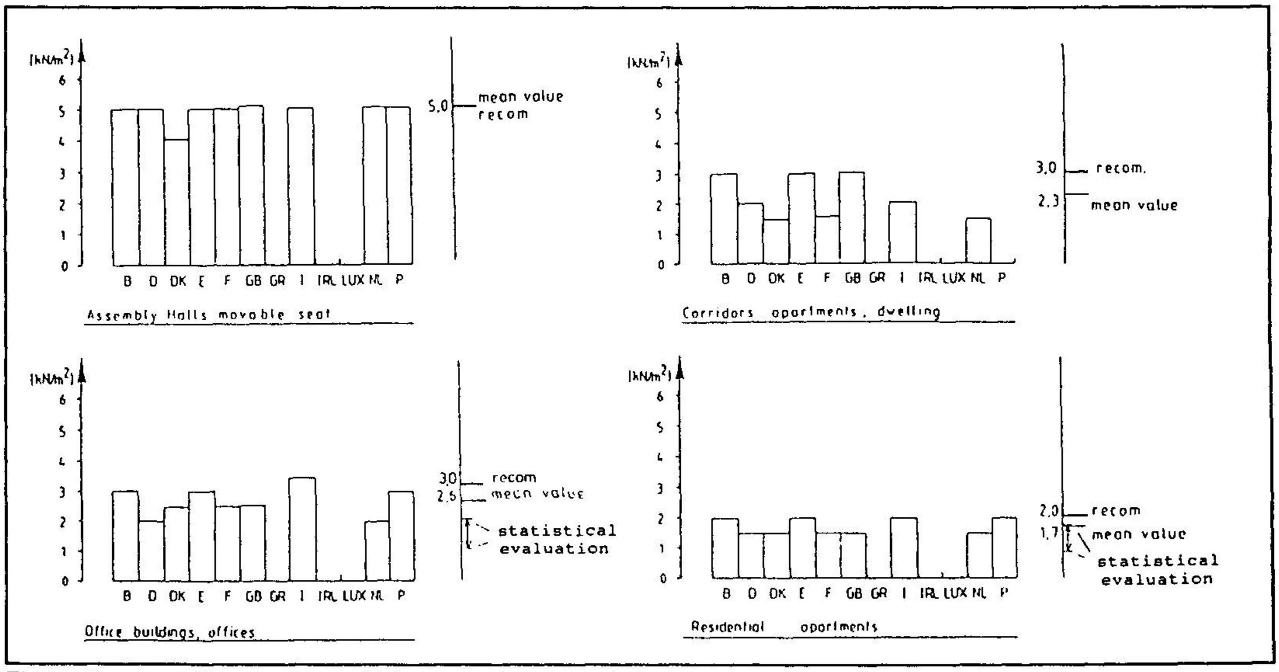


Figure 5: Comparison of European load regulations.

As the justification of all characteristic values on the basis of statistical data could not be reached, a more pragmatic way of deriving the load values was adopted in addition: they are derived from a comparison of the existing European national load regulations.

loaded areas		q_k [kN/m²]	Q_k [kN]	ψ_0	ψ_1	ψ_2
category A (domestic and residential activities)	- general	2,0	2,0	0,7	0,5	0,3
	- stairs	3,0	2,0	0,7	0,5	0,3
	- balconies	4,0	2,0	0,7	0,5	0,3
category B (public buildings, offices, schools, hotels)	- general	3,0	2,0	0,7	0,5	0,3
	- stairs, balconies	4,0	2,0	0,7	0,5	0,3
category C (assembly halls, theatres, restaurants, shopping areas)	- with fixed seats	4,0	7,0	0,7	0,7	0,6
	- other	5,0	7,0	0,7	0,7	0,6
category D (areas in warehouses, department stores)	- general	5,0	7,0	1,0	0,9	0,8

Figure 6: Imposed loads on floors in buildings.

Fig. 5 gives some examples from these comparisons.

Fig. 6 gives the the final proposals for the characteristic values of the uniformly distributed loads q_K and the combination factors ψ_i and for a concentrated load Q_K acting alone in dependance of the category on use of the floor.

2.2 Garage and vehicle traffic areas [1],[3]

In general the quasipermanent imposed load part does not exist in parking garages. Schematic diagrams for the daily fluctuations of the total number of cars in car parks depending on the location may be taken from fig. 7.

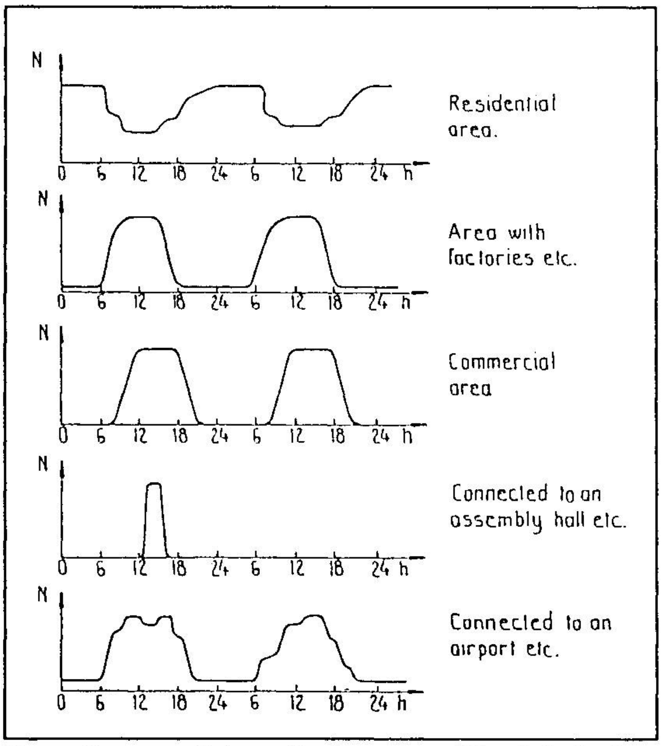


Figure 7: Schematic diagrams of the daily fluctuation of the total number of cars in car parks with different locations.

A propabilistic approach to determine the characteristic values of the uniformly distributed loads on parking areas may be based on the following assumptions:

- the spatial variability between different parking places which all are marked and have the same shape and magnitude in the whole car park is such that there is no correlation between the load values for the individual places and the same statistical data (Gaussian distribution) for the vehicle weights Q_i are valid for all of them.

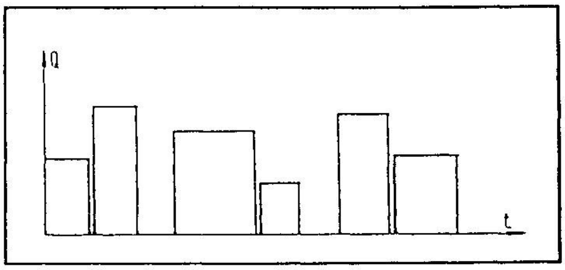


Figure 8: Rectangular wave renewal process

- the temporal characteristics of the loads at the individual parking places are modelled by a rectangular wave renewal load process, see fig. 8, that can be defined by the busy time t_d (hrs per day) when the car park is occupied and the dwell time t_u when a specific parking place is occupied continuously by the same car. The mean number of cars per day is then $\overline{p} = \frac{t_d}{t_u}$.

Design values and characteristic values calculated with these assumptions are given in fig. 9.

Imposed load on traffic areas	Tributary area [m²]	p_K [kN/m²]	ψ_0
parking areas :			
vertical	10	4,00	0,55
	50	2,11	0,62
diagonal	10	3,55	0,54
	50	1,83	0,60
approach ways	10	2,19	0,84
	50	0,76	0,79

Figure 9: Characteristic values and combination values determined on a statistical basis.

traffic areas		q_k [kN/m²]	Q_k [kN]	ψ_0	ψ_1	ψ_2
category E	vehicle weight: ≤ 35 kN	2,0	20	0,7	0,7	0.6
category F	vehicle weight: 35 kN - 160 kN	5,0	85	0,7	0,5	0.3

Figure 10: Imposed loads on garages and vehicle traffic areas.

These values have been used in defining the characteristic values and combination values in Part 2.4 of EC 1, which are given in fig. 10. By the simultaneous action of uniformly distributed and concentrated loads the influence of the tributary area has been taken into account.

2.3 Roofs

roofs	q_k [kN/m²]	Q_k [kN]
category G	0,75	1,5

Figure 11: Imposed load on roofs

Numerical values for uniformly distributed loads and concentrated loads acting independently are given for the roof category, where the roof is not accessible except for maintenance, repair and cleaning, see fig. 11. These values have been derived from a comparison of national codes.

2.4 Horizontal Loads on Partition Walls and Barriers due to Persons.

use of the loaded area	q_k [kN/m]
Category A	0,5
Category B	1,0
Categories C and D	1,5

Figure 12: Horizontal loads on partition walls and barries due to person

For barriers or partition walls having the function of barriers, horizontal forces due to persons are given as shown in fig. 12.

These values are not suitable for the design of railings in sports stadia.

2.5 Influence of the loading area

The influence of the loading area is taken into account in a different way for the loading area within one storey and for loading areas from several storeys.

For loading areas within one storey the influence if any is modelled by the simultaneous action of an area independent uniformly distributed load and a concentrated load.

For loading areas from several storeys (only relevant for areas with category A to D) a reduction factor

$$\alpha_n = \frac{2 + (n - 2)\ \psi_o}{n}$$

is used that is related to the number of storeys ($n > 2$) and the combination factor ψ_o.

3. REFERENCES

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EC 1: Snow Loads

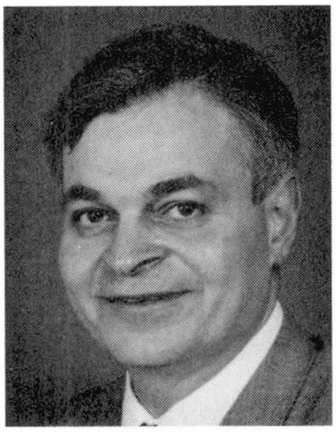
EC 1: Charges dues à la neige

EC 1: Schneelasten

Luca SANPAOLESI
Professor
University of Pisa
Pisa, Italy

Haig GULVANESSIAN
Civil Eng.
Build. Res. Establ.
Garston, UK

Luca Sanpaolesi born in Florence, Italy, in 1927, graduated in Civil Engineering at the Technical University of Pisa. At present he is Professor of Structural Engineering at the Pisa University and is involved in the studies of Eurocode EC 1.



Haig Gulvanessian, born 1941, is a graduate of the Universities of London and Wales. Before being appointed to his present post he was at the Departments of Transport and Environment UK where he was concerned with the design of bridges, elevated roadways, and buildings.

SUMMARY

This paper describes the background against which Eurocode 1: Part 2.5: Snow Loads is being drafted. It identifies some of the problems in achieving a fully harmonised code and provides some proposals to overcome them. It discusses topics for future developments of the code.

RESUME

Cet article décrit l'esprit dans lequel l'Eurocode 1: partie 2.5 «Snow Loads» a été écrit. Il souligne les difficultés à réaliser un code adapté harmonieusement aux besoins de tous les pays et présente quelques suggestions pour remédier au problème. L'article aborde aussi les développements futurs du code.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die Grundlagen, auf denen der Entwurf zu Eurocode 1, Teil 2.5 über Schneelasten beruht. Er streicht einige Probleme heraus, die sich aus der angestrebten Harmonisierung ergeben und zeigt mögliche Lösungen auf. Themen der zukünftigen Entwicklung werden diskutiert.

SUMMARY

This paper describes the background against which Eurocode 1: Part 2.5: Snow Loads is being drafted. It identifies some of the problems in achieving a fully harmonised code and provides some proposals to overcome them. It discusses topics for future developments of the code.

INTRODUCTION

Most of the Member States of the European Commission and EFTA have code clauses covering snow loads. The task of drafting the part of Eurocode 1 dealing with snow loads has been one of attempting to produce a code that in the first instance gives approximately the same values for a particular region as is given by the National Code covering the region. The Code being drafted, is capable of being developed as new meteorological data for the various CEN countries becomes available. In addition to taking account of the National Codes, the European Snow Code closely follows the International Standard on Snow Loads ISO 4355: 1981[1]

The Project Team drafting the code are - Professor L Sanpaolesi (Convenor)(Italy), the Technical Secretary of CEN/TC250/SC1 Mr H Gulvanessian (UK), Mr M Gränzer (Germany), Mr J Raoul (France), Mr R Sandvik (Norway) and Mr U Stiefel, (Switzerland).

SCOPE AND FIELD OF APPLICATION

Scope

The Code will provide guidance for the calculation of snow loads on roofs which occur in calm air or windy conditions; for the calculation of loads imposed by snow sliding down a pitched roof to a fence or other obstruction, and for loads due to snow overhanging the cantilevered edge of a roof.

The code will be applicable for use in all CEN Member States but will exclude regions where snow is present all the year and for sites at altitudes higher than 1500 metres above mean sea level.

Further limitations of the Code are that it does not provide guidance for:

- Impact snow loads resulting from snow sliding off or falling from a higher roof;
- Loads which could occur if snow and ice block gutters;
- the additional wind loads which could result from changes in shape or size of the building structure due to the presence of snow or the accretion of ice;
- ice loading (which will be covered elsewhere in Eurocode 1); and
- lateral loading due to snow (eg. lateral loads exerted by drifts).

Field of Application

The code applies to:

(a) new buildings and new structures; and

(b) significant alterations to existing buildings and existing structures;

designed in accordance with Eurocodes 2 to 9.

FORMAT OF CODE CONSIDERING CLIMATIC VARIATION

Both the initial deposition and any subsequent movement of snow on a roof are affected by the presence of wind. However, there is little data on the combined action of wind and snow to allow a direct statistical treatment. In design this is normally overcome by considering one or more critical design situations. These are usually snow deposited when no wind is blowing and snow deposited when the wind speed is sufficient to cause drifting, but without quantifying the precise wind speed. Due to the climatic variability across Europe the Eurocode provides different rules for the 'single snow event' concept and the 'multiple snow event' concept.

Single snow events occur in regions where the snow that falls is considered to be associated with single weather systems of about 3 to 4 days and where between one weather system and the next there is a reasonable expectation that the snow deposited on roofs will thaw. This requires the consideration of either uniform load or a drift load as the two are not expected to occur together.

Multiple snow events occur where snow is more persistent and where for example snow falling in calm conditions may be followed by further snow, carried by another weather system driven by wind and there may be several repetitions of these events before there is significant thawing. In these cases the accumulations are combined in a single load case.

The Eurocode does not actually call these concepts single and multiple events; but provides rules for such eventualities and it is the responsibility of the National Competent Authority to specify which should be used for a particular region.

SNOW LOAD ON THE GROUND

Characteristic value of snow load on the ground

The snow load on the ground is that assumed to occur in perfectly calm conditions. It is usually determined from records of snow load or snow depth measured in well sheltered areas. (ISO 4355 recommends in a deciduous forest). The characteristic value for the snow load on the ground is defined in the Eurocode as the value with an annual probability of being exceeded of 0.02. The variation of this snow load with geographical location will be given in map form.

Snow Maps

It is highly likely that the initial version of the Eurocode will rely on present National Data extracted almost directly from National Snow Loading Codes. The data will be adjusted to standardize on a definition of ground snow related to a return period of 50 years.

The reference altitudes for ground snow load vary in the codes of the CEN Member States. Standardization on this point is difficult as it would require extrapolation from the most reliable sources of information.

Two attempts have been made by members of the CEN Project Team for this Part of Eurocode 1 to establish a single European harmonised snow map to be included in the final version of the Eurocode.

The first (Gränzer)[2], considered all the available basic meteorological data for European Commission countries. This will not be completed in time because

- of the difficulty of establishing a common statistical relation between snow load and altitude;
- there is at present insufficient consistent data for many of the countries; and
- of the difficulty of extending the methodology to the EFTA countries within the time scales in the programme of work to produce the Eurocode.

The second (Sanpaolesi/del Corso)[3], defines six climatic regions, each being divided into snow load zones based on current National Code values of ground snow load correlated to a common return period and reference altitude. This should be looked upon as an interim solution and not a final harmonised map and it offers a compromise solution to the cross-frontier problem.

The investigation into producing a harmonised snow map of Europe will continue and be introduced in future developments of the code as this is an essential objective for the harmonisation process.

METHOD OF ASSESSMENT OF SNOW LOAD ON THE ROOF

Basis of Assessment

As most of the 18 CEN Member States approved ISO 4355: 1981 during voting with ISO and several of the National Codes are based on the same, the format of the Eurocode was based on that adopted in the ISO standard. By this it is meant that the snow load on the roof is derived by multiplying the snow load on the ground (S_o) by snow load shape coefficients (μ). The Eurocode contains sufficient data to allow the determination of both S_o and shape coefficients μ. In addition the Eurocode makes provision for further modification of the roof snow load by the introduction of a thermal coefficient factor for heat loss through the roof and an exposure coefficient factor to allow for abnormal exposure to the elements. These coefficients are taken from the proposed amendment of ISO 4355 (ISO DP4355 (1992))[4].

The Eurocode recommends that the snow roof load is treated as a variable action of medium term duration unless otherwise defined for particular regions in Annex A of the Code "Characteristic values of snow load on the ground".

Snow Load Shape Coefficients

Several different snow load shape coefficients must be considered for every design. These relate to different climatic conditions before, during and after the snow fall.

The Eurocode in the first instance will provide shape coefficients for monopitch, duo-pitched and multi-pitched roofs and coefficients for drifting at abrupt changes in roof height and at obstructions to roofs.

In general three primary loading situations can be identified and are accounted for in the coefficients provided in the code.

a) that resulting from a uniformly distributed layer of snow over the complete roof, likely to occur when snow falls with little wind (balanced load part);

b) that resulting from either an initially unbalanced distribution, local drifting at obstructions or a redistribution of snow which affects the load distribution on the complete roof, eg. snow transported from the windward slope of a pitched roof to the leeward slope (unbalanced load part due to drifting);

c) that resulting from a redistribution of snow from an upper part of the building (unbalanced load part due to sliding).

In the clauses from which the snow load shape coefficients are calculated, the limiting values applying to the 'single snow event' are based on the current UK code[5]. The limiting values for the multiple snow event are based on ISO 4355:1981.

FUTURE DEVELOPMENT

The draft being developed at the present time will be presented in a 'final' form to CEN/TC250/SC1 for submission for voting as a prENV by 31 January 1993.

After this the development of the code must continue to cover the requirements of the designers and to present them with information based on the latest results of research. It is possible these later versions will be based upon the latest ISO 4355. (ISO DP4355 (1992)).

Two particular areas needing development to ensure a fully harmonised code are the reappraisal of the ground snow loads and hence the production of a coordinated European Map and the reconsideration of the roof snow load shape coeffients and the duration with reference to Serviceability Limit States.

CONCLUSIONS

This paper has described the basis of Eurocode 1: Part 2.5: Snow Loads, the first steps to produce a fully harmonised European Snow Code. It describes difficulties in producing a fully harmonised code at this stage and provides items for future research from which a fully harmonised code will be completed.

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1. ISO4355/1981 - Basis of Design of Structures - Determination of Snow Loads on Roofs.
2. Gränzer - "Committee Paper"
3. Sanpaolesi/Del Corso - "Committee Paper"
4. ISO4355/1981 (DP 1992) - Basis of Design of Structures - Determination of Snow Loads on Roofs.
5. BS6399 Part 3 1988 - Loading for Buildings - Code of Practice for Imposed Roof Loads

EC 1: Wind Loads

EC 1: Charges dues au vent

EC 1: Windeinwirkungen

Hans P. RUSCHEWEYH
Prof. Dr.
RWTH Aachen
Aachen, Germany

Hans Ruscheweyh, born 1937, received his mechanical engineering, his Dr.-Ing. degree and his habilitation at the Technical University Aachen (RWTH), Germany. After working in the Aeronautical Industry he returned to the University, for research work in the field of wind engineering and dynamics of structures. He is now Prof. at the faculty of Civil Engineering at the RWTH Aachen.

SUMMARY

This paper explains the principle of the two procedures for the calculation of the wind load on structures: simplified and detailed methods. The first one has been developed from the detailed method for those buildings and structures which are not sensitive to dynamic effects. Examples are given for buildings with heights from 10 to 200 m, for a low rise large steel building and a 150 m high concrete tower.

RESUME

L'article présente le principe des deux méthodes de calcul, la méthode simplifiée et la méthode détaillée, utilisées pour calculer des surcharges de vent exercées sur des bâtiments. La première a été déduite de la méthode détaillée pour des bâtiments non sensibles aux actions dynamiques du vent. Il donne des exemples pour des bâtiments dont la hauteur varie entre 10 et 200 m, pour un grand hangar de faible hauteur en acier et pour une tour en béton armé.

ZUSAMMENFASSUNG

Es wird das Prinzip der beiden Berechnungsmethoden, die «vereinfachte» und die «detaillierte» Methode, zur Windlastberechnung an Bauwerken, erläutert. Die erstere ist aus der detaillierten Methode für solche Bauwerke entwickelt worden, die nicht schwingungsempfindlich sind. Es werden Beispiele für Gebäude mit Höhen zwischen 10 und 200 m, für eine niedrige, grosse Stahlhalle und einen Stahlbetonturm gegeben.

1. INTRODUCTION

This paper presents a brief description of the actual draft of the Eurocode 1, part 2.7 "Wind Action" [1]. The process of developing the code is still going on, but the principles are more or less fixed. The existing EC-draft is based on modern knowledges in the field of windengineering which has been introduced into new national standards or drafts of national codes. Furthermore it follows the guideline of ISO-Standard of Wind load, TC 98 [2].

2. OBJECT, FIELD OF APPLICATION AND SCOPE

The code gives rules and methods for calculating the **static** and **dynamic** response of buildings and other structures, i. e. towers, masts, chimneys, bridges, walls etc. Because of the large variation of types of buildings and structures as well as its site location a **detailed method** is proposed which covers most of the practical cases and which is presented in a form for computer application. The detailed method covers all dynamic effects.

It is known, that the majority of the buildings require only a simple rule. For those buildings and structures not very sensitive to wind load, i. e. the wind load is not significant for the design, a **simplified method** is presented, too. These method does not cover dynamic effects and is therefore only applicable for buildings and structures where dynamic effects are negligible.

The simplified method does not take into account the reduction of the wind load with increasing building size (aerodynamic admittance function). Therefore this simple rule gives higher values than the detailed method. Fig. 1 illustrates this effect.

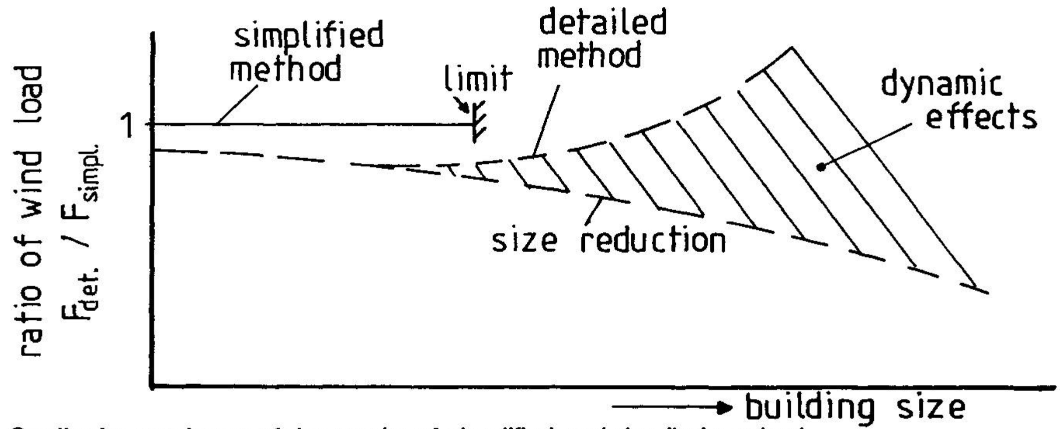


Fig. 1 Qualitative tendency of the results of simplified and detailed method.

In all cases it is allowed to apply the detailed method. These should be done if the economy of the design is significantly influenced by the wind load. Nevertheless the simplified method can be used within its limit of application for a first rough estimation.

3. CRITERIA FOR THE RANGE OF APPLICATION OF SIMPLIFIED METHOD

For buildings the criteria is only related to alongwind response and is given in Fig. 2. The graphs have been calculated using the detailed method and varying the design wind speed, the b/d ratio as well as the roughness category of the site.

The criterion includes an approximation for the fundamental frequency, n_x:

$$n_x \geq \frac{46}{h} \quad [Hz]$$

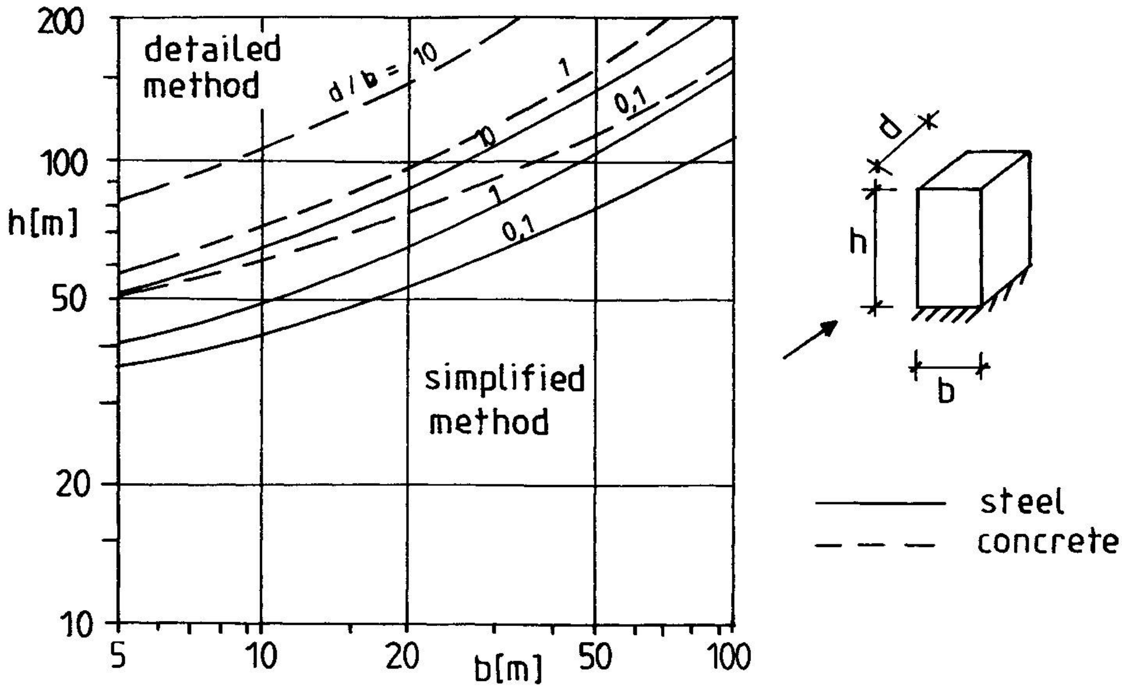


Fig. 2 Geometry criteria for application the simplified method for buildings.

From Fig. 2 it is obvious, that a wide range of buildings can be covered by the simplified method.

For chimneys, towers, bridges and similar slender structures the vortex shedding is the domain criteria. The criteria can be simple expressed by the slenderness h/d (s. Table 1)

	simplified method, if l/d ≤	
chimneys, towers	12	
posts	8	
bridges d	slabs	girder
l ≤ 200 m	12	20
	24	40

Table 1 Criteria for application the simplified method for bridges and elongated vertical structures.

4. COMPARISON OF SIMPLIFIED AND DETAILED METHOD

The comparison is made here for the resultant wind force, only. The resultant wind force, F_W, in the detailed method is

$$F_w = c_f \cdot A_{ref} \left(1+2g_x I_v \sqrt{Q_o^2+R_x^2}\right) c_r^2(z_{ref}) \cdot c_t^2(z_{ref}) \cdot c_x^2(x) \frac{\rho}{2} v_{ref}$$

where the quantities g_x, I_v, Q_o, R_x, $c_r(z_{ref})$, $c_t(z_{ref})$, $c_x(x)$ depend on the size of the structure, its natural frequency n_x, its damping δ_x, the aerodynamic parameters of the wind and the characteristic of the site fetch. For the simplified method the following approximation has been made:

g_x	=	peak factor = 3.5
$\sqrt{Q_o^2+R_x^2}$	=	background (Q_o) and frequency function (R_x) = 1
$c_t(z_{ref})$	=	topography coefficient = 1
$c_x(x)$	=	transition coefficient = 1

We get the expression for F_w in the simplified version:

$$F_w = c_f \cdot A_{ref} (1 + 7 I_v) c_r^2(z_{ref}) \cdot \rho/2\, v_{ref}^2$$

or

$$F_w = c_f \cdot c_e(z_{ref}) \cdot \rho/2\, v_{ref}^2 A_{ref}$$

where

$c_e(z_{ref})$ = $(1 + 7 I_v) c_r^2(z_{ref})$ = exposure coefficient (see Fig. 3), which includes the turbulence of the wind, I_v, and the roughness coefficient, $c_r(z_{ref})$, (wind speed variation versus heigth of the four roughness categories).

c_f = force coefficient which is given in Annex B of EC 1, part 2.7 for the different structures types

A_{ref} = reference area, which is related to c_f and defined in Annex B, together with c_f

z_{ref} = reference height of the structure which is referred to c_f and defined in Annex B together with c_f

ρ = air density = 1,25 kg/m^3

v_{ref} = reference wind speed, which is the mean wind velocity at **10 m above flat open country** (category 2 of the roughness categories) averaged over a **period of 10 min** with an annual **probability of exceedence of 0,02** (50 years return period). It can be found in the detailed wind maps of Annex A of EC 1, part 2.7 for the different countries.

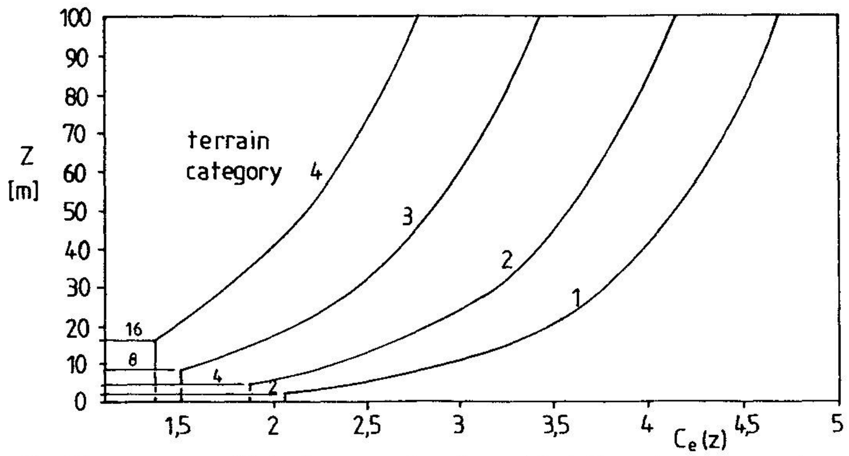
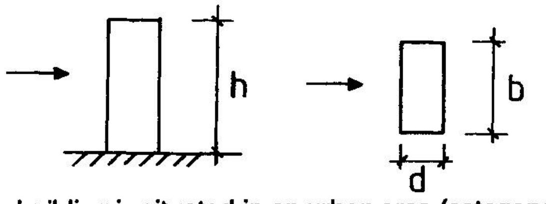


Fig. 3 Exposure coefficient c_e as a function of heigth z above ground.

5. EXAMPLES

5.1 Steel building of different height



h	=	10 ÷ 200 m
b	=	20 m
d	=	10 m

The building is situated in an urban area (category 2) with a reference wind speed of v_{ref} = 27,5 m/s. The force coefficient is set to constant, c_f = 1,3. From the criterion of Fig. 2 the simplified method may be applied up to a heigth of 50 m. In Fig. 4 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.

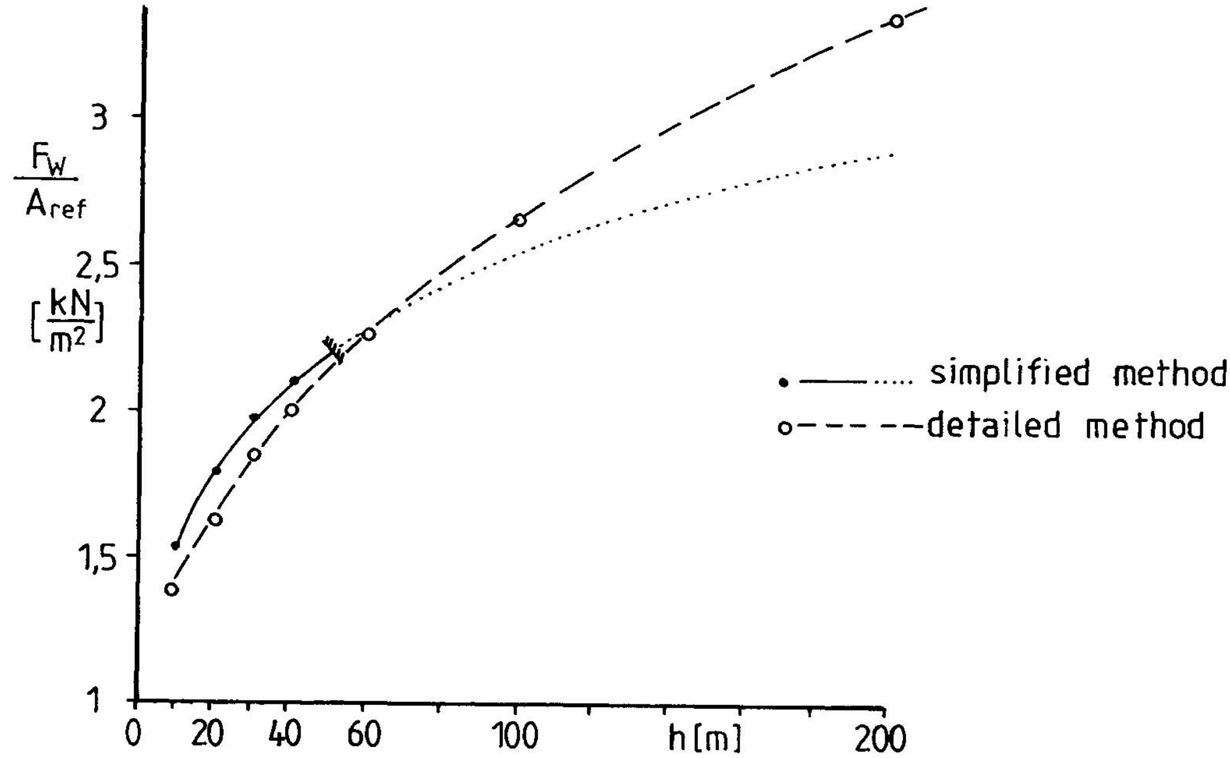
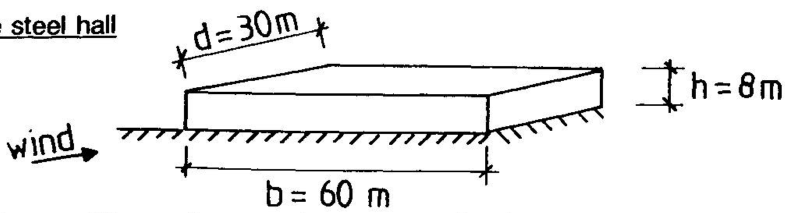


Fig. 4 Calculated wind force per m² for the buildings of example 5.1. Comparison of simplified method with detailed method.

5.2 Large low rise steel hall

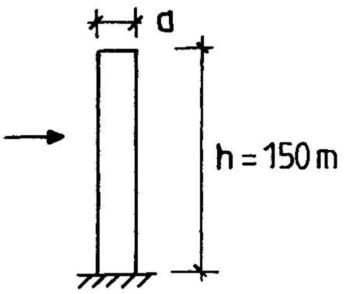


under the same site condition as in example 5.1 we get for the

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 factor in the simplified method.

5.3 Concrete tower



v_{ref} = 27,5 m/s

roughness category 2

5.3.1 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

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

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

5.3.2 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

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

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

6. CONCLUSION

The simplified method presents results which are close to those of the detailed method, if the structures are small or rather slender, i. g. the size factor as well as the dynamic effects are negligible. For large but rigid buildings, the size effect reduces the wind load. These effect is included in the detailed method only, and 20% reduction or more can be expected.

If the structures become sensitive to dynamic effects, the detailed method must be used. The simplified method will lead to unsafe results for those cases.

7. REFERENCES

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[2] ISO/TC 98/SC 3/WG 2 Wind Action on Structures, Final Draft, July 1990

EC 1: Actions on Structures Exposed to Fire

EC 1: Actions sur les structures exposées au feu

EC 1: Brandbelastung von Tragwerken

L. TWILT
Head
Techn. Centre Fire Prevention
Rijswijk, The Netherlands

Marita KERSKEN-BRADLEY
Dr. Eng.
Consulting Engineer
Munich, Germany

SUMMARY

Chapter 20 of the Eurocode on Actions specifically deals with actions on structures exposed to fire. It is intended for use in conjunction with the parts on structural fire design of the material orientated Eurocodes 2 to 6. A first draft was presented in 1990. Based on various comments, the draft has been improved and should be finalized at the beginning of 1993. This paper deals with some practical aspects of the 1990 draft and some future developments are highlighted.

RESUME

Le chapitre 20 de l'Eurocode sur les actions traite spécifiquement des actions sur les structures exposées au feu. Il est destiné à un emploi conjoint avec les éléments traitant du projet des structures sous l'effet des incendies des Eurocodes 2 à 6, consacré aux matériaux spécifiques. Un premier projet a été présenté en 1990. Il a été successivement amélioré sur la base de commentaires et un projet final devrait être établi au début de 1993. Cette présentation traite d'aspects pratiques du projet de 1990 et des développements possibles.

ZUSAMMENFASSUNG

Kapitel 20 des Eurocodes über Einwirkungen behandelt ausschliesslich die Brandbelastung von Tragwerken. Es ist für die gemeinsame Anwendung mit den Teilen über Brandbemessung in den werkstoffspezifischen EC's 2 und 6 gedacht. Ein erster Entwurf wurde im Juni 1990 vorgestellt. Aufgrund verschiedener Stellungnahmen wurde er überarbeitet und sollte Anfang 1993 in der Endfassung vorliegen. Der Beitrag behandelt einige praktische Aspekte der Fassung 1990 und beleuchtet zukünftige Entwicklungen.

1. INTRODUCTION

Chapter 20 of the Eurocode on Actions specifically deals with actions on structures exposed to fire. It is intended for use in conjunction with the parts on structural fire design of the material orientated Eurodes 2 to 6. A first draft was presented at a symposium in Luxembourg in June 1990 [1]. EC member states have been invited to send in their comments. Redrafting has started in autumn 1991. Versions for SC voting should be available by the end of 1992 or the beginning of 1993.

This paper deals with some practical aspects of the 1990 version of EC-Actions, chapter 20 [1]. Also some future developments will be highlighted.

2. GENERAL FEATURES

The following general features apply:

- accidental situation;
- fire situation;
- post-fire situation.

A direct consequence of the assumption that fires may be considered as accidental situations is that simultaneous occurrence with other (independent) situations need not to be considered.
Clearly, fire constitutes the dominant action in a fire design. Nature and extention of the fire should therefore be identified. As far as the nature is concerned, only fully developed fires inside the buildings are considered. If a building is divided into fire compartments, fire exposure is only in one compartment at a time. From this rule, the way in which building components are exposed (from one side only, or from more sides) can be determined.
In view of the generally accepted objectives of designing for fire (i.e. limiting risk with respect to life and property loss as a direct result of fire), [1] does not consider any post-fire situations.

3. THERMAL ACTIONS

3.1 General

In order to provide optimal guidance for practical application, in [1] the generally accepted design procedures for fire design are taken as a starting point. I.e., a central role is for the standard fire approach and the related grading system in terms of fire resistance. Using an analytical approach - as specified in EC 2 to 6 - rather than an experimental one, renders a relatively simple possibility to achieve unambiguous results. This is a key element in standardization. On the other hand one should realize that the standard fire concept is very global and that solutions are often far from reality. Under circumstances, economic building design requires therefore a more nuanced analysis. For this reason, in [1] the door is opened for more physically based, differentiated approaches as well. Details of these approaches are intended to be given in the appendices. In [1], these appendices are only outlined regarding their possible scope and contents in order to collect options during the national inquiry to the extent to which the various items should be pursued for further incorporation.

3.2 Standard fire exposure

For the gas temperature time relationship used in standard fire conditions, refer to Fig. 1. It is emphasized that the temperature curve is not sufficient to define fire exposure conditions. Also the (radiative and convective) heat transfer characteristics from the environment to the exposed members should be specified. The assumptions made in this respect in [1] are presented in Fig. 1 as well. It is noted that both the standard fire curve and the heat transfer characteristics have a conventional rather than a physical meaning.

3.3 Compartment fire exposure

Fully developed compartment fires (i.e. fires characterized by full involvement of all combustible material) are taken as a basis for fire engineering design. During the last decades, various calculation models have been developed for the calculation of the gas temperature in such fires. See for example [2,3]. The models are based on the heat & mass balance for a given situation and generally take into account the effect of ventilation conditions, fire load density and thermal properties of the construction elements surrounding the fire compartment. Extensive experimental research has been carried out to verify the models and a reasonable agreement between theory and experiment can be achieved. See Fig. 2.

The calculations result in quite nuanced relationships between gas temperature and time. For practical use such curves are felt to be too cumbersome. Moreover, the models generally take only physical parameters into account, i.e. any human interference with the fire process is excluded. It is suggested, therefore, to conventionalize the calculated fire curves to "design natural fire curves". For a set of such design curves, based on the model described in [3], refer to Fig. 3 [4,6]. For other design curves, see [2,5]. Note that specification of the heat transfer characteristics and the field of application is necessary.

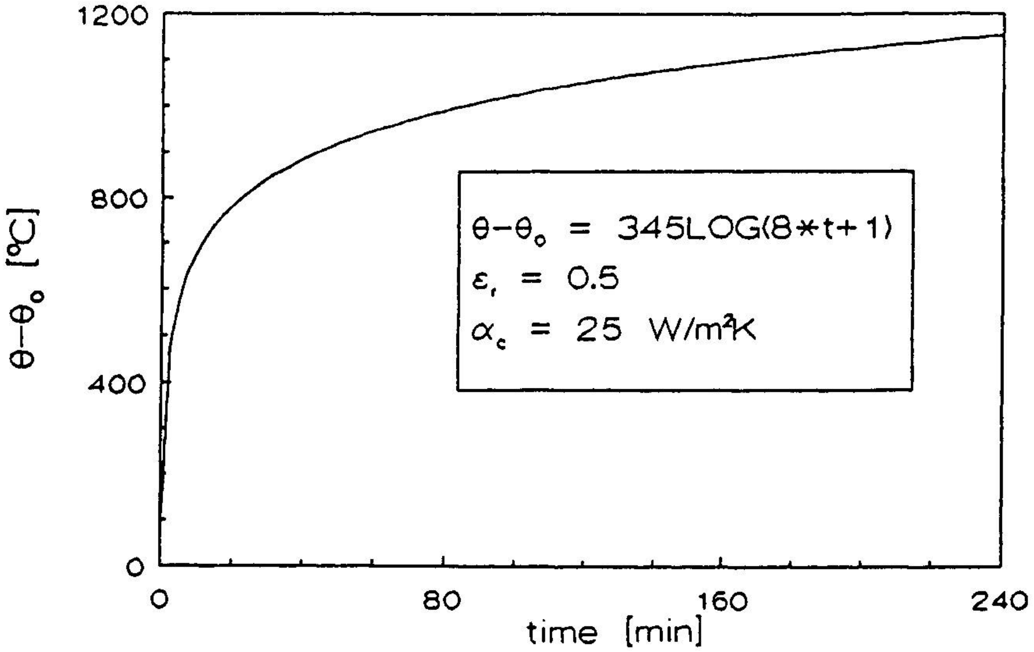


Fig. 1: The standard fire curve and associated heat transfer characteristics

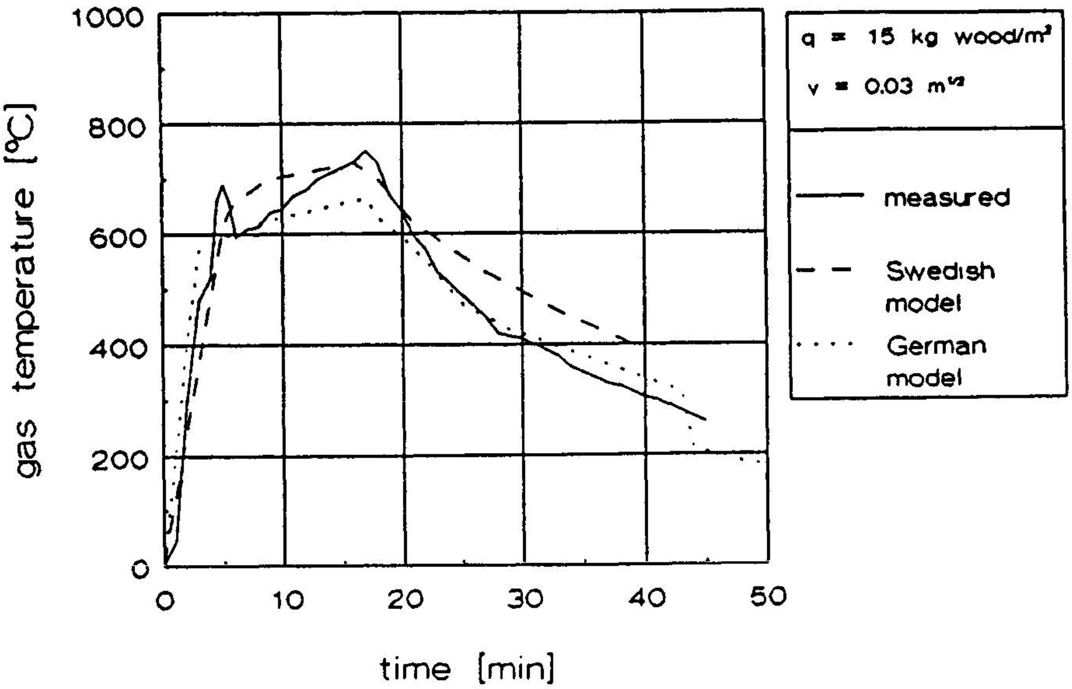


Fig. 2: Measured and predicted compartment temperatures (fire load density 15 kg wood/m², opening factor 0.03 $m^{1/2}$).

3.4 Practical implications

Using natural fire exposure together with the standard fire concept in one code, brings a question of consistency with regard to required levels of safety. In an approximate way, this problem has been solved by using the concept of effective fire duration. The effective fire duration (t_{eff}) is a quantity which relates compartment fire conditions to standard fire conditions;

$$t_{eff} = q \cdot w \cdot c \cdot \gamma \cdot \gamma_n \quad (1)$$

with:

q = fire load density
w = ventilation factor
c = conversion factor
γ, γ_n = safety, adaption factor

Alternatively, one can also express the fire load density in terms of effective fire duration. An effective fire duration equal to the required fire resistance gives the so-called nominal fire load density (q_n):

$$q_n = w_{c,\gamma} \cdot t_{f,r} \quad (2)$$

with:

$t_{f,r}$ = required fire resistance (= t_{eff})
$w_{c,\gamma}$ = $1/(w \cdot c \cdot \gamma \cdot \gamma_n)$

By way of convention it is postulated that an assessment based on standard fire exposure and a certain required fire resistance gives rise to the same safety level as an assessment based on compartment fire exposure and a corresponding fire load density

4. MECHANICAL ACTIONS

4.1 General

Mechanical actions cover:
- actions from normal conditions of use;
- indirect fire actions.

Indirect actions may occur as result of restrained thermal expansion and depend on the temperature development in the structural system and differences in stiffness. Indirect actions may develop in both isostatic and hyperstatic systems. A typical example of indirect actions due to fire are temperature induced stresses due to non-uniform temperature distribution over the cross section. These will occur in the centrically loaded concrete filled HSS-column, exposed to fire from all sides, presented in Fig. 4. For a qualitative presentation of the temperature distribution over the cross section and the pattern of additional stresses due to restrained thermal elongation, refer to Fig. 4a. The effect on the load bearing capacity is exemplified by the two buckling curves presented in Fig. 4b. Both curves are calculated for reinforced, concrete filled HSS columns (□ 300 x 300 x 7 mm), after 90 minutes standard fire exposure [7]. The solid curve includes the effect of the thermal induced stresses; this effect is ignored in the dashed curve. Depending on the buckling length, significant differences appear to occur.

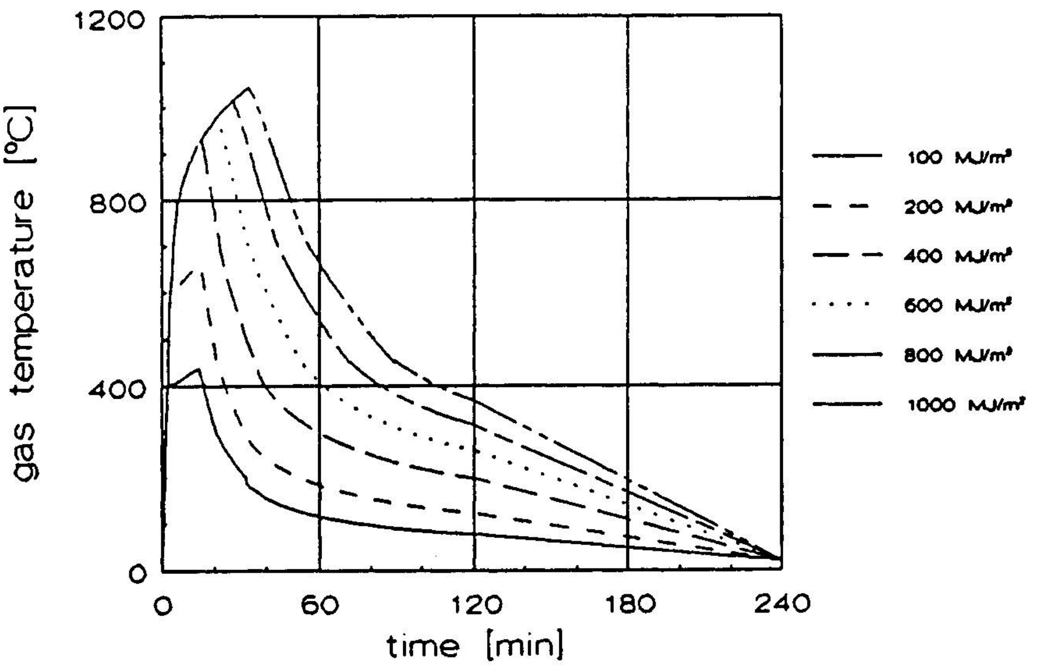


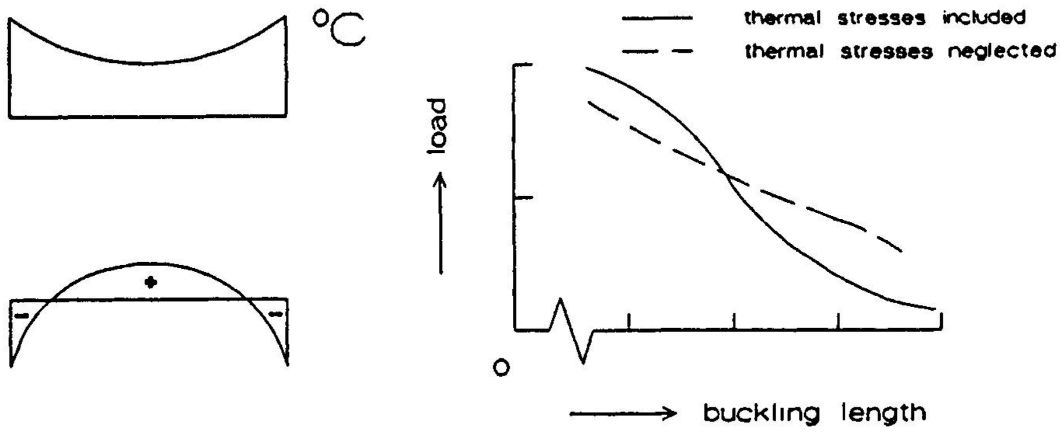
Fig. 3: Design natural fire curves for an opening factor of 0.06 $[m^{1/2}]$ and fire loads in the range of 100 to 1000 MJ/m².

4.2 Combination rule

In symbolic form the combination rule for action effects for room temperature conditions (so-called fundamental combination rule) reads [8,9]:

$$E_d = \gamma_G \cdot G_k + \gamma_{Q,1} \cdot Q_{k,1} + \Sigma\, \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \qquad (3)$$

with:



(a) distribution of temperature and thermal stresses (schematic)

(b) buckling curves at elevated temperatures

Fig. 4: The effect of restraint of thermal elongation on the load bearing capacity of fire exposed concrete filled steel columns.

E_d : design value of effect of actions for normal conditions,
γ_G : partial safety factor for permanent actions,
G_k : characteristic value of permanent actions,
$\gamma_{Q,i}$: partial safety factors for variable actions,
$Q_{k,1}$: characteristic value of the main variable actions,
$\psi_{0,i}$: combination factor for variable loads,
$Q_{k,i}$: characteristic value of the other variable actions,

The corresponding rule for the accidental situation reads [8,9]:

$$E_{d,acc} = G_k + \psi_{1,1} \cdot Q_{k,1} + \Sigma\, \psi_{2,i} \cdot Q_{k,i} + A_d \qquad (4a)$$

where:

A_d : the design value of the accidental action,
$\psi_{1,1} \cdot Q_{k,1}$: frequent value of main variable load,
$\psi_{2,i} \cdot Q_{k,i}$: time average of other variable loads.

Safety factors γ_g and γ_q are set to unity to account for the rare occurrence of an accidental situation. The main variable action is represented by its frequent value, the other variable actions are combined using their quasi-permanent (time average) values. These values account for the fact that it is unlikely that (all) variable actions will attain their characteristic value during the short duration of the accidental action.

In traditional fire testing, generally a "service load" was applied, resulting from the self weight of the construction and imposed loads. This is more or less equivalent to $G_k + Q_{k,1}$, i.e. no distinction is made between structures with small and large portions of self weight, implying a lower safety level for the latter. Hence, applying Eq. (4a) for fire design will give a more uniform safety level. With regard to the average safety level, it should be noted that, traditionally, indirect actions from fire exposure in terms of $A_d = A_{d,ind}$ were not considered.

The indirect actions are related to the fire, hence the corresponding combination factor obviously equals to unity.

It is suggested to use the accidental combination rule, for fire design with frequent and quasi-permanent values as specified for room temperature design [8,9]. With adopted notations the rule reads:

$$E_{d,f} = G_k + \psi_{1,1} \cdot Q_{k,1} + \Sigma \psi_{2,i} \cdot Q_{k,i} + A_{d,ind} \tag{4b}$$

From tentative calculations it follows that the combination factors in case of fire do not differ significantly from those specified for room temperature design [9]. In view of the uncertainties involved in both the physical and the statistical model, it has been decided to use room temperature values for the combination factors in the above combination rule. These depend on the category of the area under consideration and may vary for the main variable action in offices etc. between 0.5 and 1. See also [8].

4.3. Practical implications

Application of the above combination rule requires a complete global analysis for fire design. An important simplification may be achieved as follows:

If indirect actions due to fire do not occur or are negligible and only one variable (leading) action needs to be taken into account, the ratio between the design action effect for the fire situation and the corresponding value for the room temperature design follows from:

$$\frac{E_{d,f}}{E_d} = \frac{r + \psi_1}{\gamma_G r + \gamma_Q} \tag{5}$$

with:

$r = G_k/Q_k$

E_d, $E_{d,f}$, G_k, Q_k, ψ_1, γ_G, γ_Q: as defined under 4.2.

For values for the partial safety factors as suggested in [10] (i.e. $\gamma_G = 1.35$ and $\gamma_Q = 1.50$) and a calibration case defined by:

$\psi_1 = 1.0$ representative for imposed loads, area category D (i.e. public premises susceptible to overcrowding and accumulation of goods), this rendering the traditional service load,

$r = 1.0$ practical value for r, valid for heavy weight structures (e.g. normal weight concrete)

equation (5) yields:

$$E_{d,f} \approx 0.7\ E_d$$

This value is suggested in [1] of the Eurocode on Actions.

According to Eq. (5), the ratio between the design value for the action effect in case of fire and the corresponding value for normal conditions of use, depends, for a given set of partial safety factors, on two parameters only:

- the ratio between permanent and the main variable action (= r);
- the frequent value factor for the main variable action (= ψ_1).

In Fig. 5, $E_{d,f}/E_d$ is presented as function of r (= G_k/Q_k) and some practical values for ψ_1, taking into account partial safety factors for actions as suggested in [10] (i.e. γ_G = 1.35 and γ_Q = 1.50). It follows that, within a practical range for r -between, say, 0.5 and 1.5- the variation in $E_{d,f}/E_d$ is significant.

For:

r ≈ 0.5, (which is representative for steel structures)

and

ψ_1 = 0.5 (which is representative for area category A, e.g. dwellings, offices, hotels)

it follows:

$E_{d,f} \approx 0.45\ E_d$

Hence, under the given circumstances, the design value for the action effects may taken significantly smaller taken than the global value specified in the 1990 draft of chapter 20 of the Eurocode on Actions [1].

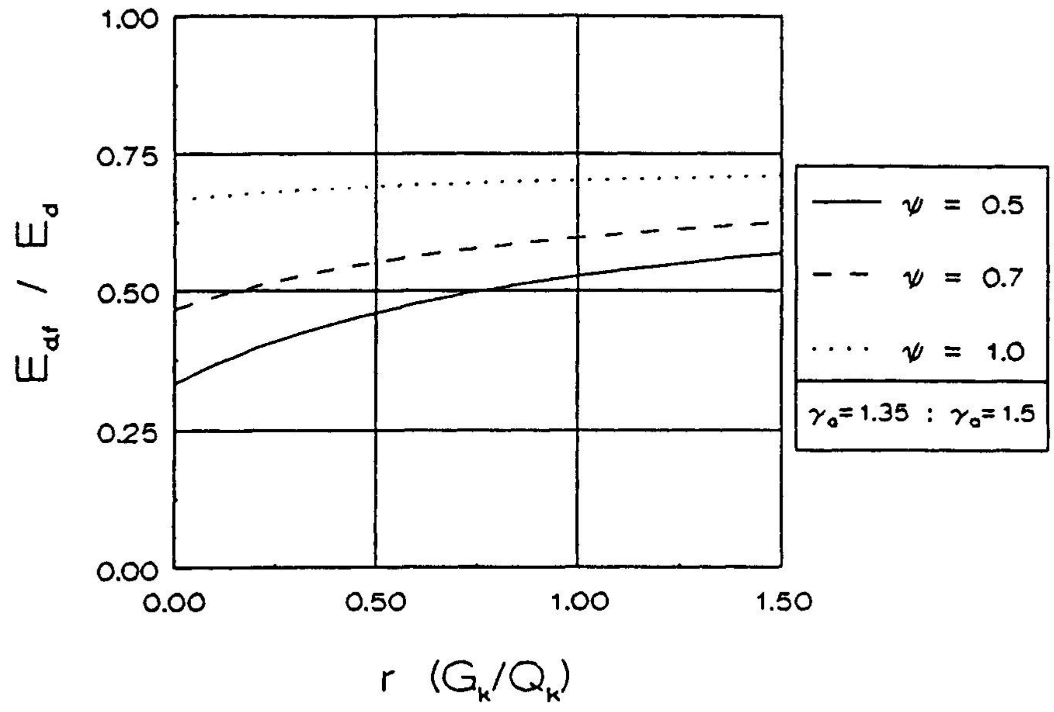


Fig. 5: Ratio between design effect of actions in case of fire and the design effect of actions at room temperature, as a function of the ratio between the permanent and main variable action, for given values of the partial safety factors (γ_G=1.35; γ_Q=1.5).

5. DEVELOPMENTS

As mentioned already under 1, work is in progress to evaluate and incorporate obtained comments on the 1990 draft of the fire part of the EC-Actions. Main modifications will be with respect to the thermal actions. Important items are:

* improved definition of the thermal actions, i.e. in terms of net heat flux to structural members, considering thermal radiation and convection from and to the fire compartment;
* specification of various sets of nominal time temperature curves i.e. (1) the (ISO) standard time curve, (2) a Hydrocarbon curve, (3) an external fire curve;
* specification of simple fire models for compartment fire exposure and external members, where appropriate in the form of design natural fire curves;
* reconsidering the suggested relationship between the action effects for room temperature design and for the fire situation; cf. discussion under 4.3.

More advanced fire modelling will be incorporated only in a later stage, i.e. not within two years from now.

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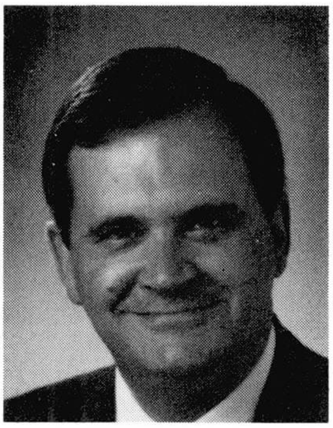
EC 1: Traffic Loads on Road Bridges

EC 1: Charges dues au trafic sur les ponts-routes

EC 1: Verkehrslastmodell für Strassenbrücken

Jean-Armand CALGARO
Professor
SETRA
Paris, France

Gerhard SEDLACEK
Prof. Dr.
RWTH Aachen
Aachen, Germany



Jean-Armand Calgaro, born in 1947, is Chief Engineer of Roads and Bridges at SETRA and Head of the Civil Eng. Dep. at the Ecole Nationale des Ponts et Chaussées of Paris. He has been a member of the Working Group for the preparation of the harmonized traffic load model for road bridges.

Gerhard Sedlacek born in 1939 is Professor for Steel Construction at the RWTH Aachen. He has been a member of the Working Group for the preparation of the harmonized traffic load model for road bridges.

SUMMARY

After describing the situation at the beginning of the work the procedure adopted for the development of a new European harmonized traffic load model for road bridges is presented. Some features of the first proposal for the model are given. As an example the actions due to this model are compared with those due to national bridge loading standards.

RESUME

Après un rappel de la situation lors du démarrage des travaux, la démarche adoptée pour l'élaboration d'un nouveau modèle européen harmonisé de charges dues au trafic sur les ponts-routes est présentée. Quelques caractéristiques de la première proposition de modèle sont détaillées. A titre d'exemple, les effets de ce modèle sont comparés à ceux de codes nationaux actuels.

ZUSAMMENFASSUNG

Nach Darlegung der Situation bei Beginn der Arbeiten wird auf die Vorgehensweise bei der Entwicklung des neuen Europäischen Verkehrslastmodells für Strassenbrücken eingegangen. Zu dem ersten bereits fertiggestellten Entwurf werden einige Angaben gemacht. An einem Beispiel werden die Schnittgrössen infolge des vorgeschlagenen Modells mit den Ergebnissen der derzeitigen nationalen Belastungsvorschriften verglichen.

1. INTRODUCTION

- In 1987 a working group has been formed by the Eurocode Steering committee chaired by the Commission to prepare a draft for a European loading code for actions on road bridges to be used together with the design rules for concrete, steel and composite structures as being worked out for the Eurocodes 2, 3 and 4.
- Whereas in the field of railway bridges the national railway authorities had already organized their international cooperation for harmonizing technical specifications in UIC and hence were in a position to present an agreed UIC loading model, such cooperation of the ministries concerned with road bridges did not yet exist. So it had to be established by inviting delegates from the ministries to join the working group and to give the necessary input to the works from the users side.
- Under the chairmanship of Mr Mathieu (SETRA, France) the works were planned and executed in the following way:

1. In 1987 a feasibility study [1] was carried out to clarify the state of the art and define the objectives, namely:
   - The load model should consist of a normal traffic load model calibrated on the effects of measured traffic data and a classified abnormal traffic load model that may be chosen in case exceptional vehicles not covered by the normal traffic load model have to be foreseen.
   - The normal load model should be composed of concentrated loads and uniformly distributed loads in such a way that it is both suitable for global verifications and local verifications of the bridge and of parts thereof in both the longitudinal and transverse direction of the bridge. That includes its applicability to a great variety of influence surfaces for action effects.
   - The normal load model should be suitable for ULS, SLS and Fatigue verifications and also allow for horizontal traffic effects and for accidental situations.
   - The load model should be defined with characteristic numerical values and include vehicle, pedestrian and crowding effects.
2. In 1988 subgroups were formed with experts that laid the ground for the further drafting works by performing prenormative research in those fields where according to the feasibility study so far no agreement could be achieved. The results of these works were documented in background reports, fig. 1.

- **Reference Bridges and Influence Lines** (report of Subgroup 1)
  (Calgaro, König/Sedlacek, Malakatas, Eggermont)
- **Traffic Data of the European Countries** (report of Subgroup 2)
  (König/Sedlacek, Bruls, Jacob, Page, Sanpaolesi)
- **Definition and Treatment of Abnormal Loads** (report of Subgroup 3)
  (DeBuck,Kanellaidis,Eggermont,Hayter,Merzenich)
- **Definition of Dynamic Impact Factors** (report of Subgroup 5)
  (Sanpaolesi, König/Sedlacek, Astudillo, Bruls, Jacob)
- **Other Action Components** (pedestrians, braking etc) (report of Subgroup 6)
  (Gilland, Pfohl, Mehue, O'Connor)
- **Guidelines for the Definition of Fatigue Loading** (report of Subgroup 7)
  (Bruls, König/Sedlacek, Jacob, Flint, Sanpaolesi)
- **Methods for the Prediction of Extreme Vehicular Loads and Load Effects on Bridges**
  (report of Subgroup 8) (Jacob, Flint, König/Sedlacek, Bruls)
- **Reliability Aspects** (report of Subgroup 9) (Flint, König/Sedlacek, Bruls, Jacob, Sanpaolesi)

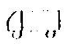
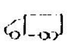
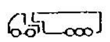
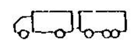
Figure 1 : List of reports from prenormative research

3. In 1991, when the works on the Eurocodes had been transferred from the Commission to CEN and the background works came to an end the drafting work on Part 3 (traffic loads on bridges) of Eurocode 1 (Basis of Design and Actions on Structures) started. In early 1992 the first completed draft was sent to the CEN-member organisations for comments and discussions were opened with other Project Teams that for other Eurocodes prepare the design rules for concrete, steel and composite bridges. These comments and discussions shall help to finalize the draft and to achieve an European loading model fitting to the requests of the bridge authorities, industry and consulting engineers.

- In the following some features of the first draft and some background informations are given.

2. GENERAL PROCEDURE FOR THE DETERMINATION OF THE TRAFFIC LOAD MODEL FOR ROAD BRIDGES

- From a comparison of the data from traffic measurements carried out at different bridge sites in Europe the Auxerre traffic (highway Paris - Lyon) recorded by the LCPC was chosen for defining the European traffic load model for road bridges for the following reasons:
  1. the data were obtained only recently and represent an homogeneous traffic sample (comprising 1st and 2nd lane data)
  2. the composition of the traffic was considered as representing already a future trend in the traffic development on other roads in view of the percentage of articulated heavy vehicles, the loading rates and the weights, fig. 2.

		LANE 1	LANE 2
Vehicle Flow	[veh/24 hrs]	8158	1664
Lorry Flow	[veh/24 hrs]	2650 (=32.3%)	153 (=9.2%)
Percentage of lorries in the lorry flow : (lorry flow = 100%)			
	[%]	22.7	27.6
	[%]	1.3	3.5
	[%]	65.2	58.4
	[%]	10.8	10.5

GROSS WEIGHT OF ALL LORRIES OF LANE 1

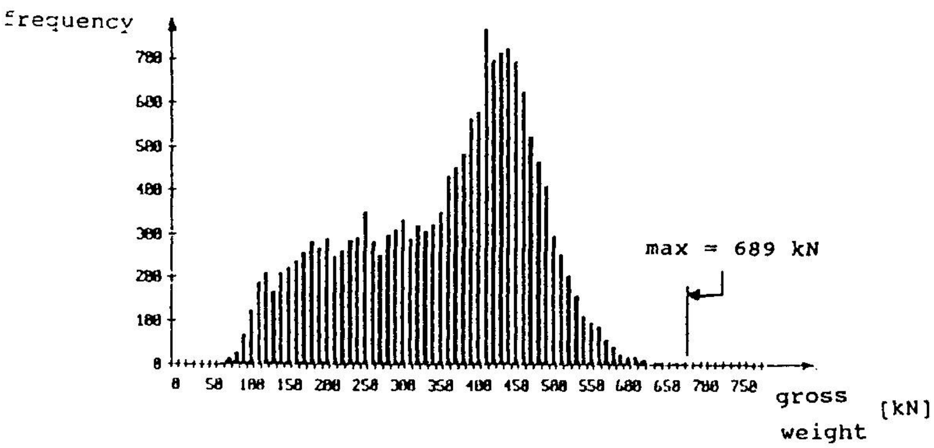


Figure 2 : Characteristics of the Auxerre traffic

- In order to evaluate the traffic data and to simulate the dynamic traffic effects on bridges numerically simulation programmes for timestep analysis have been developed, that allow to take account of
  1. the dynamic behavior of the bridges (FE modelling of stiffness, mass distribution and damping)
  2. the dynamic characteristics of the vehicles, fig. 3, taking account of the non linear hysteretical behavior of the suspensions including friction effects.

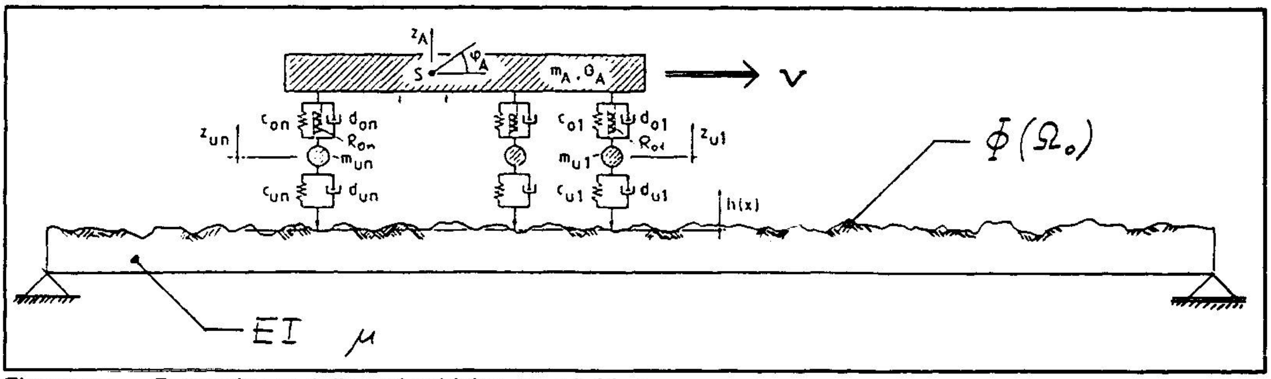


Figure 3 : Dynamic modeling of vehicles on a bridge

  3. the roadway roughness on the bridge taking account of the definition $\Phi(\Omega_0)$ in ISO-TC 208 or discrete irregulations e.g. at the transition joints.
  4. the different speeds of the vehicles.

These dynamic simulation programmes [2], [3], [4], [5] were calibrated with dynamic bridge measurements undertaken at the EMPA Dübendorf [6], [7], fig. 4 and proved to be rather accurate.

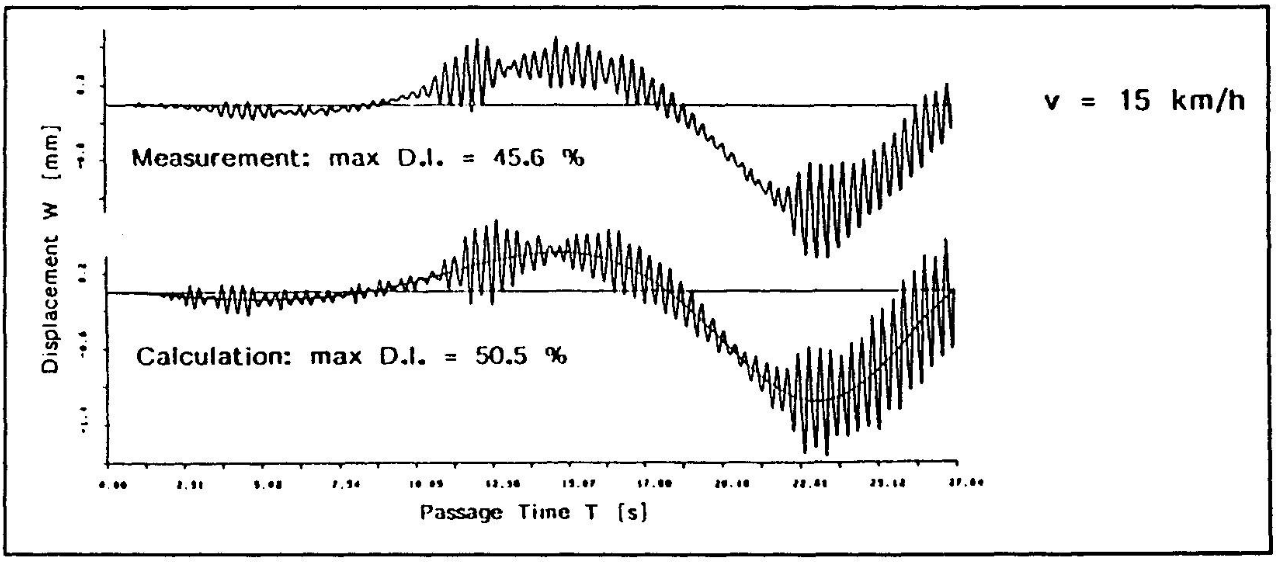


Figure 4 : Comparison of measured and calculated bridge responses

- In a first step the simulation programmes were used for filtering the dynamic effects from the statistical distributions of the Auxerre traffic data and to obtain purely statical data, fig. 5, from which random vehicles sequences could be formed by Monte Carlo methods in order to take account of the variability of the traffic effects.
- In a second step loading scenarios for flowing (at different speeds) and jammed (slowly moving) traffic situations for a great variety of simply supported and continuous bridges with different systems (open cross section and box girders), different widths (number of lanes) and different span length were identified for which the dynamic simulation programme served to calculate the maximum action effects (e.g. bending

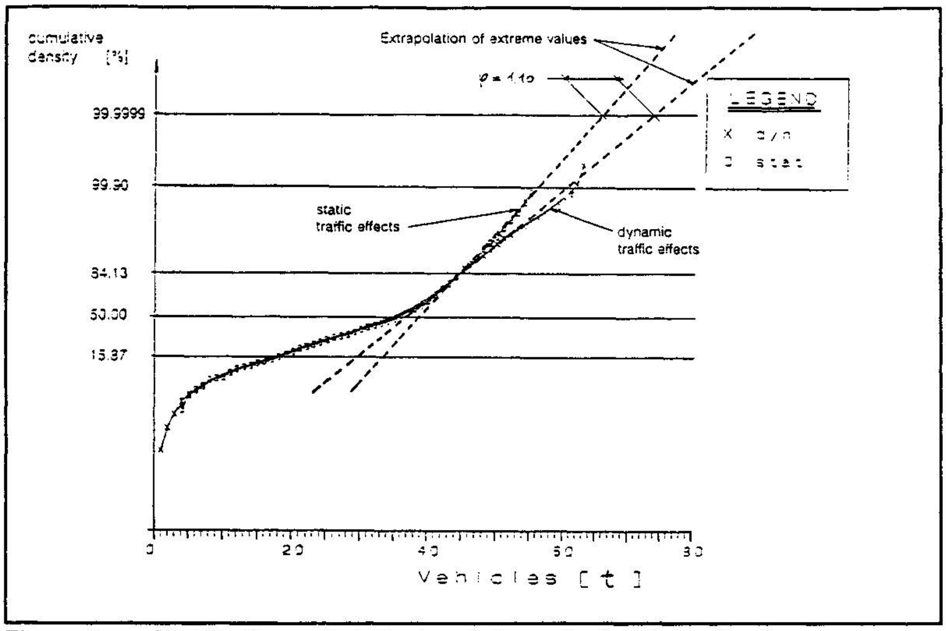


Figure 5 : Distributions of statical and dynamic data from the Auxerre measurements

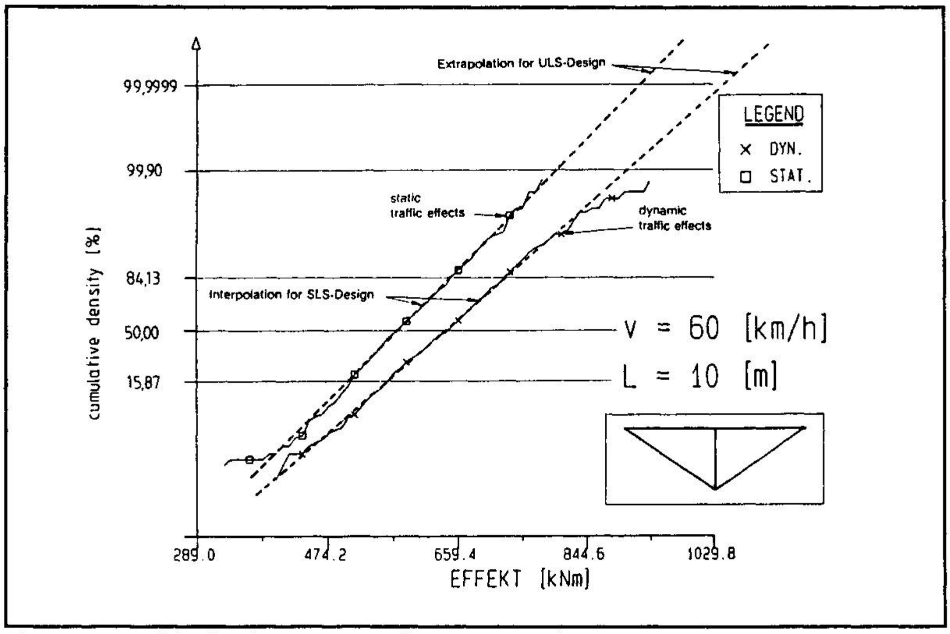


Figure 6 : Distribution curves of maximum effects

moments at midspan or at the supports or shear forces) in order to establish distribution curves of these maximum effects, fig. 6.

- The maximum dynamic action effects representing the 95 % fractiles for a reference period of 50 years were considered as the characteristic target values the traffic load model should cover, fig. 7. From comparing the characteristic values of the dynamic action effects with those yielding from distribution curves for statical action effects only (where the dynamic influence of the vehicles and the bridge were neglected), fig.6, "effective impact factors" were defined that could be applied to target values determined by simple statical calculations only without using the dynamic simulation programmes.

- Several proposals for loading scenarios were developed in order to obtain target values for 1-4 loaded lanes. A synthesis of these proposals permitted to define a set of harmonized target values on the basis of which the main model was calibrated.

- In the last step the traffic load model was synthesized by optimizing its free parameters for the geometrical load pattern and the numerical values such, that for given weighting factors for the types of influence areas, number of lanes and span lengths a minimum of the square of the deviation of the characteristic target values from the effects of the load model was achieved, fig. 7.

- The works in each step were carried out redundantly by independent groups to avoid systematic and incident mistakes and ensure reliable results.

3. SOME FEATURES OF THE FIRST DRAFT OF THE TRAFFIC LOAD MODEL ON ROAD BRIDGES

- The normal traffic load model is given in fig. 8. It consists of tandem axle loads in lane 1, 2 and 3, superimposed to uniformly distributed loads. The contact area of the tyres was chosen in accordance with the actual trends for the conception of lorries and tyres [8].

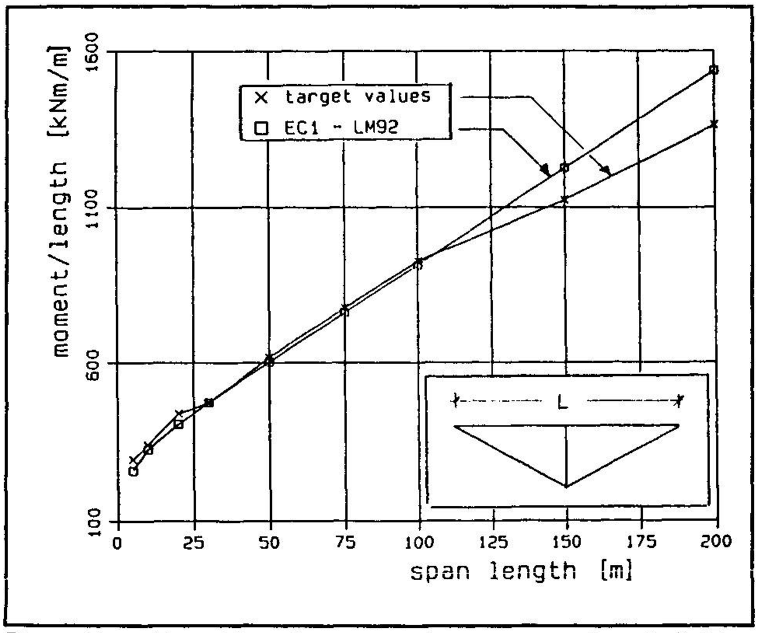


Figure 7 : Target bending moments versus span length (bridge width corresponds to 4 loaded lanes)

- Fatigue verifications may be carried out on different levels. A detailed method is envisaged on the basis of a set of realistic fatigue vehicles according to fig. 9. A conservative approach is also envisaged by using a unique fatigue equivalent vehicle according to fig. 10.

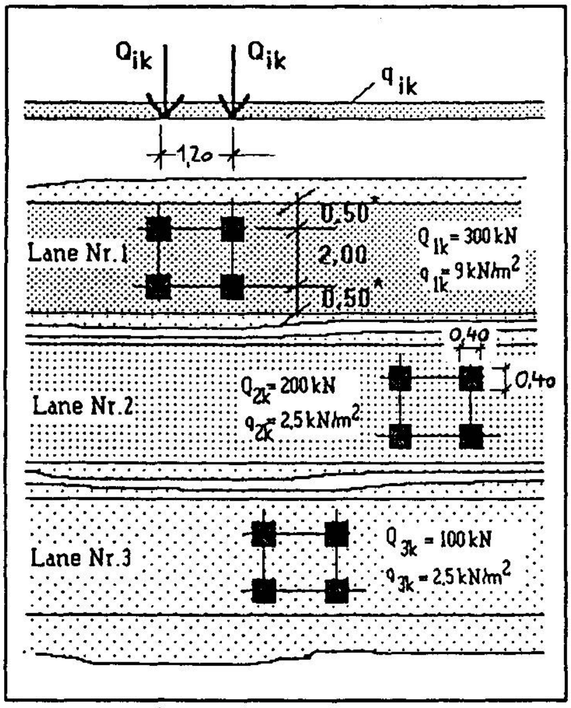


Figure 8 : Normal traffic load model

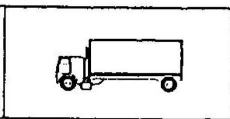
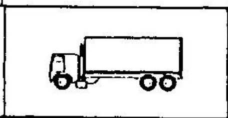
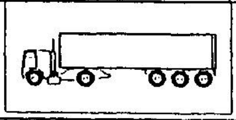
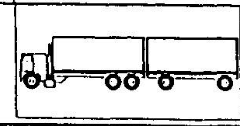
VEHICLE Silhouette	AXLE SPACING (m)	AXLE LOAD (kN)	Vehicle percentage
	4,50	75 120	25%
	4,20 1,30	70 90 90	2%
	3,20 5,20 1,30 1,30	70 120 85 85 70	37%
	3,20 5,50 1,30	70 120 90 90	23%
	4,20 1,30 3,50 4,50	70 90 90 100 100	13%

Figure 9 : Set of fatigue vehicles

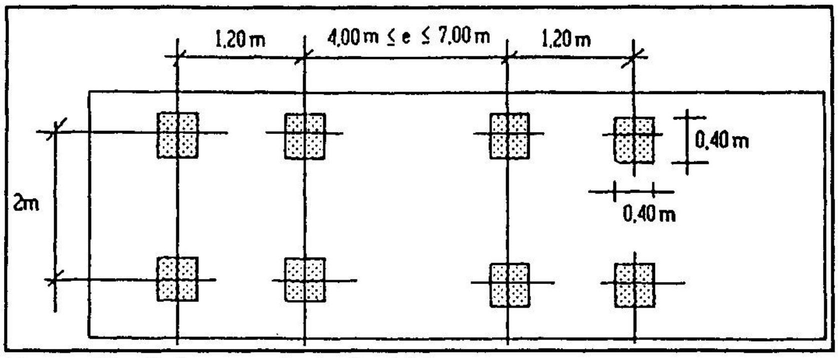


Figure 10 : Fatigue equivalent vehicle

4. COMPARISON OF THE PROPOSED TRAFFIC LOAD MODEL WITH NATIONAL CODES

- The load scaling factors $\lambda_p = \frac{M_{Nat.Code}}{M_{EC\ Load}}$ representing the ratio between the bending moments due to national bridge loading codes and to the Eurocode loading model as presently proposed for the example of the midspan moments of a continuous bridge may be taken from fig. 11.
- The differences are enormous due to the fact that the national design rules for the bridges differ as well. A better comparison will be possible as soon as both the action and the resistance side will be harmonized and the design results (e.g. in terms of quantities built in) can be compared.

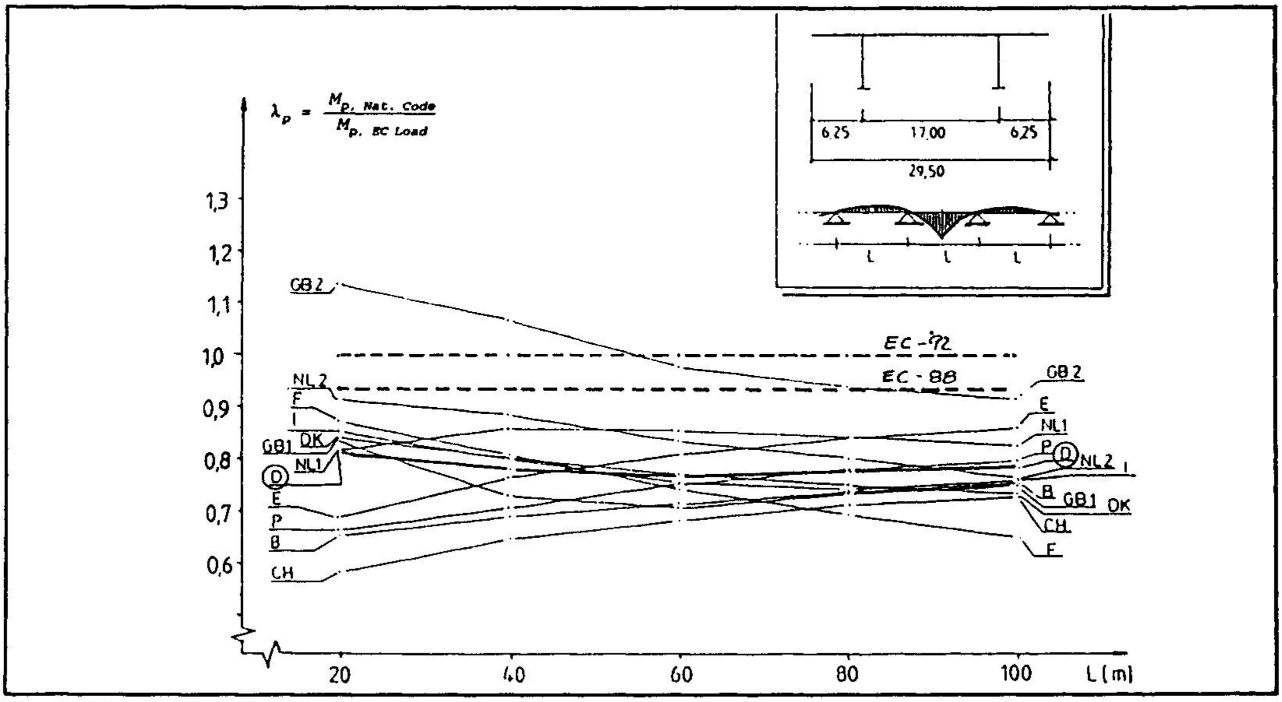


Figure 11 : Load scaling factor λ_p

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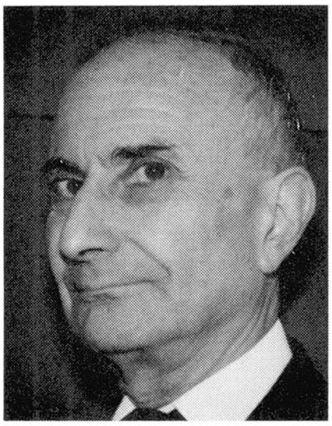
EC 1: Traffic Loads on Bridges – Rail Traffic Loads

EC 1: Charges dues au trafic sur les ponts-rails

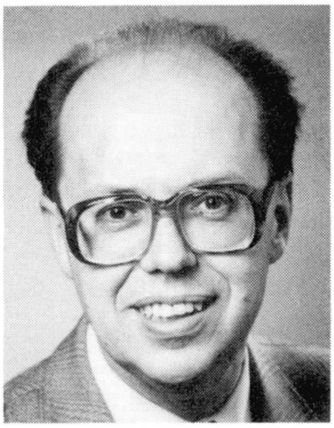
EC 1: Verkehrslasten auf Brücken – Bahnverkehrslasten

J.E. SPINDEL
Dr.
(formerly of British Railways Board)
London, UK

Marcel A. TSCHUMI
Head of Bridge Section
Swiss Federal Railways
Berne, Switzerland



Dr. Spindel, born 1925, graduated from Imperial College, University of London, 1945; held various appointments in bridge and works design with British Rail; was involved in the development of standard railway loading as a member and, later, chairman of the Bridge Sub-Committee of the International Union of Railways up to January, 1988.



Marcel A. Tschumi, born 1938, obtained his civil engineering diploma at the Swiss Federal Institute of Technology in Zurich in 1962. Since then he has been involved in the design of structures and bridges. Since February 1988 he is chairman of the Bridge Sub-Committee of the International Union of Railways (UIC).

SUMMARY

This section of volume 3 of Eurocode 1, provides the necessary information on the actions of railway traffic which bridges have to be designed to resist. The paper outlines the range of real traffic which has to be considered, and the actions associated with it. This is related to the simplified equivalent loads and load spectra to be used for design.

RESUME

Ce chapitre du volume 3 de l'Eurocode 1 donne des informations en ce qui concerne les actions dues au trafic ferroviaire, pour lesquelles les ponts doivent être dimensionnés. Le document donne un aperçu de l'étendue du trafic réel à prendre en considération et des actions qui en découlent. Celles-ci sont mises en relation avec les charges simplifiées équivalentes et les spectres de charges qui sont utilisés pour le dimensionnement.

ZUSAMMENFASSUNG

Dieses Kapitel von Band 3 des Eurocodes 1 liefert Informationen zu den Einwirkungen des Bahnverkehrs, mit welchen die Brücken rechnerisch nachgewiesen werden müssen. Dieser Beitrag gibt einen Überblick über die Vielfalt des wirklichen Verkehrs, der berücksichtigt werden muss, sowie die damit verbundenen Einwirkungen. Diese werden in Bezug gebracht zu den vereinfachten äquivalenten Lasten und Lastspektren, die für die Bemessung verwendet werden.

1. INTRODUCTION

The loading intended to represent rail traffic loads in Eurocode 1 bears little obvious resemblance to real trains. The various forces it imposes on bridges appear to be quite independent. It, therefore, seems wrong to consider most of them as part of one action.

The purpose of this paper is to outline the facts on which rail traffic loads are based with a view to explaining these matters. The first part considers what a given train does to a given bridge. The second part considers the general case of traffic on a population of bridges.

2. THE ACTION OF A GIVEN TRAIN ON A GIVEN BRIDGE

2.1 Vertical Forces

2.1.1 Static Load

A train is a mass, supported at discrete points (the axles), which is subject to various accelerations. The action of these on the part of this mass which is on a bridge causes the actions of the train on the bridge. This mass, therefore, links all of them either directly or indirectly.

The primary acceleration is that of gravity. This always acts, is constant, and causes the static load.

2.1.2 Dynamic increment

This, directly associated with the movement of the train, is due to the deformation and vibration of the bridge crossed by the train at speed. It can be considered as the total of three components:
- a dynamic amplification of the static deflection of the structure (forced vibration)
- a damped free vibration at the natural frequency of the structure
- random vibrations due to the movement of the unsprung mass of the train caused by track and wheel irregularities.

For a simply supported beam, the first two of these can, as a close approximation, be derived by replacing the train, with its complex system of sprung and unsprung masses on an elastically supported track, by the forces it exerts under gravity. The result is shown in Fig. 1 as the ratio of "dynamic" to static deflection at mid-span for a single force crossing the beam.

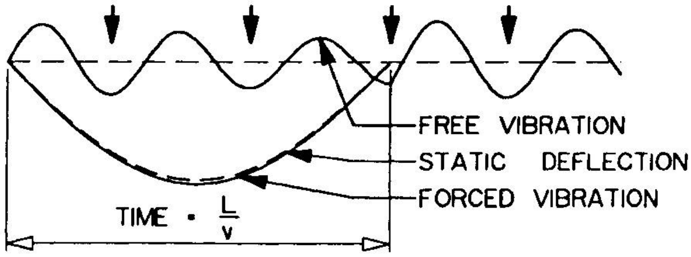


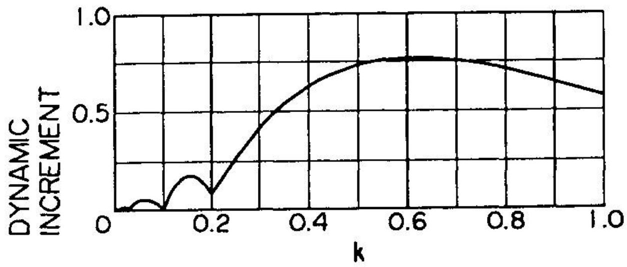
Fig. 1

Mid-span deflection of a simply supported beam under the passage of a single force (k=0.182)

It will be noted that the free vibration is such as to produce zero velocity when the force comes on the beam. (The velocity is proportional to the slope of the lines). It also shows that the maxima do not coincide with the maximum of the amplified static deflection. Whether this coincidence occurs or not depends on a parameter k, as does the amplitude of the free vibration. ($k = v/2Ln_o$, where v is the speed in m/sec, L the span in m and n_o the natural frequency of the unloaded bridge). As shown in Fig. 2, the total dynamic effect does not increase steadily with speed at low values of k.

Fig. 2

Dynamic increment for a single force as a function of k



The lines on Fig. 1 are influence lines from which the effect of a sequence of forces can be determined. It follows from this that a series of forces at the centres shown by arrows will cause the greatest dynamic effect on the bridge, while forces at half that spacing will produce practically none, as would any reasonable approximation to a uniformly distributed load.

Real cases lie between these limits. This, together with the effect in Fig. 2, accounts for the wide scatter in dynamic effects for the same value of k. A limited statistical analysis of a large number of test results for steel and concrete bridges produced standard deviations of the order of 70% and 55% of the mean for steel and concrete bridges respectively.

However, for a given train crossing a given bridge at a given speed the dynamic effect is determinate. This was confirmed by model tests and on a bridge under normal traffic. All fast passenger trains caused the same dynamic effect which was near the maximum. Even the apparently random effect of a track irregularity was reproduced.

2.2 Horizontal Forces

2.2.1 Lateral Forces

The easiest of these is the so-called centrifugal force. Given the mass of the train and the radius of the curve, it is as determinate as the speed of a train running to a given timetable. It is simply the mass multiplied by v^2/R, where v is the speed in m/sec and R the radius of the curve in m.

If the speed which the train can reach is at least the greatest allowed through the curve, the greatest horizontal force it can cause is the vertical force multiplied by $(c+d)/s$, where c and d are cant and cant deficiency allowed and s is the distance between centre-lines of rails, all in consistent units. On most railways this ratio has a value of about 0.2.

Another lateral force is that due to the lateral oscillation of vehicles. A value of 100 kN was deduced for this force on the bridge from measurements of rail seat forces. Research on this subject is in progress. As a very rough approximation, this force is also a measure of the dynamic effects due to centrifugal forces. It is, therefore, considered as combined with them though centrifugal forces tend to suppress lateral oscillations of vehicles.

2.2.2 Longitudinal Forces

Traction and braking, always mentioned together, differ so much that they require separate consideration and deserve different treatment.

Both act along the axis of the bridge, in opposite directions, and are limited by the adhesion (friction) between wheel and rail. Given clean wheels and rails, this can reach 42% (of the applied vertical force).

Traction is the force needed to accelerate a train from rest or after a speed restriction, and to keep it moving at speed and up gradients. Modern locomotives have devices to control wheel slip, thus ensuring that maximum adhesion is attained.

Traction acts continously, often at its maximum, and is proportional to the vertical load on the driving axles.

Since the heaviest trains need the greatest traction, design rules assume that traction is due to two heavy locomotives. This means that the force on the bridge remains constant once the loaded length exceeds about 29m.

Braking forces depend on adhesion, passenger comfort and, critically, on the type of brake used.

Comfort limits service braking to a deceleration of about 0.1g. Disc brakes, used mainly on modern passenger coaches, can, at most, increase this to about 0.13g. Brakes with cast iron brake shoes, used mainly on freight trains and on locomotives, will produce the same order of deceleration up to ½ second, or so, before the train stops. In that ½ second the deceleration will rise to the maximum limited by adhesion, 0.42g. It is this peak value which makes braking forces the most contentious issue in railway loading.

Braking deceleration acts on the mass of the whole train. When it reaches its peak the train has nearly stopped. At that time the only dynamic effect is the transfer of load from the rear bogies of the vehicles to the front ones. This was found to entail an increase in bogie load of some 30%.

Given the short duration of the peak braking force and that it is only likely to occur after emergency braking, there is an argument for treating it as an accidental load. It certainly justifies special treatment in the design of abutments and foundations.

Longitudinal forces tend to move the bridge as a whole in the direction in which they act. This movement is limited by the stiffness of the abutments and piers to which fixed bearings are attached. Unless the track across the bridge is isolated by expansion switches, some of this movement will be resisted by the track beyond the bridge.

This restraint can transfer some 30% to 60% of the longitudinal force to the track. The increase in force in the rails which can be tolerated, however, is limited by considerations of the stability of the track.

The proportion of load transferred depends on the relative stiffness of bridge and track.

The stiffness of the bridge can be taken as linear, but it includes the stiffness of the foundations under long and short term loading. Estimates of such stiffness are notoriously inaccurate. The stiffness of the track varies with its type and condition. It is non-linear because, after elastic movement of 1 or 2 mm, there is progressive slip between rail and sleeper and sleeper and ballast.

The difficulties in all this calculation are not so much those of computation but the accuracy, or lack thereof, of the basic data.

3. THE ACTION OF RAIL TRAFFIC ON BRIDGES

3.1 Railway traffic

The derivation of actions outlined in section 2 above can be applied only to bridges which carry only one kind of train at one speed. Examples are metro systems (without service or ballast trains) and railways built for one traffic, say from a mine to a port.

Most bridges are built to carry a mixture of traffic which is likely to change during their life of some 100 years.

The trains they will, or may, have to carry can be grouped as passenger and freight trains. All the latter are locomotive hauled. Table 1 shows their speeds, axle loads and average weights per metre, all as ranges of values commonly encountered or planned.

Type of train	Speeds km/h	Axle loads kN	Average weight kN/m
Passenger trains:			
. suburban multiple units	100 - 160	130 - 196	20 - 30
. locomotive hauled trains	140 - 225	150 - 215	15 - 25
. high speed trains	250 - 350	170 - 195	19 - 20
Freight trains:			
. heavy abnormal load	50 - 80	200 - 225	100 - 150
. heavy freight	80 - 100	225 - 250	45 - 80
. trains for track maintenance	50 - 100	200 - 225	30 - 70
. fast, light freight	100 - 160	180 - 225	30 - 80

Table 1 Trains

In relation to the above table it should be noted that:
- the average weight of locomotives ranges from 50 to 70 kN/m
- the length of the vehicles classed as heavy abnormal loads ranges from 15 to 60m; they mainly affect the support moments of continuously supported bridges and simply supported medium span bridges,

Where what trains run depends on any physical restrictions on a line (curves, gradients, weak existing bridges) and on commercial and operating requirements. All these are known and planned at any given time, but may, and probably will, change in the course of time. At present, for example, heavy abnormal loads are not allowed on a number of lines, including most suburban and high speed passenger lines.

High speed passenger lines, however, do also carry all kinds of freight on one railway, fast light freight only on another, and very high speed passenger traffic only on a third. This is the result of policy - not any physical limitations.

This is confirmed by the fact that all lines carry trains with machines and materials for track maintenance.

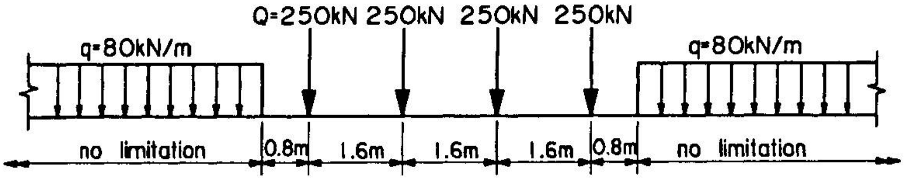
It is, therefore, reasonable to build new bridges so that they are capable of carrying any of the present and anticipated traffic, or at least that which is not highly likely to remain subject to restrictions.

3.2 Static load due to railway traffic

It is not difficult to produce a loading which will cover the greatest static actions of all known and planned trains on simply supported bridges, particularly if heavy abnormal loads are treated as a separate case. This is what the UIC loading shown in Fig. 3 does.

Fig. 3

UIC loading 71



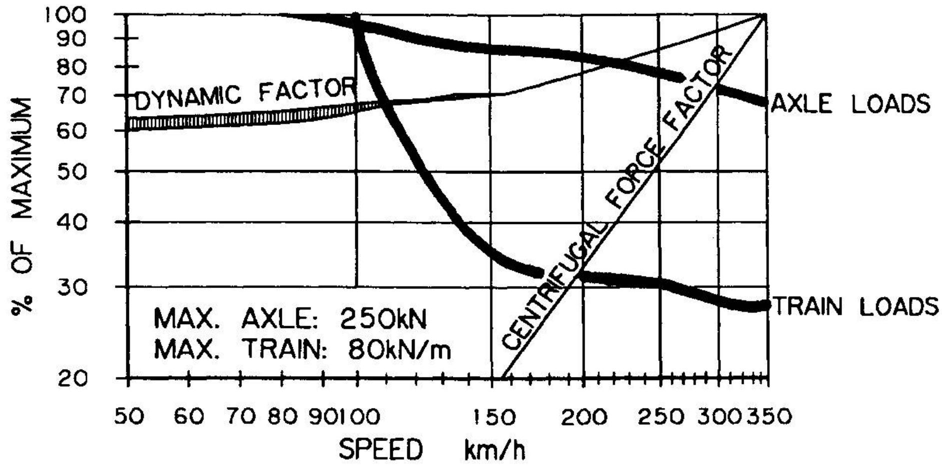
It was, of course, based on a much more detailed and extensive investigation than that outlined above.

3.3 Dynamic effects and centrifugal forces due to railway traffic

When it comes to dynamic effects and centrifugal forces, however, account must be taken of the greatest speeds of the various trains as well as their weight. It is a characteristic of railway, and road, traffic that heavy vehicles are slow and fast vehicles light. This is apparent from Table 1 and is shown, in broad outline, on Fig. 4 for "normal " trains.

Fig. 4

Changes in loads, dynamic effects and centrifugal forces with speed



The two lines from left to right show how axle and average train loads drop from their maximum values as speed increases. Values for bending moments on various spans will lie between these lines.

The curve from right to left shows how the dynamic factor, by which the static load has to be multiplied to arrive at the total vertical action, drops as the speed decreases from a maximum of 350 km/h.

Since the figure is plotted on a logarithmic scale, the required product is the sum of the ordinates of this line and those from one of the lines from left to right. For axle loads this product remains roughly constant. For train loads it drops considerably at very high speeds.

To arrive at a reasonable allowance for dynamic effects the calculations outlined in 2.1.2 above were repeated for a selection of trains and vehicles over a range of speeds to obtain an upper limit for the dynamic effects due to "all" trains as a function of the parameter k. By using this function with upper and lower bounds for the natural frequency of bridges of a given span, the total action of various trains on a given simply supported span was obtained. The ratio of the envelope of these total actions to those due to the static loading shown in Fig. 3 produced the apparently simplistic formulae for dynamic factors as a function of span, L, only, such as

$$\Phi = \frac{1.44}{\sqrt{L}-0,2} + 0.82$$

Similar considerations apply to formulating rules for centrifugal forces. The straight line from right to left in Fig. 4 relates to the v^2 term by which the mass of the train has to be multiplied. It will be noted that this line drops more steeply with decreasing speed than the mass of the train or axle rises.

The line has been drawn on the assumption that the radius of curvature is such as to allow a speed of 350 km/h. If it were such as to limit the speed to say 150 km/h, a parallel line would have to be drawn through the point where the 150 km/h ordinate intersects the 100% line. Again, the v^2 term decreases more steeply than the mass term rises.

It follows from this that, at least at speeds above 120 km/h, it is the fastest train which causes the greatest horizontal force.

It does not follow, however, that this force, combined with the reduced vertical load of the lighter train, produces the greatest load on the bridge as a whole or on one of its elements. It is, therefore, necessary to check that a slower, or even stationary, train does not produce a worse effect.

Again, an extensive and detailed investigation on the lines described above produced the rules given in the draft for EC1.

4. FATIGUE EFFECTS CAUSED BY RAIL TRAFFIC

4.1 General considerations

Fatigue failure is the result of the accumulation of the fatigue damage caused by large numbers of individual stress ranges of varying magnitude applied to an element of a structure.

Consequently rail traffic needs to be defined in terms of all the stress changes it causes in a structure and not just as its greatest effect.

This means consideration of axle spacing as well as load. The stress ranges which the sequence of axles produces are critically sensitive to the type and length of influence line for the part of the structure. Influence lines for bending moment and shear, for example, produce very different results.

In planning these calculations it must be remembered that a summation is involved.

This has the consequence that random variations in loads do not significantly affect the sum. Mathematically, it can be shown that the only effect of a random variation of a load about a mean value, assuming the variation to be log normally distributed, is an increase in the Palmgren-Miner sum by a factor of $[\exp(\frac{1}{2}m^2s^2)]$ where m is the slope of the S/N line and s the standard deviation of the ln of the load. For a coefficient of variation of 10% and a slope of 5, the increase is 12% in the sum, which corresponds to a reduction of 2.3% in permissible stress.

Another consequence is that any number of suitably selected trains can cause identical fatigue damage - a point of some importance when considering traffic rather than individual trains.

4.2 Load spectra for design

The stress changes for a given train can be collected and expressed as a load spectrum. Such spectra can be combined to give a spectrum for a traffic.

Traffic can vary in composition - various mixtures of passenger and freight trains - and in volume expressed as gross tonnes per annum, t/an., usually in millions.

Traffic may be all one train, for example on a suburban line, or a mixture of practically every kind of train. For the purpose of fatigue calculations occasional heavy trains can be neglected. For a given type of influence line complex real traffic can be, and is, represented by a carefully selected mixture of a few "typical" trains even if they do not look very realistic.

Volume of traffic may be as low as $0.5 \cdot 10^6$ t/an. for a branch line to a factory or quarry, and rise to $20 \cdot 10^6$ or $30 \cdot 10^6$ t/an. on a busy main line or, surprisingly, for a light railway. The most that has been claimed is $63 \cdot 10^6$ t/an.

In these circumstances design rules have to be flexible to allow type and volume of traffic to be varied to suit traffic on a given line. Making this choice does imply a prediction of traffic for the 50 or 100 year life of the bridge. Overestimating costs money in building bridges; an unforeseen great increase in traffic can probably pay for the earlier replacement of bridges.

The load spectra produced on the basis of the considerations outlined above can then be used for fatigue calculations as required, for example, in chapter 9 of EC3.

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EC 1: Silos and Tanks

EC 1: Silos et réservoirs

EC 1: Silos und Behälter

Jørgen NIELSEN
Dr. Eng.
Danish Build. Res. Inst.
Hørsholm, Denmark

Jørgen Nielsen, born 1944, is a graduate of the Technical University of Denmark. For about fifteen years he undertook research (mainly on silos) and teaching at the same university. For several years now he has worked at the Danish Building Research Institute.

J. EIBL
Prof. Dr.
Univ. Karlsruhe
Karlsruhe, Germany

J. Eibl, born 1936, civil eng. TU München; Assoc. Professor TU Braunschweig; 1974 Professor University Dortmund; since 1982 Professor and Director of the Institute of Concrete Structures and Building Materials, University of Karlsruhe.

Michael ROTTER
Prof. Dr.
Univ. of Edinburgh
Edinburgh, Scotland

Michael Rotter was born 1948 and graduated from Cambridge University. He spent eighteen years at the University of Sydney in Australia. He took up his present post in 1989. He has authored over a hundred technical papers, and undertaken many investigations into silo designs and failures.

SUMMARY

The paper briefly describes the background to the preparation to the part of the Eurocode on Actions in Silos and Tanks. The work of the ISO/TC98/SC3/WG5 formed the starting point for the Eurocode. The paper summarizes the background for the development of the code, its limitations, and progress to date. The plans for completing the work are also described.

RESUME

L'article traite brièvement des bases pour la préparation de la partie de l'Eurocode 1 traitant des actions sur les silos et réservoirs. Le travail du comité ISO/TC98/SC3/WG5 est à la base de cet Eurocode. L'article résume les éléments essentiels ayant servi à l'établisement de cette norme, présente ses limites, ainsi que les progrès réalisés à ce jour. L'évolution, nécessaire à l'établissement définitif de ce projet est également présenté.

ZUSAMMENFASSUNG

Der Beitrag beschreibt den Hintergrund der vorbereitenden Arbeiten am Teil des EC 1 über Einwirkungen auf Silos und Behälter. Ausgangspunkt waren die Ergebnisse der Arbeitsgruppe 5 vom Subcommittee 3 des ISO-Kommittees 98. Es wird einen Überblick über die Entwicklung der Norm, ihre Beschränkungen, den Arbeitsfortschritt und geplanten Abschluss geben.

1. INTRODUCTION

1.1 Scope

The part of Eurocode 1 which deals with actions in silos and tanks will contain recommended methods for determining the actions that arise from the storage of bulk materials and liquids. Reference is also given to actions from gravity, snow, wind, earthquake, temperature and differential settlements as well as accidental actions from fire and explosions.

The calculated actions are for use in designing silos and tanks and their components. However, the field of application is subject to a series of limitations, which means that the structures to be covered can be characterized by:

- Silos with a limited eccentricity of inlet and outlet, with small inertia effects (impact) associated with filling, and with discharge devices that do not cause shock or eccentricities beyond the prescribed limitations.
- Silos containing particulate materials which are free-flowing and have a low cohesion.
- Tanks with liquids stored at normal atmospheric pressure.

1.2 Background

The work was initiated by the Commission of the European Communities. The first step was to collect information on codes and recommendations for loads in silos and tanks. The study was mainly concentrated on EEC member states and reported in 1987 [1]. It was concluded that the work initiated by ISO in ISO/TC98/SC3/WG5 "Loads from Bulk Materials" should be used. When the Eurocode work was transferred to CEN the ISO group had almost finished its work and a draft from June 1990 was accepted as the starting point for the CEN work [2].

Following the CEN procedure a Project Team, PT8, was formed. PT8 has experts from Denmark, France, Germany, Great Britain, and Greece.

The June 1990 draft [2] was circulated for informal comments from national contacts and from national standards organizations. The comments have been evaluated by the Project Team and several changes have been introduced. At the same time an attempt to transform the document to the Eurocode format has been made. The latest draft [3] dated February 1992 thus deviates considerably from the previous draft, and it has been circulated for informal comments during Spring 1992.

2. PARTICULATE MATERIALS AND STORAGE LOADS IN GENERAL

2.1 Material behaviour

To understand the distribution of loads in silos it is important to be aware of a few facts about the behaviour of particulate materials: If the material is compacted by a vertical compressive stress, σ, the response will be a lateral pressure, $\lambda\sigma$, where λ is less than 1 (Fig. 1a). Further, if you slide a portion of the particulate material along the wall, friction stresses will act against the movement controlled by a coefficient of friction, μ (Fig. 1b).

2.2 Storage loads

The following behaviour occurs during filling of the silo: At a given point, the vertical stress increases when more material is placed at the top surface. The vertical stress evidently depends on the bulk weight density, γ. As the vertical stress increases ,both the horizontal stress and the friction against the wall also rise, as explained above. If the silo is tall, the situation may develop in which the wall friction stresses between two horizontal sections almost counterbalances the weight of the material between the sections. When this happens, almost none of the weight of the material between these sections is borne by the material beneath it, so the vertical stress does not continue to rise when further filling occurs. Thus, the vertical stress, and the consequent horizontal pressure against the wall, approach asymptotic values with depth (Fig. 2a). This pressure pattern is very different from the well known hydrostatic pressure distribution found in tanks (Fig. 2b).

The main differences between the two distributions may be evaluated by assuming that two identical containers are filled with materials of equal density: one is filled with a particulate material and the other with a liquid. The pressure distributions will be as seen in Fig. 2a and Fig. 2b. At the top the pressure increases faster with depth in the tank since the pressure in the silo is multiplied by λ, which is smaller than one - typically about 0.5. Lower in the silo, the friction forces make the silo load even smaller compared to the hydrostatic pressure, making the silo load much more favourable for the structure than the hydrostatic load.

Unfortunately, for silos the picture is not quite so simple. A particulate material under compression acquires a certain strength, which means that several pressure distributions are possible depending on different circumstances.

The most important event is the gravity discharge of the particulate solid (Fig. 3). During discharge the particulate material must reach plastic stress states to be able to deform sufficiently to move through the outlet. The pressure near the outlet tends to zero, but equilibrium of the entire mass must be maintained, so pressures tend to increase elsewhere. As a result, a very different pressure distribution may be found during discharge. This change is particularly important in eccentrically discharging silos, where the pressure distribution may become very unfavorable for the structure.

Even with central discharge the pressure distribution may not be symmetrical because the rupture pattern in the solid may develop slightly unsymmetrically or because eccentric filling or irregularities in the wall may cause an asymmetrical distribution. Some of these factors may be controlled by the designer, but others cannot, which means that a symmetrical pressure distribution cannot be expected in practice.

It is important to stress that in silos containing strong particulate materials (small λ's), the magnitude of pressure changes from filling to discharge may be bigger than for weaker materials. A liquid may be seen as an extremely weak material; zero shear strength and thus no deviation from the hydrostatic (filling) pressure distribution.

2.3 Load models

The ISO Committee decided to base the load model on the classical Janssen theory being prescribed for filling. The deviations during discharge are taken into account by empirical factors. It has been questioned if it would not be better to base the code on one of the many newer theories that have been developed. The arguments for retaining Janssens theory for filling are mainly the following:
- it is simple
- it can be used for a wide range of shapes of silo cross-section
- it has been found to be fairly accurate for filling

The arguments to cover discharge and special cases by empirical parameters have been:
- it is simple
- sufficient consistency has not been demonstrated between experimental observations and theoretical predictions to justify a more complicated load specifications, though it is recognised

that such calculations may be useful in certain cases.

2.4 Material parameters

The ISO Committee decided to introduce an interval in the specification of values for the material parameters reflecting the scatter found in practice due to differences in production processes for the materials to be stored, different treatment of the interior surface of the silo, ageing, polishing, etc.

The literature shows a very wide range of values for the same parameters reflecting not only the scatter described above but also reflecting the fact that these parameters are not true material constants, and that they have been determined under a wide variety of different test conditions.

Parameters like γ, λ, and μ are also determined in the fields of soil mechanics and powder technology. It has been natural to adopt the test methods from these areas, but to specify special test conditions which are intended to produce values which are relevant for loads in silos.

One of the questions has been the determination of λ (which is used for filling) from the internal angle of friction, φ, knowing that in principle λ depends on the wall friction. For pressures against a stiff wall two equations for λ are widely accepted to be fairly accurate. One, $\lambda = 1\text{-}\sin\varphi$, is approximately valid for a completely smooth wall. The other, $\lambda = (1\text{-}\sin\varphi^2)/(1+\sin\varphi^2)$, is valid for a completely rough wall. The ratio between the λ's calculated from these formulas is about 1.2 for the range of φ which covers most stored materials. This means that the wall friction angle only influences the value of λ by about 20 per cent. A value of $\lambda = 1.1(1\text{-}\sin\varphi)$ has been introduced as a simple and fairly accurate rule.

Another discussion concerns the relation between the parameters λ or φ and μ. If the wall is rough, sliding down the wall takes place in the form of internal rupture in the particulate material, mainly controlled by φ, so that the parameters μ and φ are strongly correlated. In the case of a smooth wall, sliding does not mobilize the strength of the material and the parameters are independent. For simplicity the parameters are taken to be always independent, which is thus not always correct.

3. CONTENT OF THE DRAFT CODE

3.1 Storage Loads in Silos

Rules are given for tall silos, squat silos, and homogenizing silos.

Tall silos are calculated for loads as indicated in Fig. 4. All loads are treated as variable.

For the filling condition, the horizontal wall load, hopper load, and wall friction load are each calculated from silo geometry and the parameters described above. For different designs, one, two or all three of these three load sets may be controlling. For each of these loads, the most adverse value of the load occurs when a different set of extreme values is chosen for the parameters λ and μ. Thus the horizontal wall load is at its most adverse value when λ is at its maximum and μ at its minimum. One combination of values may control the horizontal reinforcement in a concrete silo, whilst another may control the buckling of a steel silo. Thus the complete design must examine both minimum and maximum values for both λ and μ.

The above loads are all fixed symmetrical loads, but the incidence of unsymmetrical loading on silos is so high that some means of guaranteeing strength under unsymmetrical loads must be ensured. To this end, a notional 'patch load' is added to the symmetrical loads. The patch load is

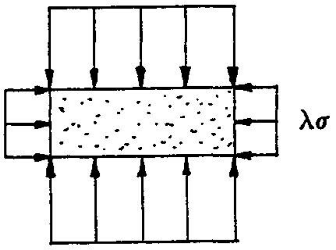

Figure 1a: Relation between vertical and horizontal stresses in a silo

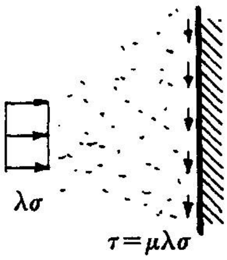

Figure 1b: Relation between horizontal stress and wall friction stress

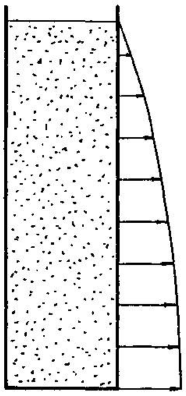

Figure 2a: Horizontal pressure distribution in a silo

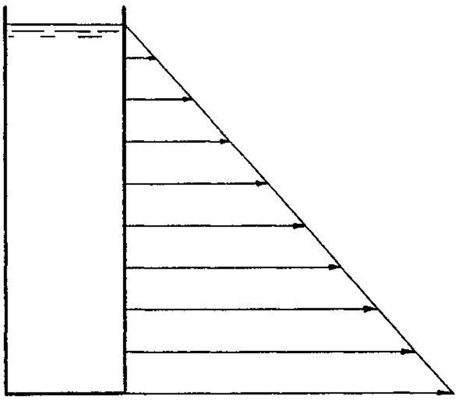

Figure 2b: Horizontal pressure distribution in a tank

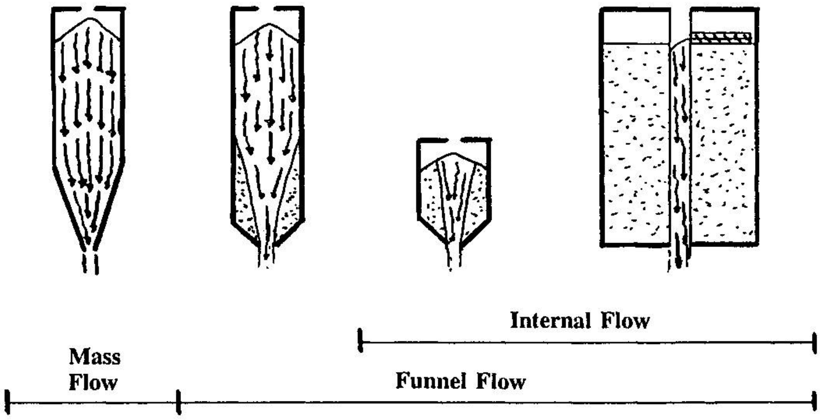

Figure 3: Flow patterns

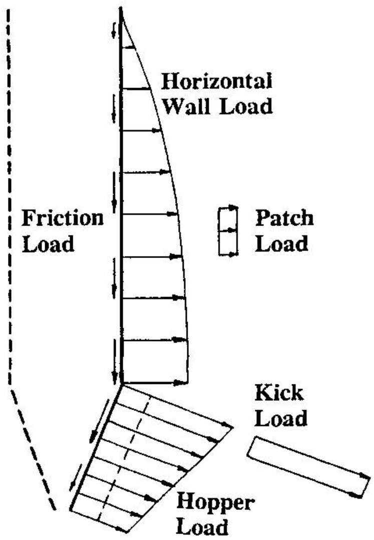

Figure 4:
Loads in a tall silo

a live load that acts on any part of the silo wall on two opposite square areas. The magnitude of the patch pressure increases as the eccentricity of the inlet or outlet increases.

The discharge loads on the vertical wall are in the same pattern as the filling loads, but increased by an over-pressure coefficient C, which takes values between 1.0 and about 1.5. The value of 1.0 is for top unloaded silos (no gravity discharge) and for squat silos with height to diameter ratios below 1. The type of material also influences C, so that the materials that are most likely to produce big discharge pressures have higher values of C. The loads on silo bottoms are defined as being the same for discharge as for filling, except that a 'kick load' is prescribed for the upper part of the hopper in mass flow silos.

Squat silos (height to diameter ratio less than 1) are essentially calculated for the filling pressure, with two additional rules. One allows for a more realistic pressure distribution on the upper part of the silo wall. The other prescribes a more realistic distribution of the bottom load, limiting it to a maximum of the density times the distance to the surface.

Homogenizing silos and silos with high filling velocity must be calculated as for other silos but with an additional loading case: a hydrostatic pressure distribution with an aerated density of 80 per cent of the bulk density.

The parameters to be used in all the above calculations may be taken from a table included in the code or may be determined according to specifications given in the code and guidelines given in an annex.

3.2 Storage Loads in Tanks

Tanks are simply calculated for the hydrostatic pressure.

3.3 Other Actions on Silos and Tanks

References are given to other parts of the Eurocode concerning gravity loads, snow loads, wind loads, thermal actions, differential settlements, and accidental actions. However, some information of special relevance for silos and tanks is included. This consist of some guidance related to filling with hot stored materials and some rules concerning pressure distributions for seismic actions to be used together with the Eurocode on seismic loads.

3.4 Combination values

Noticing that silos and tanks are often filled most of the time and that the loads are likely to be present at a high value it is recommended that the load factor as an accompanying action shall be 0.9 times the load factor as a predominant load. The same value is proposed for serviceability limit states. For combinations with accidental actions, 90 per cent of the load should be prescribed.

4 REMAINING TASKS AND TIME SCHEDULE

Especially the following items need more consideration:
- the values of the material parameters given in the draft, and
- the relation between the part on silos and tanks and the other parts of the Eurocodes. The present draft should be seen as only the starting point for the linkage to the other parts of which the most important in this respect are the other parts of EC1, the code for soil mechanics, the seismic code, and the structural codes for concrete and steel.

The remaining tasks and changes as a result of the ongoing inquiry are planned to be finished by September 1992 and forwarded to CEN/TC250/SC1 for its evaluation.

5 CONCLUSION

The present draft gives rules for loads in silos and tanks. The rules are simple and based on well defined physical parameters.

The preparation of the draft is well advanced, and it is expected that the last major changes will be included in a draft by September 1992.

6 REFERENCES

[1] "Silos and Tanks. Outline for part 11". Report for the Eurocode Task Group on Actions on Structures. June 1987.

[2] "Eurocode for Actions on Structures". Draft, June 1990, CEN TC 250/SC1/90.

[3] Eurocode 1: Basis of Design and Actions on Structures
Volume 4: Actions in Silos and Tanks
Draft, PT8, February 1992, CEN/TC250/SC1

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