Plenary Session 3

Eurocode 1: Actions on Structures

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EUROCODE 1
Basis of Design and Actions on Structures
Part 2.1: Densities, Self Weight, Imposed Loads

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Summary

This paper is divided into two parts. The first Part (Section 1) describes the background against which the Sections in ENV 1991-2-1 on Densities and Self Weight has been drafted and identifies some of the problems in achieving a fully harmonised code. The second Part (Section 2) describes the background of the choice of the loading models and numerical values for loads and roofs in the Section in ENV 1991-2-1 on Imposed Loads.)

1 ENV 1991-2-1 Sections on Densities and Self Weight

1.1 Introduction

In developing the Sections on Densities and Self Weight of ENV 1991-2-1, consideration was given to the contents of the National Codes of the CEN Member States and the International standard ISO 9194 [5].

There are however differences in the scopes and specifications of the codes of the CEN Member States relating to Self Weight 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 off 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 2.1.

1.2 Scope and Field of Application

These Sections of ENV 1991-2-1 apply to the weight 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 moveable 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; floor and walls, cladding 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.

1.3 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 these sections of ENV 1991-2-1 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.

1.4 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 (e.g. beams or columns) be estimated from their dimensions and the densities of their constituent materials; the weights of parts of the structures (e.g. whole floors or whole storeys) and of non-structural elements (e.g.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 (e.g. 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 (e.g. 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; e.g. 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.

1.5 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.

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 during the period leading to the transposition of ENV 1991-2-1 into an EN.

2. Imposed loads on buildings

2.1 Scope of ENV 1991- Part 2.1 Section 6

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 Figure 1.

Part 2.4 Section 6 Imposed Loads on Buildings

6.1 Representation of actions
6.2 Load arrangements
6.2.1 Horizontal members
6.2.2 Vertical members
6.3 Imposed loads - characteristic values
6.3.1 Residential, social, commercial and administration area
Table 6.1: Categories of building areas
Table 6.2: Imposed loads on floors in buildings
6.3.2 Garage and vehicle traffic areas
Table 6.3: Traffic areas in buildings
Table 6.4: Imposed loads on garages and vehicle traffic areas
6.3.3 Areas for storage and industrial activities
6.3.4 Roofs
Table 6.5: Categorization of roofs
Table 6.6: Imposed loads on roofs
6.4 Horizontal loads on partition walls and barriers due to persons
Table 6.7: Horizontal loads on partition walls and barriers due to persons

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

2.2 Areas of dwellings, offices etc.

For areas of dwellings, offices etc. the imposed loads depend on the type of occupancy, see Figure 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 Figure 2a.
- 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 Figure 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, Figure 2c.

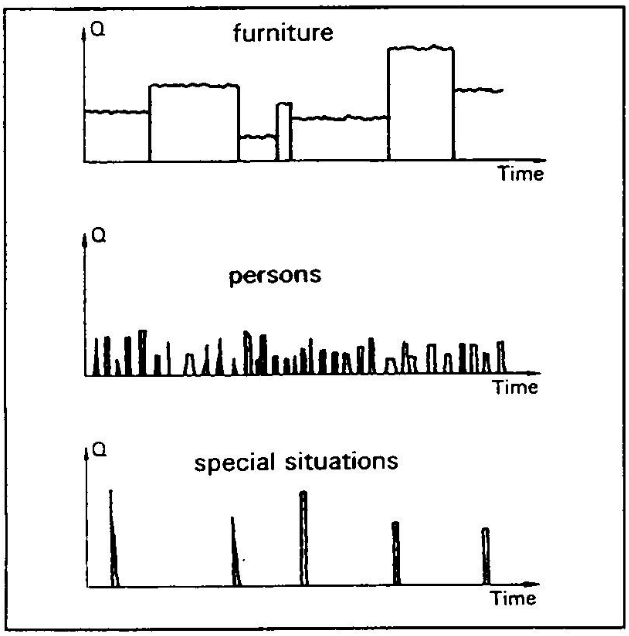

Figure 2:
Time variability of the load:
- Furniture and heavy equipment
- By persons in ordinary load situations
- Special load situations

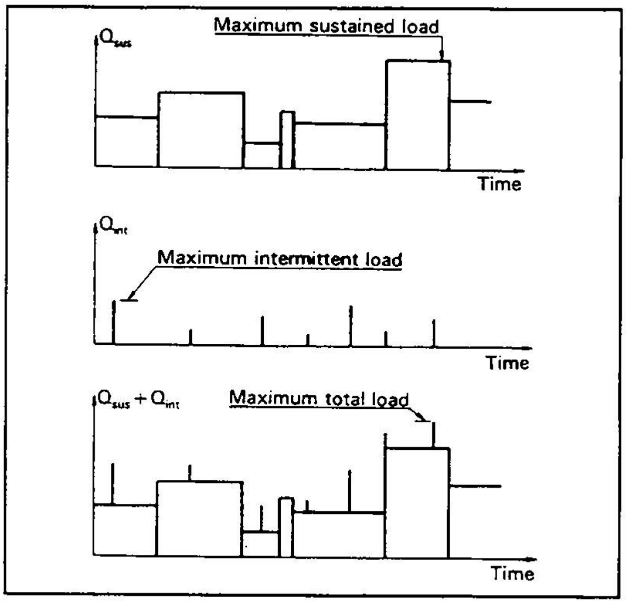

Figure 3:
Sustained load (Q_{sus}), intermittent load (Q_{int}) and total load as stochastic process representing the variability

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:
   - a quasipermanent (sustained) load, Figure 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 intermittentd loa, Figure 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 intermittend live load is shown in Figure 3c.

4. To determine the design values a reference period of 50 years and a reliability index ß = 3.80 has been adopted and the characteristic values p_K were determined from the design values p_d by $p_k = p_d / \gamma_Q$ where $\gamma_Q = 1.50$ was used.

Unfortunately the statistical database for the determination of the characteristic values is rather poor; the numerical load measurements in the literature [ ] 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. Figure 4 gives some values determined in this way.

Imposed Load	Tributary area			
	m^2	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.

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.

Figure 5 gives some examples from these comparisons. Figure 6 gives the the final proposals for the characteristic values of the uniformly distributed loads q_K and the combination factors y_i and for a concentrated load Q_K acting alone in dependance of the category on use of the floor.

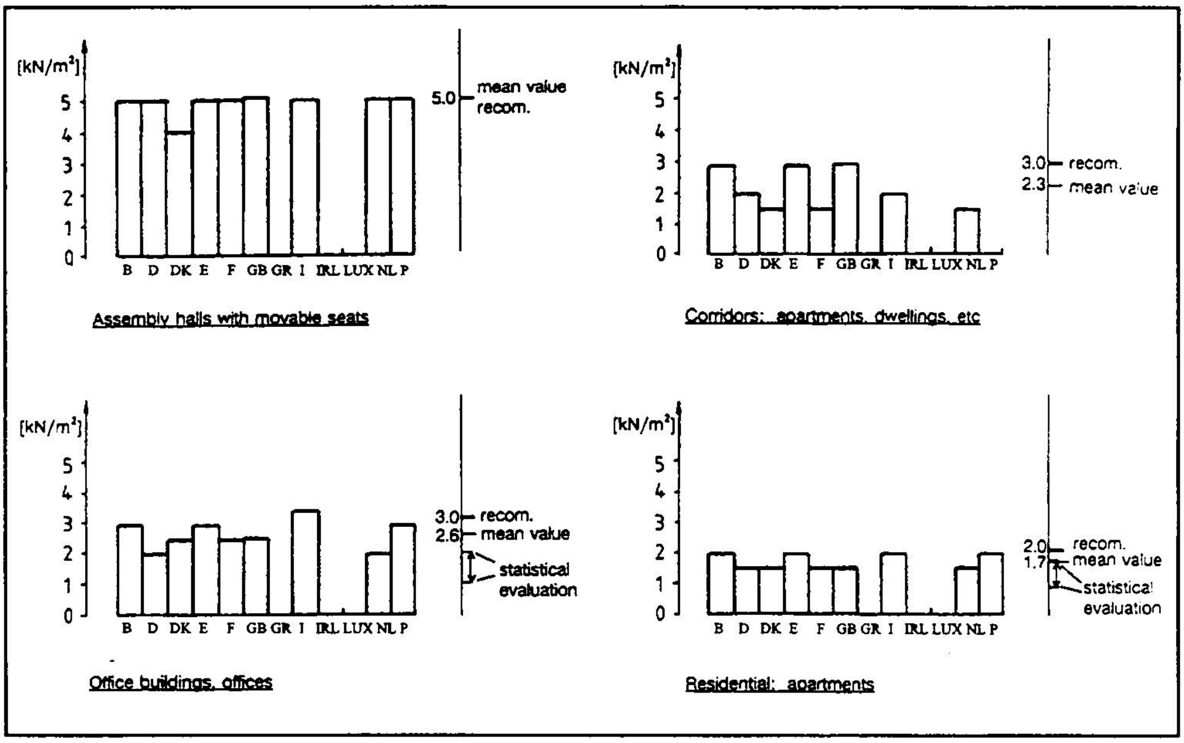

Figure 5: Comparison of European load regulations.

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

Figure 6: Imposed loads on floors in buildings

2.3 Garage and vehicle traffic areas

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 Figure 7. 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.
- the temporal characteristics of the loads at the individual parking places are modelled by a rectangular wave renewal load process, see Figure 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 t_d/t_u.

Design values and characteristic values calculated with these assumptions are given in Figure 9. These values have been used in defining the characteristic values and combination values in Part 2.4 of EC 1, which are given in Figure 10. By the simultaneous action of uniformly distributed and concentrated loads the influence of the tributary area has been taken into account.

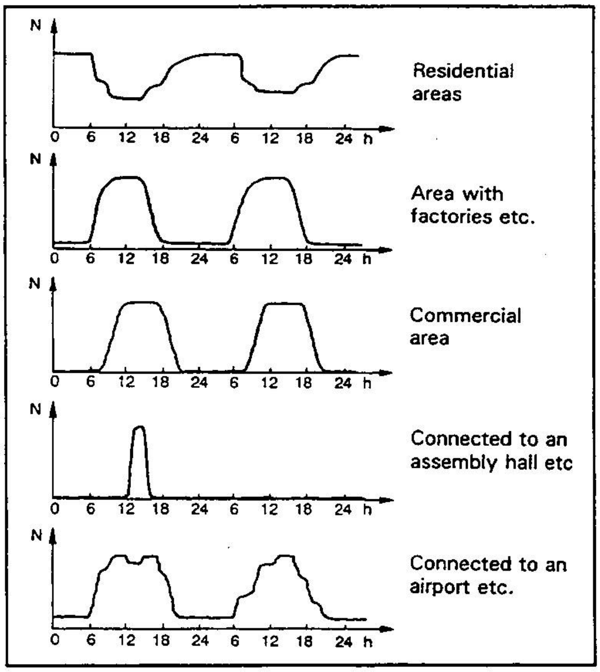

Figure 7: Schematic diagrams of the daily fluctuation of the total number of cars in car parks with different locations

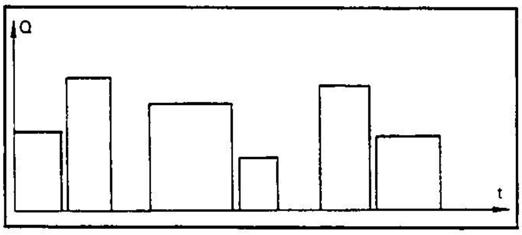

Figure 8: Rectangular renewal wave proces

Imposed load	Tributary area m^2	p_k kN/m^2	ψ_o
standard	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 $[kN/m^2]$	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 E 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

2.3 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 Figure 11. These values have been derived from a comparison of national codes.

Roofs	q_k $[kN/m^2]$	Q_k [kN]
Category G	0,75	1,5

Figure 11: Imposed load on roofs

2.5 Horizontal Loads on Partition Walls and Barriers due to Persons

For barriers or partition walls having the function of barriers, horizontal forces due to persons are given as shown in Figure 12. These values are not suitable for the design of railings in sports stadia.

Use of the loaded area	q [kN/m]
Category A	0,5
Category B	1,0
Category C and D	1,5

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

2.6 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.

REFERENCES

[1] Background Document: Chapter 6: Imposed Loads on Floors and Roofs, June 1990

[2] CIB-W81-Report Publication 116: Actions on Structures - Live Loads in Buildings

[3] CIB-W81-Report: Actions on Structures - Loads in Car Parks, Sept. 1991

[4] Sentler, L: Live Load Surveys: A review with discussions, report 78 Lund, Sweden 1976

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

Eurocode 1 / Part 2.2
Actions on structures exposed to fire

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Summary

This Part 2.2 of Eurocode 1 is concerned with actions on structures exposed to fire. It is intended for use in conjunction with the fire design Parts of ENV 1992 to 1996 and ENV 1999 which give rules for designing structures for fire resistance. Thermal actions given in the main text of ENV 1991-2-2 are mainly confined to nominal thermal actions. Some data and models for physically based and more realistic thermal actions are given in informative annexes which may be improved by ongoing research. Mechanical actions shall be combined in accordance with ENV 1991-1 "Basis of design" using the accidental combination.

1. Introduction

The European Commission issued on 21 December 1988 [1] a directive concerning the products used in the construction of buildings and civil engineering works (Construction Product Directive "CPD").
The term "construction product" refers to products produced for incorporation, in a permanent manner, in the works and placed as such on the market. It includes materials, elements, and components of prefabricated systems or installations which enable the works to meet the essential requirements.

The following essential requirements have to be fulfilled:

1. Mechanical resistance and stability,
2. Safety in case of fire,
3. Hygiene, health and environment,
4. Safety in use,
5. Protection against noise,
6. Energy economy and heat retention.

Concerning "safety in case of fire", the Directive states:

"the construction works must be designed and built in such a way that in the event of an outbreak of fire:
* the load bearing capacity of the construction can be assumed for a specific period of time,
* the generation and spread of fire and smoke within the works are limited,
* the spread of fire to neighbouring construction works is limited,
* occupants can leave the works or be rescued by other means,
* the safety of rescue teams is taken into consideration".

For each of these essential requirements, an Interpretative Document was written by a specific Technical Committee of the Standing Committee set up by the CEC to follow the implementation of CPD.

In the Interpretative Document "safety in case of fire" [2], it is foreseen that the essential requirement may be satisfied as far as structural elements are concerned by:
* tests according to harmonised standards or EOTA (European Organization for Technical Approvals) guidelines or,
* harmonised calculation and design methods or,
* a combination of tests and calculations.

Testing methodology standards are mainly developed by CEN TC 127 (Technical Committee N° 127) and calculation methods (Structural Eurocodes) are developed by CEN TC 250. These sets of European standards, which contain the sum of European and world-wide knowledge, gathered during the last decades, in the field of fire resistance and more specifically on the behaviour of structures in fire, should lead to an uniform manner of assessing the fire resistance of structures throughout Europe.

In EC2 to EC6 and EC9, Parts 1.1 deal with normal design at room temperature and Parts 1.2 deal with structural fire design [4 to 9]. In Part 2.2 of Eurocode 1, the actions in case of fire [3] include both mechanical actions, given by the probable loads applied to a structure during a fire, and thermal actions, represented by the temperature increase in the air and due to a fire.

2. Mechanical Actions

As regards mechanical actions, it is commonly agreed that the probability of the combined occurrence of a fire in a building and an extremely high level of mechanical loads is very small. In this respect the load level to be used to check the fire resistance of elements refers to other safety factors than those used for normal design of buildings. The general formula to be used to calculate the relevant effects of actions is:

$$\sum \gamma_{GA} \cdot G_{k,j} + \psi_{1,1} \cdot Q_{k,1} + \sum \psi_{2,1} \cdot Q_{k,i} + \sum A_{d(t)} \qquad \text{(F.1)}$$

where:

$G_{k,j}$ = characteristic value of the permanent action ("dead load")
$Q_{k,1}$ = characteristic value of the main variable action
$Q_{k,i}$ = characteristic value of the other variable actions

γ_{GA} = partial safety factor for permanent actions in the accidental situation, [1,0] is suggested
$\psi_{1,1}$; $\psi_{2,i}$ = combination factors for buildings according to table 9.3 of ENV 1991-1 [10]
$A_{d(t)}$ = design value of the accidental action resulting from the fire exposure.

This accidental action is represented by:
* the temperature effect on the material properties and
* the indirect thermal actions created either by deformations and expansions caused by the temperature increase in the structural elements, where as a consequence internal forces and moments may be initiated, either by thermal gradients in cross-sections leading to internal stresses.

For instance, in a domestic, residential or an office building with imposed loads as the main variable action ($Q_{k,1}$) and wind or snow as the other variable actions, the formula is

$1,0\ G_k + 0,5\ Q_{k,1}$ since ψ_2 for wind and snow are equal to zero.

For a storage building the formula becomes

$1,0\ G_k + 0,9\ Q_{k,1}$.

When, in a domestic, residential or an office building, the main variable action is considered to be the wind load ($Q_{k,1}$ in the case) and the imposed load ($Q_{k,2}$ in this case) is the other variable action, the formula is

$1,0\ G_k + 0,5\ Q_{k},1 + 0,3\ Q_{k,2}$.

In the case of snow as the main variable action, the formula becomes

$1,0\ G_k + 0,2\ Q_{k,1} + 0,3\ Q_{k,2}$.

Generally this leads in the fire situation to a loading which corresponds to 50 to 70 % of the ultimate load bearing resistance at room temperature for structural elements.

3. Thermal Actions

Concerning thermal actions, a distinction is made between nominal fires and parametric fires.

Nominal fires are conventional fires which can be expressed by a simple formula and which are assumed to be identical whatever is the size or the design of the building. Nominal fires are mainly (see figure 1) the standard fire (ISO-834), the hydrocarbon fire reaching a constant temperature of 1100°C after 30 min, and the external fire (used only for external walls) reaching a constant temperature of 680°C after 30 min (see 4.2 of ENV 1991-2-2). They have to be used when it is required to prove that an element has the necessary level of fire resistance to fulfil national or other requirements expressed in terms of fire rating related to one of these nominal fires.

"Parametric fires" is a general term used to cover fire evolution more in line (compared to nominal fires) with real fires expected to occur in buildings. They take into account the main parameters which influence the growth and development of fires. In this respect the temperature-time curve (and subsequently the heat flux) varies when the size of the building or the amount or kind of fire load, varies.
This more realistic way of determining the thermal action due to an expected fire can only be used in association with an assessment by calculation methods. Due to the large variety of possible temperature-time curves in a building, the assessment method would have been very expensive if the only possibility was to test components in furnaces for each particular temperature-time fire curve.

In the current version of Part 2.2 of Eurocode 1, there are two methods of representing parametric fires:

* For internal elements (elements inside the building) simplified formulas can be used which take into account the following main parameters: the fire load, the opening factor $O = A_v \cdot \sqrt{h} / A_t$, (with A_v: area of vertical openings, h: height of vertical openings, A_t: total area of enclosure), and the thermal properties of the surrounding walls of the compartment (see 4.3 and Annex B of ENV 1991-2-2).

An example of the results of using these formulas with a fire load $q_{f,d} = 600$ MJ/m^2, and an opening factor varying from 0,02 m$^{1/2}$ to 0,20 m$^{1/2}$ is shown in figure 2 (a). However according to a research founded from 1987 to 1991 by ECSC [12], similar parametric temperature-time curves were established. Results obtained by this approach, for the same set of previous data, are shown in figure 2 (b), and seem to be more realistic.
Indeed the heating curves of figure 2 (a) show that the fire is ventilation-controlled [11] for all opening factors from 0,20 m$^{1/2}$ to 0,02 m$^{1/2}$, and that in the cooling phase the temperature-time curve is strictly linear!

On the contrary the heating curves of figure 2 (b) show that the fire is fuel-controlled for opening factors from 0,20 m$^{1/2}$ to 0,10 m$^{1/2}$ and becomes ventilation-controlled for smaller opening factors. Further more in the cooling phase the temperature-time evolution is curved!

** The temperature of structural elements outside the building can be evaluated by using a calculation method in which the maximum temperature in the compartment and in the flames going out of openings are calculated (see Annex C of ENV 1991-2-2).

As explained in (23) of the foreword of Part 2.2 of Eurocode 1, it is planned to introduce, in the final stage, a more general concept dealing with "natural fires" in order to permit the use of commonly agreed fire models [13, 14, 15].

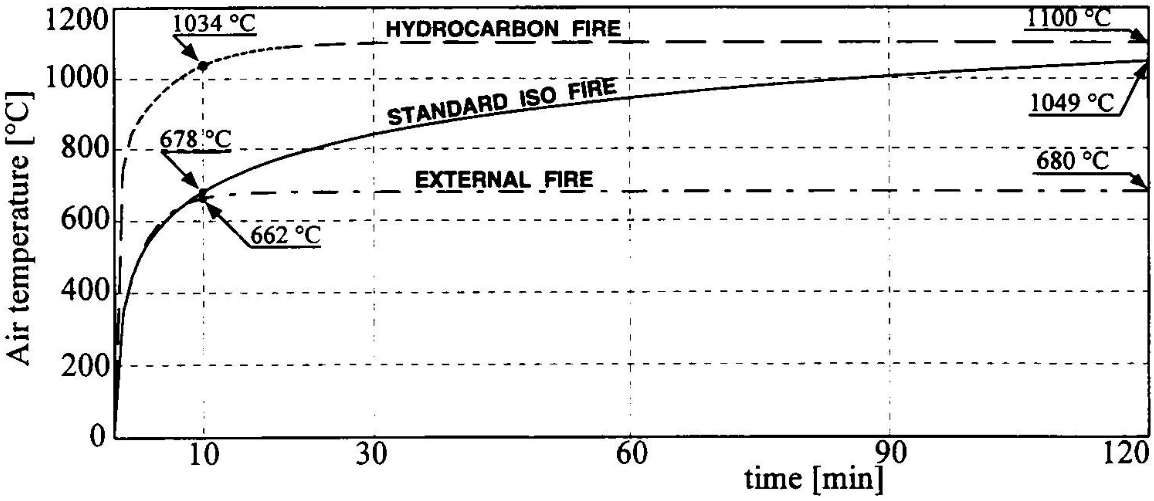
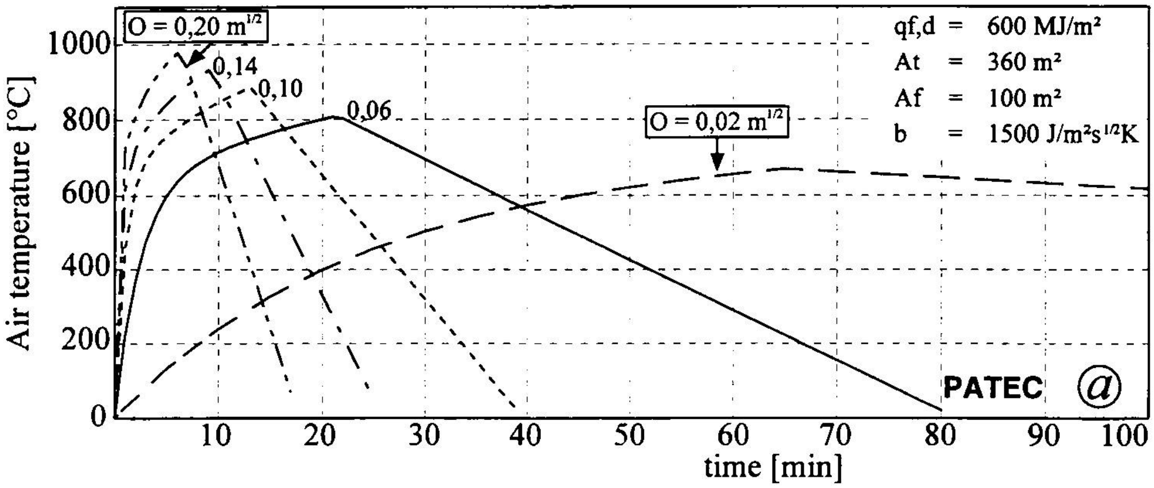


Fig. 1. Nominal temperature-time curves according to 4.2 of ENV 1991-2-2 [3]



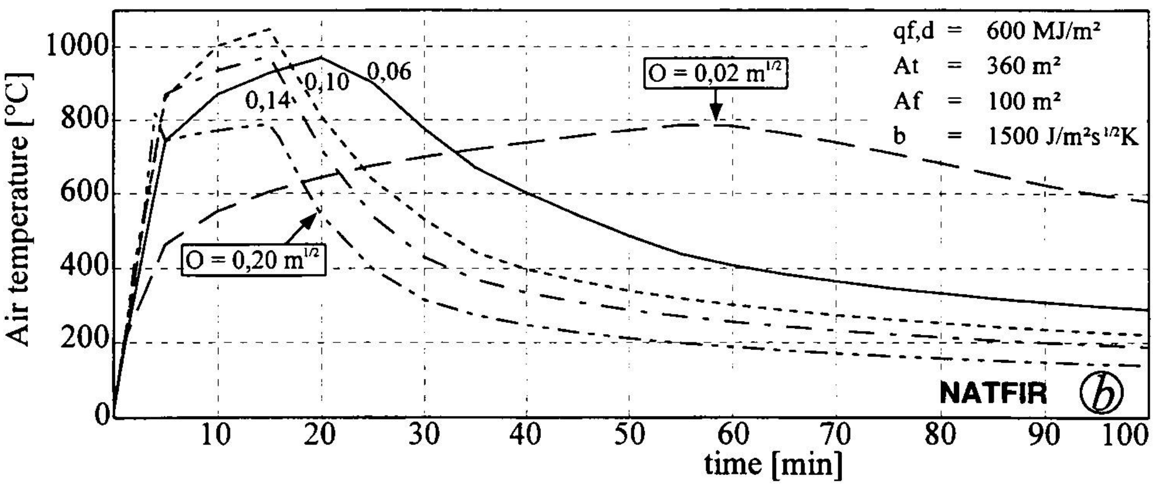


Fig. 2. Parametric temperature-time curves according to 4.3 and Annex B of ENV 1991-2-2 (see ⓐ) and following E.C.S.C. RESEARCH 7210-SA/112, Activity C1 [12], (see ⓑ).

4. Fire Load Densities

The fire load density q_d should be a design value either based on a fire load classification of occupancies, either determined specific for an individual project by performing a survey of fire loads from the occupancy.

The fire load density is one of the main parameters used in all existing fire models, as well the simple calculation models like the parametric fires (Annex B of ENV 1991-2-2), or the equivalent time of fire exposure (Annex E of ENV 1991-2-2) or any one-zone or two-zone fire models, as the advanced calculation models like multi-zone fire models or computational fluid dynamics (CFD) models [13, 14, 15].

The design fire load density is defined as

$$q_d = \gamma_q \cdot \gamma_n \cdot q_k \qquad \text{(D.1)}$$

where q_k is the characteristic fire load density either taken from a classification of occupancies like Annex 1 of SIA 81 [16], or established by calculation using equations (D.2), (D.3), (D.4) and table D.1 of ENV 1991-2-2.

The safety factor γ_q depends on the consequences of a failure and the frequency of a fire.

$$\gamma_q = \gamma_{q1} \cdot \gamma_{q2}, \text{ normally} \geq 1 \qquad \text{(D.1.1)}$$

Whereas the safety factor γ_{q1} related to the consequences of a failure is function of the size of the compartment under fire and of the number of storeys of the building (see figure 3), the safety factor γ_{q2} related to the frequency of fires is depending on the danger of fire activation and therefore is function of occupancies (see figure 4).

The differentiation factor γ_n is accounting for various active fire safety measures able to reduce the practical heating effect of the characteristic fire load density.

$$\gamma_n = \gamma_{n1} \cdot \gamma_{n2} \cdot \gamma_{n3} \cdot \gamma_{n4} \cdot \gamma_{n5} \leq 1 \qquad \text{(D.1.2)}$$

Referring to various national regulatory documents [16 to 19] and following (3) of D1 of ENV 1991-2-2, figure 5 was established showing that γ_n could be split up in different contributions due to the fire extinction effect of approved sprinklers γ_{n1}, the automatic fire detection γ_{n2}, the automatic fire alarm transmission γ_{n3}, the independent water supply for sprinklers γ_{n4} and the existence of a work fire brigade γ_{n5}.

By considering such active fire safety measures, the effect of the design fire load density q_d may be reduced down to 20 % of its initial value. The severity of the corresponding natural heating will of course drop in a significant way, whereas as well the safety of people as the safety of the building will be largely improved. Based on the previously given national regulatory documents [16 to 19] and on the ECSC Research "Natural Fire Safety Concept" [15], managed by ProfilARBED Research from 1994 to 1998, the comprehensive effect of all active fire safety measures should be included in the forthcoming EN 1991-2-2.

Surface of compartment (m^2)	Safety Factor γ_{q1}	
	One storey building	Building with several storeys
≤ 2500	1,00	1,25
5000	1,05	1,35
10000	1,10	1,45
20000	1,20	1,55
30000	1,25	1,60
60000	1,35	/
120000	1,50	/

Fig. 3. Safety factor γ_{q1} according to DIN 18230-1 [19] in function of the size of the compartment under fire and for different types of building

Safety factor γ_{q2}	Danger of Fire Activation	Examples of Occupancies
0,85	small	artgallery, museum
1,00	normal	residence, hotel, paper industry
1,20	mean	manufactory for machinery & engines
1,45	high	chemical laboratory, painting workshop
1,80	very high	manufactory of fireworks or paints

Fig. 4. Safety factor γ_{q2} according to SIA 81 [16] in function of the occupancy of the building.

Official Document		γ_{ni} Function of active Fire Safety Measure							$\gamma_n = \gamma_{n1} \cdot \gamma_{n2} \cdot \gamma_{n3} \cdot \gamma_{n4} \cdot \gamma_{n5}$
		Automatic Water Extinguishing System	Automatic Fire Detection and Alarm	Automatic Alarm Transmission to Fire Brigade	Independent Water Supplies			Work Fire Brigade	
					0	1	2		
Title	Date of Publication	γ_{n1}	γ_{n2}	γ_{n3}	γ_{n4}			γ_{n5}	$\gamma_n^{max} = \gamma_{n1} \cdot \gamma_{n2}$
ENV 1991-2-2	1995	0,60	/	/	/			/	0,60
DIN 18230-1 Project	1995	0,60	0,90	/	/			0,6 / 1,0	0,32 / 0,54
New Zealand Limit State Des. Guide	1993	0,60	/	/	/			/	0,60
ANPI (B)	1988	0,58 ❄	0,82	included in ❄	1,0	0,86	0,65	0,5 / 1,0	0,16 / 0,48
SIA 81	1984	0,59 / 0,50	0,83	0,83 / 1,0	/			0,67 / 1,0	0,23 / 0,49

Fig. 5. Differentiation factor γ_n accounting for various active fire safety measures like fire extinction by sprinklers γ_{n1}, automatic fire detection γ_{n2}, automatic fire alarm transmission γ_{n3}, independent water supply for sprinklers γ_{n4} and the existence of a work fire brigade γ_{n5}, in function of different regulatory documents [3, 16 to 19].

5. The Equivalent Time of Fire Exposure

The following approach, given in Annex E of ENV 1991-2-2, allows to use realistic fire conditions depending on the design fire load density $q_{f,d}$ and on the ventilation, even when the design of members is by tabulated data or simplified rules related to the standard fire (see ENV 1992-1-2, ENV 1993-1-2, ENV 1994-1-2).

In fact by definition the equivalent ISO time is the time during which a given structural element has to be submitted to the ISO fire curve in order to obtain, in that element, the same maximum temperature than the natural fire curve would have produced. It was when applying this principle to concrete cross-sections, with reinforcing bars protected by a 3 cm thick concrete layer, that equation (E.1) was established.

$$t_{e,d} = q_{f,d} \cdot k_b \cdot w_f \tag{E.1}$$

The equivalent ISO time $t_{e,d}$, formulated in this way with k_b and w_f given in (4) and (5) of Annex E, is material independent, but $t_{e,d}$ should in fact be and in reality is, material dependent.

Indeed when using Annex B, giving parametric temperature-time curves according equations (B.1) to (B.6), and applying these natural heating curves as well as the standard fire to different cross-sections, the non linear finite element code CEFICOSS allowed to establish the differential & transient temperature fields in those cross-sections [22].

* The conclusion drawn when considering the previous definition of the equivalent time was:

- The two methods, Annex E and Annex B + CEFICOSS, lead to similar equivalent times for concrete cross-sections or sections made of protected profiles.
- However if the cross-section is an unprotected steel profile, these two methods give contradictory results. In fact equation (E.1) of Annex E gives too high values for $t_{e,d}$, or finally leads to a much too severe heating up of the steel section under a given natural fire. Indeed,
  if A_v (or O) $\nearrow$, $t_{e,d} \nearrow$ according to Annex B
  but $t_{e,d} \swarrow$ according to Annex E.

Therefore equation (E.1) may be improved to apply to unprotected steel:

$$t_{e,d} = \left(q_{f,d} \cdot k_b \cdot w_f\right) k_c \tag{E.1.1}$$

with k_c correction factor function of the material composing structural cross-sections and defined in figure 6.

** For composite construction elements, the equivalent time as defined before, based on an unique temperature equivalence, is not valid anymore. Indeed such a procedure would depend on the considered element, beam or column, and on the considered point in the cross-section. When applying this method to the composite frame tested under natural fire conditions in Braunschweig April 12 and 24, 1989 [20], it was found that the equivalent ISO time $t_{e,d}$ would scatter from 42 to 80 minutes (see figure 7).

In fact for composite structures the equivalence between a natural fire and the ISO-fire has to be based on the equivalence of the load bearing capacity. This was done for the previously named composite frame test [20], and the equivalent ISO time $t_{e,d}$ obtained was 46 minutes (see figure 8)!

Cross section Material	Correction factor k_c
Reinforced Concrete	1,0
Protected Steel	1,0
Not protected Steel	13,7 · O

Fig. 6. Correction factor k_c, to be applied to the equivalent time $t_{e,d}$ of Annex E, in order to cover various cross-sections. (O is the opening factor defined in Annex B in $m^{1/2}$)

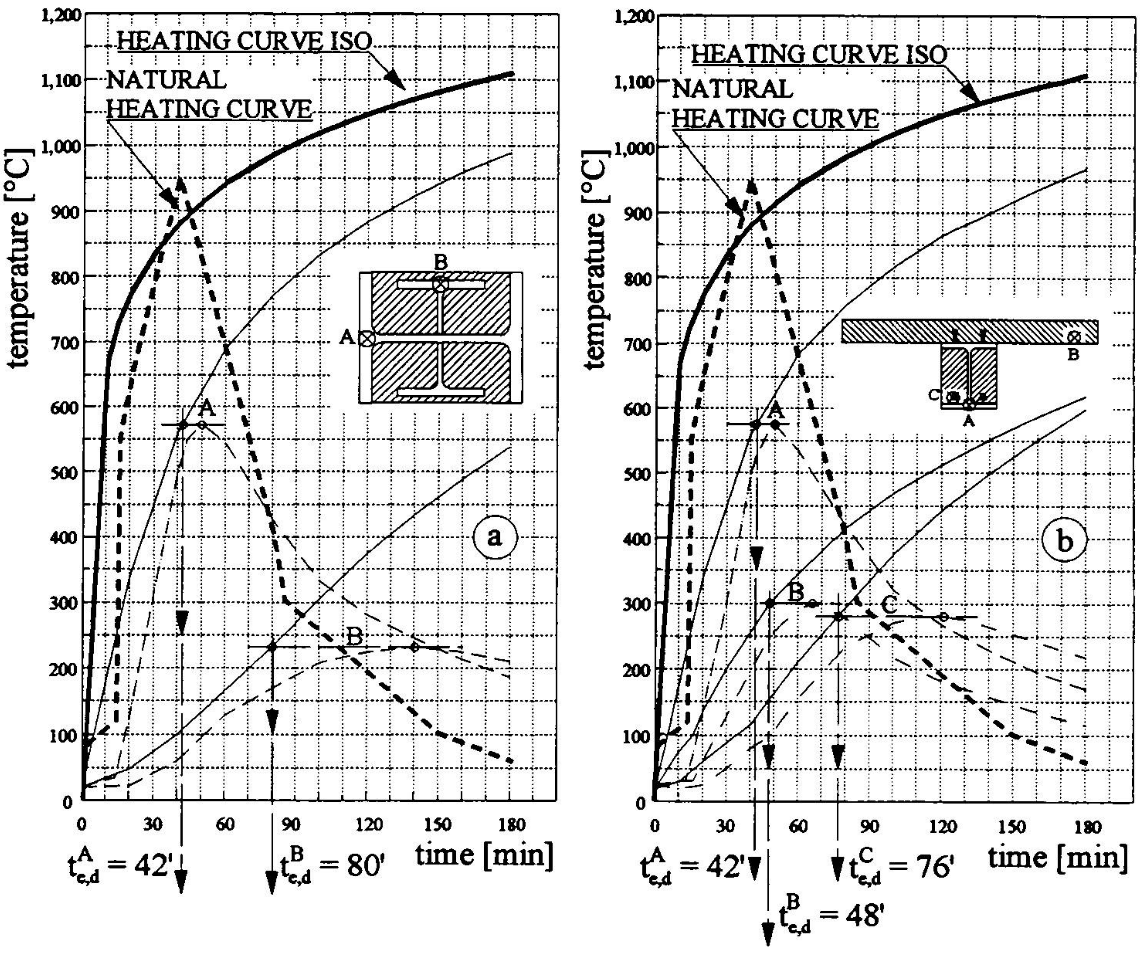


Fig. 7. Temperatures calculated by CEFICOSS in the composite column (a) and the composite beam (b) for the ISO heating (——) or the natural heating (- - -) according to the Braunschweig tests on April 12 and 24, 1989 [20].

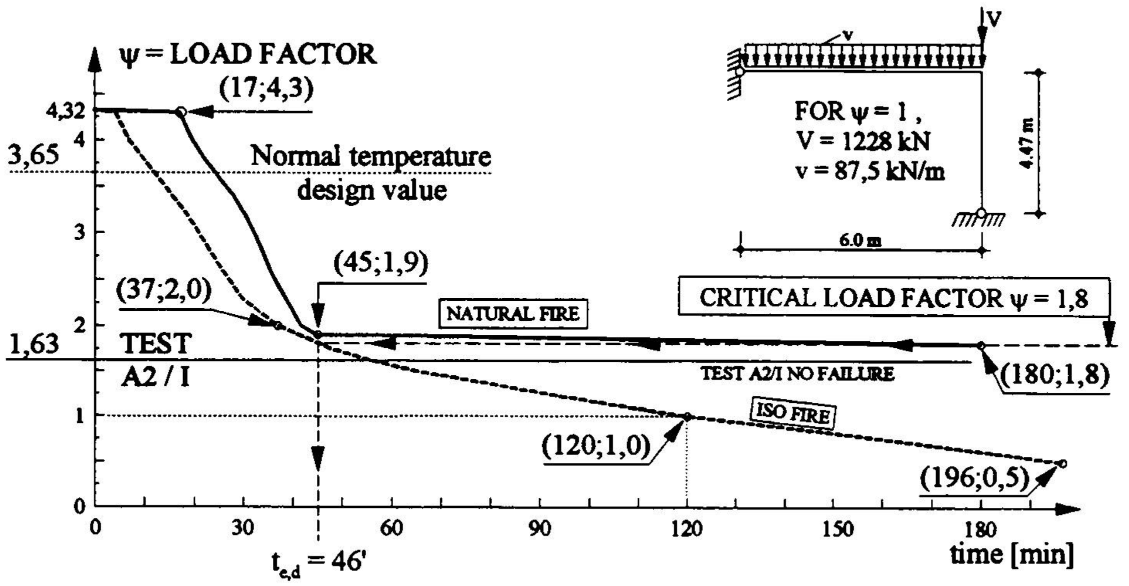


Fig. 8. Composite frame A2-I / Fire resistance calculated by CEFICOSS in function of ψ, for the ISO-Fire and the Natural Fire defined in Figure 7 [20]

6. Conclusions

The main aspects of the present ENV 1991-2-2, officially issued by C.E.N. on February 9, 1995, have been highlighted. In some domains like that of parametric fires, of two-zone and multi-zone fire models, of the equivalent ISO time, and of the consideration of active fire safety measures, research is proceeding. Efforts need to be undertaken to determine the properties of combustible materials, their combustion behaviour and net calorific values, and to clarify the problem of the rate of heat release RHR in function of time [21, 22, 23].

Undoubtedly it will be possible to improve in a significant way EN 1991-2-2, the next version of the present prestandard ENV 1991-2-2, which even now allows the use of more realistic fire models.

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Design load and structural safety - Developments in Japan

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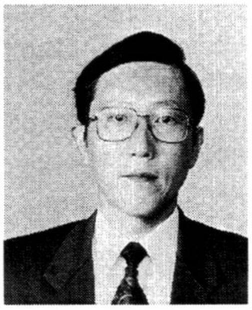

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Summary

Developments of Japanese building codes are reviewed with emphasis on load specifications. Current load specifications are briefly summarised for live loads, snow loads, wind loads and earthquake loads mostly defined for the allowable stress design. Specified Load intensity values are based on those determined in an empirical manner in or before1950. Recent developments are introduced paying attention to Recommendations for Loads on Buildings published by the Architectural Institute of Japan. Further discussions are developed for recent activities towards a new concept of performance-based structural design.

1. Introduction

The major purpose of structural design is to make structures safe against anticipated actions and loads in their lifetime. As far as Japanese building code is concerned, the intensities of all design loads are specified numerically and most engineers can easily take those numbers for their structural analysis calculations. Since environmental actions could often exceed the specified intensity, the engineer should consider the safety margin in various ways. However the degree of safety is not explicitly stated in current regulations, then individual engineers have to face the difficulty to make their judgments on the structural safety. They have to accept the safety according to codified numbers without any quantitative measure of safety, although such codified values tend to determine automatically the safety degree irrespective of indivitual environmental conditions and users' demands.

Among many parameters related to the structural safety, the maximum load intensity generally has most significant uncertainty. This means that the design load controls the structural safety to a fairly large extent. Then engineers should pay much more attention to the design load determination. Developments in Japanese building code are reviewed and current specifications of design loads are critically discussed. Then activities for new concept of structural design are introduced for future developments towards a performance-based structural design and/or a limit state design.

2. Developments of Japanese building codes [1), 2)]

The first Japanese building code, Urban Building Law and Urban Planning Law were promulgated in 1919 to regulate building constructions and city planning in six major cities. Seven chapters in strength requirements are 1). General, 2). Wood construction, 3). Masonry and brick work, 4). Steel construction, 5). Reinforced concrete construction, 6). Independent chimney and 7). Strength calculation. The allowable stress design method was used specifying allowable stresses for structural materials. Only vertical loads were specified and no descriptions were given for the snow, wind and earthquake loads. Design live loads were similar to those in then New York City Building Code.

The 1923 Kanto Earthquake caused serious damages to the capital city, then earthquake resistant regulations were introduced according to proposals by Professor Riki Sano. The seismic coefficient of 0.1 was specified. The anticipated maximum seismic coefficient was estimated as 0.3 and was reduced to one third by considering the safety factor of 3 used in determining allowable stress level relative to the material strength.

The Urban Building Law was effective until 1950, although proposals for the revision were often discussed in the Architectural Institute of Japan (A.I.J.). The 1937 proposal by A.I.J. included 1). detailed classification of building use for live loads, 2). detailed classification of structural woods, 3). increase of allowable stresses for steel and 4). introduction of specifications for snow (unit weight of 29.4 Pa/cm) and wind (1 kPa velocity pressure for the height less than 15 m).

In 1944, Temporal Japanese Standard 532 "Loads on Buildings" and 533 "Fundamentals of structural calculations of buildings" were enacted to replace the Urban Building Law during the war time. Major revisions may be summarised as, 1). increase of the design load for important structures, 2). reduction of live loads, 3). introduction of snow load (unit weight of 19.8 Pa/cm), 4). introduction of wind load (392 $\sqrt{h}$ (Pa) as velocity pressure, where h is the height (m)), 5). horizontal seismic coefficient 0.15 for ordinary soil and 0.20 for soft soil, 6). allowable stress values are twice those specified in the Urban Building Low and 7). consideration for calculation error, construction error and variability of materials. The intentional reduction of structural safety was clearly observed in these war-time standards.

Under the new Japanese constitution, the Building Standard Law was proclaimed in 1950. The principle of requirements to structures is stated in Article 20 as, [3)]

> 1). Buildings shall be of structure safe from dead load, live load, snow load, wind pressure, ground pressure and water pressure as well as earthquake or other vibration or shock.
>
> 2). In preparing drawings/specifications for buildings as mentioned in Article 6 paragraph 1 item (2) or (3), the safety of the structure thereof shall be confirmed through structural calculation, where Article 6 paragraph 1 item (2): Wooden buildings which have three or more stories, or have a total floor area exceeding 500 square meters and item (3): Buildings other than wooden buildings, which have two or more stories or have a total floor area exceeding 200 square meters.

Design load values and related equations are specified in Articles 83 to 88 in Enforcement Order, based on the allowable stress design procedure. Allowable stresses are also specified in Articles 89 to 106 for structural materials. Special attentions have been paid for seismic resistant design after major earthquakes, e.g., the Tokachi-oki earthquake, 1968, the Miyagiken-oki earthquake, 1978. But otherwise specified values for loads and allowable stresses have been mostly unchanged since 1950.

3. Current load specifications [3]

3.1. Live loads

A table is provided in Article 85 of Enforcement Order as alternative values to actual ones as summarised in Table 1. Although in Article 85 it is mentioned that live-load values can be estimated according to actual conditions, values of Table 1 are used in most cases of practices.

Table 1 Current Live load values (kPa) for various uses —summary

member	floor	girder /column
houses	1.76	1.27
offices	2.94	1.76
shopping stores	2.94	2.35
meeting rooms/no seats	3.53	3.23
garages	5.39	3.92

Live loads are combined with dead loads to calculate stresses due to permanent loads to be compared with the long-term allowable stress, f_L. f_L for the steel tensile stress is equal to 2/3 of the nominal yielding stress and f_L for the concrete compression is equal to 1/3 of the nominal ultimate compressive stress. Recent live load survey data are summarised as in Figure 1. [4] When 99 percentile values of load intensities are compared with live load values in Table 1, e.g. for houses, offices and shopping stores, the latter is 1.2 to 1.8 times greater than the former by considering typical unit areas for the floor and the girder as 20 m^2 and 50m^2 respectively.

3.2 Snow loads

Deepest snow fall values are specified by special administrative agencies. The ratio of those values to statistically obtained values associated with 50 year return period varies from 0.61 to 1.5. [5] These ratios indicate that the snow load in current design practices varies in terms of the return period in a very wide range such as 5 years to 2000 years. The unit weight of snow is specified as 19.8 Pa/cm or more, and in heavy snow regions, special administrative agencies increase its value to 29.4 Pa/cm.

Stresses due to snow loads are combined with stresses due to permanent loads and compared with the short-term allowable stress, f_S. In heavy snow regions long-term stress checking is also in practice for reduced snow loads. f_S for the steel tensile stress is equal to the nominal yielding stress and f_S for the concrete is twice f_L.

3.3 Wind loads

The velocity pressure is given by 588 $\sqrt{h}$ (Pa) for $h \leqq 16$ (m) and 1176 $h^{1/4}$ (Pa) for $h > 16$ (m). The latter was introduced in 1981 by considering significant conservatism of the former when applied to a part of the height greater than 16 m. 1176 $h^{1/4}$ was originally used for wind load for the first tall building in Japan, Kasumigaseki Building constructed in 1968 whose height is 147 m, by Dr. Kiyoshi Muto, and has been introduced in the cladding design in a form of notice of Ministry of Construction Since 1978.

Zoning factor was prepared in a form of notice of Ministry of Construction in 1959, but has not been used in practices in most administrative agencies. The ratio of the velocity pressure value at h = 10 (m) to corresponding statistically obtained values associated with 50 year returm period for flat open terrain varies from 1.1 to 2.2 covering most of Japanese islands. [5] These ratios indicate that the design wind load in terms of the return period in a wide, mostly conservative, range such as 80 years to 6000 years.

Stresses due to wind loads are combined with stresses due to permanent loads and compared with the short-term allowable stress. Such conservatism mentioned above may not be seriously criticised by practice engineers as earthquake loads often dominate the wind loads except for very light and/or very tall structures. For tall buildings with height over 60 m, return period based wind loads have recently been used according to A.I.J. Recommendation.

3.4 Earthquake loads

Basic base shear coefficients are specified as 0.2 for the short-term allowable stress design and 1.0 for the capacity design. The latter was introduced in 1981 by considering the necessity of introduction of capacity design. Zoning factor is applied to reduce seismic shear force to 0.9 or 0.8 in lower seismicity regions, except for Okinawa where Zoning factor of 0.7 is used.

Vibration characteristic factor is defined as a function of natural period of the structure and the estimated dominant period of the soil, and is applied to multiply the basic base shear coefficient. Structural characteristic factor is specified to take into account the ductility performance of post-yielding structural behavior to the earthquake load in the capacity design. Values vary between 0.3 and 0.7 for reinforced concrete structures and between 0.25 and 0.5 for steel structures.

Many seismic hazard maps have been available and as far as statistical estimations concerned in a range of relatively short period such as less than 100 years, a fairly good agreement among maps can be pointed out.[5] The ratio of design earthquake load for the allowable stress design to the 50 year return period value varies from 0.42 at Tokyo to 1.3 at Fukuoka for six major cities.[5] These values seem to correspond to the return period of 10 to 80 years.

Dynamic response analyses are commonly used to examine the elastic and inelastic response behaviour of tall buildings with height over 60 m. The basic intensity of input earthquake ground motions is 25 cm/s for the elastic response and 50 cm/s for inelastic response to the criteria of story ductility factor of 2. Both El Centro, NS, 1940 and Taft, EW, 1952 motions have been still used as respresentative input motions since the time of Kasumigaseki building, although the irrationality has been pointed out for their particular spectral characteristics.

4. A.I.J. load recommendations [4)]

The Architectural Institute of Japan has been producing various types of standards and recommendations. Design specifications for steel structures and Standard for structural calculation of reinforced concrete structures have been used widely in practice in accordance with Building Standard Law and Enforcement Order. Recommendations for Loads on Buildings was first published in 1975 then revised in 1981, in 1986 and in 1993.

The principles of 1993 version may be summarised as, 1). common basic load intensity for various loads based on statistical data, 2). design loads for both allowable stress design and limit state design, 3). equivalent static loads for dynamic actions such as winds and earthquakes, and 4). providing variability information for physical parameters involved in load estimation.

Values associated with 100 year return period are used commonly as a basic load intensity for snow, wind and earthquake and 99 percentile values are used for a basic live load intensity. Return period conversion factor, R, was introduced and formulated as,

$$R = 0.40 + 0.13 \ln r \quad \text{for snow depth in heavy snow regions} \tag{1}$$
$$R = 0.22 + 0.17 \ln r \quad \text{for snow depth in other regions} \tag{2}$$

$$R = 0.54 + 0.1 \ \ln r \quad \text{for wind speed} \tag{3}$$

and
$$R = \left(\frac{r}{100}\right)^{0.54} \quad \text{for peak ground acceleration and velocity} \tag{4}$$

where r is the retrurn period.

Design loads for the allowable stress design are determined by taking an appropriate return period by applying return period conversion factor of Equations (1) to (4). Design loads for the limit state design are defined as products of the load factor and basic load values. The load factor is formulated by a commonly used form derived for log-normal ramdom variables as,

$$\gamma = \frac{1}{\sqrt{1+V_s^2}} \exp(\alpha_s \beta_T \sigma_{\ln s}) \frac{\bar{S}}{S_n} \tag{5}$$

where V_S is the coefficient of variation of load effect S, α_s is the separation facotr, β_T is the target reliability index, $\sigma_{\ln s}$ is the standard deviation of logarithm of S, $\bar{S}$ is the mean of S for a reference period and S_n is the basic value of S.

4.1. Live loads

A formula for the basic live load, L, is given by

$$L = L_o \times C_E \times C_{R1} \times C_{R2} \tag{6}$$

where L_o is the basic live load intensity corresponding to the 99 percentile value of arbitrary-point-in-time statistics for a reference influence area of 18 m², C_E is a conversion factor to Equivalent Uniformly Distributed Load (E.U.D.L.), C_{R1} is a reduction factor for changing of influence area and C_{R2} is a reduction factor for nultiple-story column loads.

4.2 Snow loads

Two types of snow loads are defined; one is roof snow loads without control based on the maximum snow depth as in conventional practices and the other is roof snow loads with control based on 7 day snow accumulation.

A formula for the basic snow load, S, is given by

$$S = d_o \times \rho_s \times \mu \times g \times C_e \tag{7}$$

where d_o is the basic snow load intensity, i.e. the 100 year return period value of maximum snow depth on the ground, ρ_s is the equivalent snow density, μ is the roof shape coefficient consisting of the basic coefficient as a function of the average wind speed in winter and the slope of the roof, a coefficient for the irregularity due to snow drift and a coefficient for the irregularity due to sliding, g is the gravity acceleration and C_e is the environmental coefficient.

The equivalent snow density is expressed as a function of design snow depth to meet recent data available as shown in Figure 2. The snow temperature seems not to be a significant parameter for the equivalent snow depth and a unique formula in Figure 2 was employed for ρ_s in Japan.

4.3 Wind loads

Basic wind load is estimated by Equation (8).

$$W = \frac{1}{2}\rho(U_o E_H)^2 C_f G_f A \tag{8}$$

where ρ is the air density, U_o is the basic wind speed, i.e. the maximum wind speed (10 minute mean) associated with 100 year return period over a flat opten terrain at an elevation of 10 m above the ground, E_H is the wind speed profile factor at the height H and defined as a product of exposure factor E_r and topography factor E_g, where five terrain categories are introduced to specify E_r with different power law index varying between 0.10 and 0.35 for a power law wind profile model, C_f is the wind force coefficient, G_f is the gust effect factor, and A is the projected area.

A contour map of basic wind speed is provided by exmanining meteorological data with a new terrain correction scheme.[6] The annual change of terrain roughnesses seem to be a considerable factor to the variation of averaged maximum wind speed over meteorological observation sites as shown in Figure 3.

Prediction procedures for wind-induced responses in both windward and lateral directions have been improved significantly based on recent experimental and analytical works and are extensively utilized to improve the accuracy of G_f.[7] For example non-dimensional critical wind speeds for buildings with a rectangular section for aeroelastic instability are tabulated for various side ratios in open and rough terrains.

A simplified procedure for the estimation of wind loads is provided for buildings satistying following conditions; 1). Shapes and structural systems of buildings are not special, 2). Mean roof height is less then 15 m, 3). Projected breadth is at least half the mean roof height but less than 30 m. Wind loads based on a simplified procedure yields slightly more conserrative estimation than that by a detailed one.

4.4 Earthquake loads

Detailed descriptions of earthquake loads appeared in A.I.J. recommendation at the first time in 1993 version, mostly bacause of the difficulty in reaching a general consensus, although many state of the art reports have been published.[8]

Basic horizontal story shear force of the i-th story is estimated by a response spectrum method as,

$$Q_i = D_s \sqrt{\sum_{m=1}^{k} \left[\left(\sum_{j=i}^{n} W_j \beta_m U_{jm} \right) S_A(T_m, h_m) / g \right]^2} \tag{9}$$

where D_s is the structural characteristic factor and equals unity in the elastic response, k is the number of necessary modes, n is the number of story, W_j is the gravity load of the j-th story, β_m is the participation factor for the m-th vibration mode, U_{jm} is the m-th vibration mode of the j-th story, g is the gravity acceleration, T_m , h_m are the natural period and the damping ratio of the m-th mode respectively and

$$S_A(T,h) = \begin{cases} \left(1 + \dfrac{f_A - 1}{d} \dfrac{T}{T_C} \right) F_h G_A A_o & 0 < T \le dT_c \\ F_h f_A G_A A_o & \text{for} \quad dT_c < T \le T_c \\ \dfrac{2\pi}{T} F_h f_v G_v V_o & T_c < T \end{cases}$$

where f_A is the acceleratioin response amplification ratio for $dT_c < T \leq T_c$, f_v is the velocity response amplification ratio for $T_c < T$, dT_c and T_c are the lower and upper bound periods of the range, where $S_A(T,h)$ is constant, respectively, F_h is the damping modification factor and $F_h = 1$ for $h = 5$ % and A_o and V_o are basic peak acceleration and velocity of earthquake ground motion at the reference firm soil accociated with 100 year return period respectively, and G_A and G_v are soil type modification factors for the peak acceleration and velocity respectively.

As discussed in 3.4, there are significant discrepancies between the difference of earthquake loads in current practices in low seismicity and high seismicity regions and that appeared in return period consistent peak ground accerelation(PGA). From the optium reliability viewpoint, a higher safety is justified for a low seismicity area and a lower safety has to be accepted for a high seismicity area[9], therefore unique retrun period value throughout the country may not be appropriate . Variation of the annual maximum PGA is much greater than that of the annual maximum snow depth or the annual maximum wind speed. Nevertheless the return period conversion factor, R, in Equation (4) shows a representatine tendency of the probabilistic characteristics as shown in Figure 4, where the Frechet distribution is consistant to the formula by Equation (4).

5. New concept of structural design

A draft standard for limit state design for steel structures was published by A.I.J. in 1990.[10] However it has not been approved by the Ministry of Construction yet and never been used in practices. The target reliability index for ultimate limit states was determined by calibrating to the current allowable stress design. The reference period for the ultimate limit state is 50 years. $\beta_T = 2.5$ for live loads, $\beta_T = 2.0$ for snow and wind loads and $\beta_T = 1.5$ for earthquake loads are used to calculate load and resitance factors for the ultimate limit state.

The ministry of construction formed new committees in 1995 to carry out a three year project to develop a new structural design frame-work, where performance-based design is discussed to replace specification-based design such as the allowable stress design. What is the performance-based design will not be answered soon, however the required performance for a structure is generally the safety and the serviceability, therefore the limit state design is regarded as one form of performance-based design.

The great Hanshin earthquake, January 17, 1995 shifted discussions of committees towards the seismic safety. The principle of 1986 revison of Building Standard Law Enforcement order explains that buildings may be slightly damaged by earthquakes occurring a few times in structural lifetime and may be seriously damaged by an possible maximum earthquake during lifetime but without human losses. Although the frequency of Jannuary 17, 1995 event is very low, when it occurred, people tend to think that their buildings should be damage free to this kind of earthquake. The retrum period of P.G.A. can be estimated as 500 years or over when estimated from statistics in A.I.J. load recommendation (1993) [11], and it seems reasonable that current seismic design criteria can not prevent property losses. Current design practices seem to have worked satisfactorily considering technical viewpoints in 1986, as most collapses of houses and buildings were caused by their deteriorations or poor maintainance or old standard design or poor workmanship.

Nevertheless demands for higher safety standard are discussed after observing many damages due to the earthquake. The minimum requirement is not necessarily to be the standard and engineers should have opportunities to provide higher safety according to owners' or users' demands. Probabilistic approahes are convenient to provide a rational measure for the safety or the frequency of earthquake occurrence, although the reliability concept has not been commonly accepted even in the engineering society.

Since building constructions are parts of economic activities, the target safety cannnot free from economic considerations. Optimum reliability based on the minimum total cost principle certainly provides a good guidance to determine the design load level.[9] Now in Japan people can see various states of damages due to strong motions and know how expensive to restore them. The performance of buildings under various levels of P.G.A. has to be described not only by engineering measures such as the maximum acceleration, the deflection, the ductility ratio etc, but also by an economical measure such as repair or replacement costs.

The reliability concept has been getting familiar throughout the world for engineered products. At the same time people have difficulties to measure the safety in a probabilistic manner. In particular when the structural safety is closely related to human losses, the appropriateness of target reliability in structural design is not easily understood by people who actually suffered from the recent earthquake. The reliability of seismic hazard in a long return period renge, i.e. a very low probability range, is also relatively poor in comparison with other variables of load intensities such as wind speed or snow depth. A great amont of works still seem necessary to include most recent findings in earthquake engineering such as active fault data, soil amplification and soil-structure interaction mechanism and so on in order to estimate lifetime maximum design earthquake load in a sophisticated manner.

6. Conclusions

The allowable stress design procedure has been used for buildings in Japan since 1919. Many improvements have been reflected in regulations in particular for the seismic resitance after every major earthquakes. However basic design load intemsity specifications have not been changed since 1950. Some attempts in Architectural Institute of Japan have been made to introduce the limit state design and to provide rational load estimation procedures including load intensity statistics. Performance-based design is now under discussion in committees formed by the Ministry of Construction to create a new structural design framework. Description of performances of structure under various levels of load conditions are to be explicitly used for design criteria. Reliability concept is also expected to be reflected in the new design procedure.

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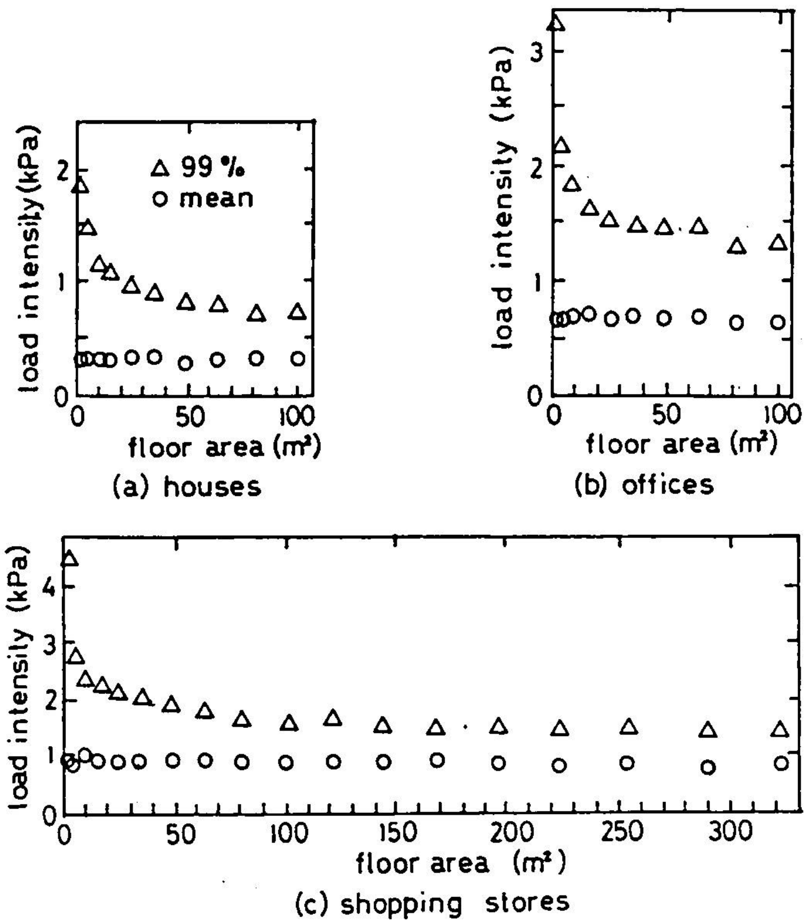

Fig. 1 Live load statistics with floor area [4]

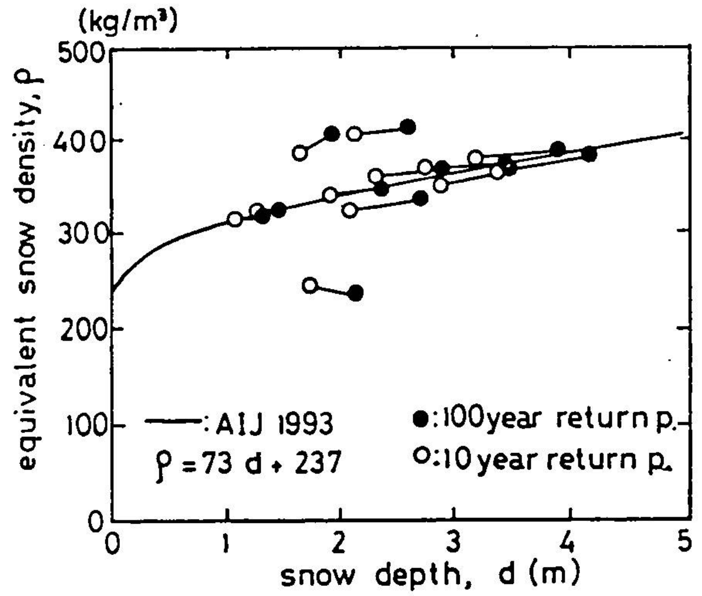

Fig. 2 Equivalent snow density with annual maximum snow depth for 12 sites in Japan [4]

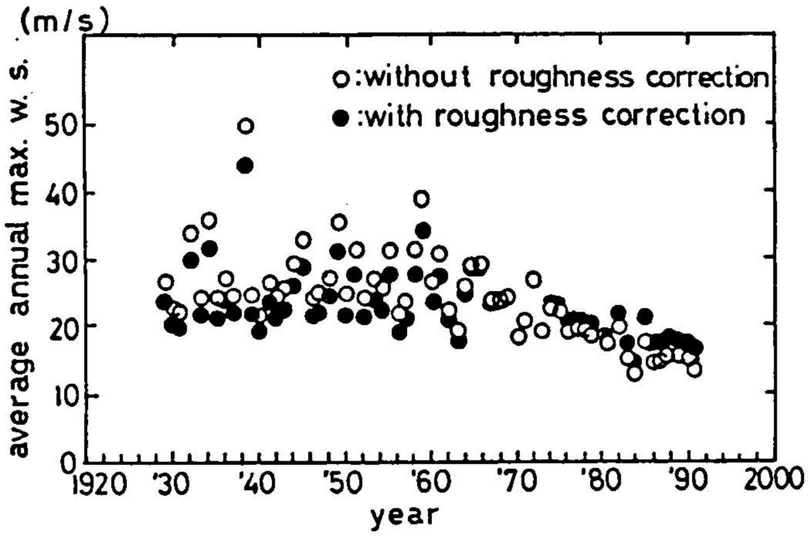

Fig. 3 *Variation of annual maximum wind speed averaged over meteorological stations in Japan* [4)]

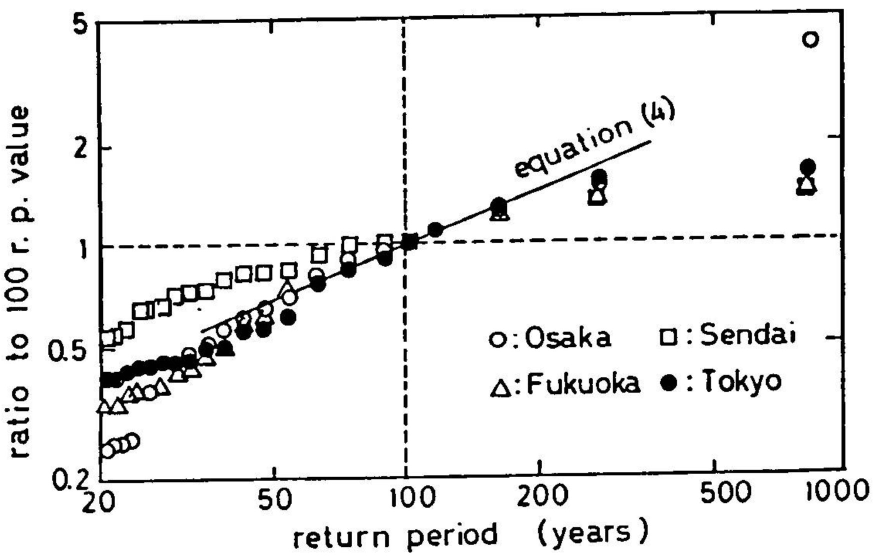

Fig. 4 *Ratio to 100 year return period value for PGA in Japan* [4)]

Structural engineering: tools, core task and interfaces

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Summary

This paper deals with aspects of code checking related to efforts, computer supported applications in code checking and the recent change of codes in The Netherlands. Moreover core tasks and tools of the structural engineer are discussed and interface aspects between the structural engineer's domain and the surrounding environment are presented.

1. Codes, artist's pencil or imposed forced labour?

The use and application of codes by structural engineers can roughly be split up as follows:
-codes as verification document
-codes as design tool.

1.1 Codes as verification document

The use of codes as verification document contains a variety of aspects:
Legislation
Specific clauses of the euro code based Dutch concrete code presently come into force form part of the legal frame work. Clauses concerned relate to safety, serviceability, health and environment.

Contractual
Integral codes or specific aspects of codes often form part of the contractual relation between parties involved in the construction industry.

Safety/reliability
As a verification document codes provide comfort to the structural engineer once his concept complies with the relevant clauses of the code.

Functional requirements
Compliance with requirements related to durability and stiffness can be demonstrated through code checks.

Code checking is normally performed as a systematic check.
As such the integral contents of a code is considered. Two specific issues are related to this integral character of the code check: level of detailing of the codes: the more detailed codes are, the more detailed code checks will be.

Process of the code check: manually or automated by computer
The level of detailing of the euro codes and the present Dutch codes is more extensive then the recently laid off codes, mainly in the field of partial factors and parameters to describe crack width and deflections:
this is a logical consequence of the research based ability to describe processes like cracking and deflections more fundamentally.

To demonstrate the differences in efforts between the previous and present Dutch concrete code, a comparative study has been carried out [1].
The structure considered is a multiple span girder, loaded with uniform distributed live loads.
The calculations were carried out by an qualified engineer, but with a relative short track record in structural engineering, thus avoiding the jump-through-the-code performance of well experienced engineers.
From an evaluation of the results following conclusions were drawn:
- If code checks are carried out systematically and without experience driven cut-off's, the checks are almost identical.
- Due to the more detailed level of the present code, there is an increase in efforts, mainly due to the extent of numerical calculations

Given the conclusion, comparable but more of the same exercise, the question of computerized code checks is of current interest.
A review of the state of the art software related to code checking [2], learned that code checking by computer is feasible to quite an extent and has significant potentials. The computerized code check has an impact on the time demand for the checking procedures and allows a more extensive parameter check.
So, shouldn't the professional community switch over to such an approach?

1.2 Codes as design tool

Code checking is more than just a numerical exercise:
From the evaluation of the comparative code checking exercise [1] and from in-house experience gained during the development of an knowledge based expert system for building pits, it can be concluded that structural engineers perform their tasks within an experience based reference frame.

This reference frame of methods and values enables the engineer to judge at intermediate steps as whether he is on the right track, heading towards a solution or digging into the ground. An example in this respect related to values is the practical shear stress value in beams, which is above the limit value for unreinforced concrete but appropriate because of practical stirrups to be applied anyway. This reference system is a personnel system for each engineer individually as it is developed by permanently setting results against choices made.
Strictly object oriented, final results can be set against initial choices; most structural engineers however perform within a more universal environment and as such need much more indicators as reference system. This reference system is experienced by the authors as of vital importance for professional performance
The recent change in the Netherlands from integral safety factors towards partial safety factors and also the amendment of specific calculational procedures results into a loss of specific parts of the reference frame.
Given the blessings of information technology and the more detailed level of modern, euro code based, codes, the temptation to fly into automated code check procedures isn't fictitious. Regardless the question as to whether the automated procedures cover all aspects, such approach would create blanks in the reference frame. Manually processed code checks provide the engineer the data to restore the blanks caused by the introduction of the new codes.
As reflected in the investigation on automated code check possibilities [2] authors have the opinion that automated code checks form part of the engineer's luggage, especially for routine work, but after restoration or build-up of the engineer's reference frame. As such an and/and strategy is opted for.

The question as raised in the heading of this section can't be answered in general as even for the individual structural engineer the answer may switch as times go by.

2. Structural engineering, playground or battlefield.

The environment in which the structural engineer nowadays performs is extensive and fast, interactive with a variety of disciplines and surrounded by an increasing spread of techniques, highly specialized software and regularly changing codes.
His performance is expected to be reliable, against low fee, fast and also to reflect the state of the art of modern technologies.
This environment is challenging but requires a strict discipline of the engineer: distinction and control of interfaces between core task and the surrounding environment; not suggesting isolation but controlled interaction.

2.1 Core task

The core task of the structural engineer is to turn degrees of freedom into solutions which comply with functional requirements, specifications and/or codes. In practice this means the selection of concepts, static schemes, materials, dimensions and details. This core task is to be performed in strong interaction with the surrounding environment at interfaces.

2.2 Interfaces

The surrounding environment can be itemized as follows:
Co-operative entities
Supporting entities
Controlling entities, not further discussed in this paper.

The co-operative entities may consist of engineering disciplines in case of multi discipline projects and/or non-engineering disciplines from the construction industry. As concluded from an internal evaluation of a complex design/construct contract [3] the structured and strictly programmed exchange of information at the interface is of vital importance to control the interface and the process as a whole. Recently developed computer systems like VDT (virtual design team) demonstrate and confirm this statement. At present VDT is applied in Delta Marine Consultants to investigate the overall tender process of a submerged tunnel project.

Supporting entities should deliver tools, fit for purpose, at the interface.
Tools may consist of

- Networks, to have access to literature, codes and data banks and to allow exchange of information
- Software; although this might sound as hammering on an open door, due attention is required. From a recent publication in Civil Engineering [4], it was concluded that, in general, there is a gap between software as offered and the engineer's needs. The suggestion to opt for object oriented technology is considered by the authors as a sensible direction, given own experience with the earlier mentioned in-house development of an object oriented system for building pits.
- Results from research and development. Given the structural engineer's working environment as sketched before and the fundamental level reached by research nowadays , it should be obvious that achievements of research have to be processed before they are offered to the structural engineer's community at the interface.
  A survey of structural engineers [5] showed that a highly sophisticated tool wasn't fully explored as the tool didn't match properly at the connection with the user. Improvements at the interface proved to be fairly effective.

Whether the environment in which the structural engineer performs develops as a battlefield or as a playground, heavily depends on the understanding of the underlying processes presented above and the discipline to stick to the consequential playing rules of all parties and above all, all individuals involved.

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A comparison of the new ISO 4355 with CEN ENV 1991-2-3

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1. Introduction

Since the first ISO 4355 "Snow Loads on roofs" was published in 1981, it has to a great extent been the most used document in the process of developing National Snow Load Specifications.

ISO TC 98 "Basis for design of structures" decided in 1986 to start revisional work on the old ISO 4355. The revisional work has resulted in a revised ISO 4355 "Snow Loads on roofs" that was adopted in 1995, and is under publication. The background for ISO 4355 is discussed in Reference (1).

In 1991 CEN formed a specific Project Team (PT) in order to produce EC 1: Snow loads. The PT-work resulted in the ENV 1991-2-3: 1995 "Actions on structures - Snow loads". The background for the ENV is discussed in Reference (2).

The paper will make a comparison between the revised ISO 4355 and the ENV 1995 on snow loads on roofs.

Various parameters that are included in the code format for snow loads on roofs, e.g.:

- Exposure effects
- Thermal transmittance effects
- Shape coefficients
- Snow drift effects

will be discussed and the resulting loads will be compared.

The question, whether a load standard should be concerned only with the load specification as such, or should also incorporate reliability and safety considerations, will be briefly discussed.

2. Formats for the determination of snow load on roofs

2.1 ISO 4355 format

ISO 4355 presents an approximation for the snow load on roofs as a sum of a balanced load part, a drift load part and a slide load part. Thus

$$s = s_b + s_d + s_s \tag{2.1}$$

in which the load parts are approximated by the introduction of product functions, i.e.

$$s_b = s_o C_e C_t \mu_b \tag{2.2}$$

$$s_d = s_o C_e C_t \mu_b \mu_d \tag{2.3}$$

$$s_s = s_o C_e C_t \mu_s \tag{2.4}$$

in which

s_o is the characteristic snow load on the ground
C_e is an exposure coefficient treated in Annex B of rev. ISO 4355 and in 3.1
C_t is a thermal coefficient treated in Annex D of rev. ISO 4355 and in 3.2
μ_b is a slope reduction coefficient
μ_d is a drift load coefficient
μ_s is a slide load coefficient

In ISO 4355 it was decided to describe variation of the parameters with the roof angle β as continuous smooth functions, for which trigonometric functions can be suitable. Moreover, it is attempted to show the consequences of variation in parameter values. Thus, the slope reduction coefficient is defined as

$$\mu_b = \sqrt{\cos(C_m 1{,}5\beta)}\,; \quad \text{for } (C_m 1{,}5\beta) < 90^\circ$$

$$\mu_b = 0\,; \quad \text{for } (C_m 1{,}5\beta) \geq 90^\circ \tag{2.5}$$

C_m is a surface material coefficient, which defines a reduction of the snow load on roofs for surface materials with low surface roughness, defined to vary between unity and 1,333, taking the fixed values:

$C_m =$ 1,333 for slippery, unobstructed surfaces, for which the thermal coefficient $C_t < 0{,}9$ (e.g. glass roofs)

$C_m =$ 1,2 for slippery, unobstructed surfaces, for which the thermal coefficient $C_t > 0{,}9$ (e.g. glass roofs over partially climatic conditioned space, metal roofs etc.)

$C_m =$ 1,0 corresponds to all other surfaces

The variation of μ_b is shown in Fig. 1.

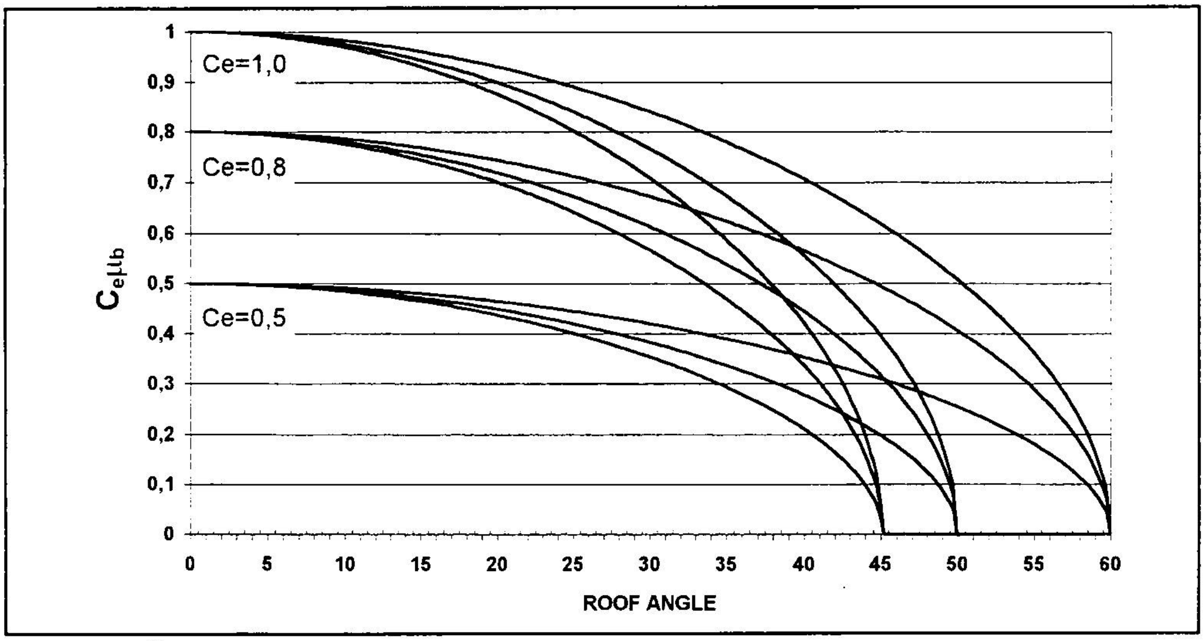

Figure 1 $C_e\mu_b$ for defined values of C_m

The drift load coefficient $\mu_b\mu_d$ is defined by the function

$$\mu_b\mu_d = \mu_b(2{,}2C_e - 2{,}1C_e^2)\sin(3\beta)\ ;\ \text{for } 0° \le \beta \le 60°$$
$$\mu_b\mu_d = 0\ ;\ \text{for } \beta > 60°$$

The form of the drift load coefficient ensures that a certain drift load part always is considered even for regions with very calm winter conditons; i.e. $C_e = 1{,}0$.

The slide load shape coefficient μ_s, giving a slide load from an upper part of a roof onto a lower roof of a multilevel roof, is defined as an approximate load model in connection with shape coefficients for multilevel roofs, in clause 5.4.5.6 of the ISO 4355.

2.2 CEN ENV 1991-2-3 format

CEN ENV 1991-2-3 proposes the following format for the snow load on roofs:

$$s = \mu_i C_e C_t s_k \tag{2.7}$$

where

μ_i is the snow load shape coefficient (see section 7)
s_k is the characteristic value of the snow load on the ground [kN/m²] (see section 6)
C_e is the exposure coefficient, which usually has the value 1,0
C_t is the thermal coefficient, which usually has the value 1,0

3. Comparison of ISO 4355 and CEN ENV 1991-2-3

3.1 Exposure coefficient C_e

In ISO 4355 the exposure coefficient C_e is defined as a reduction coefficient having its maximum value $C_e = 1,0$ for calm winter conditions.
For "normal" winter conditions it is recommended to set $C_e = 0,8$.
The exposure coefficients may be determined from Annex B, mainly depending on defined winter wind conditions and winter temperature conditions, as shown in Table 1.

		Winter wind category		
		I	II	III
Winter temperature category	A	1,0	1,0	0,8
	B	1,0	0,8	0,6
	C	0,8	0,8	0,5

Table 1 Exposure coefficient, C_e

CEN ENV 1991-2-3 has introduced an exposure coefficient C_e into the format. However, since the ENV applies the shape coefficients of the old ISO 4355, which did not have an exposure coefficient in the format, and thus had normal exposure with a value of 0,8 in the shape coefficients, the ENV had to define the normal exposure as $C_e = 1,0$.
It is unfortunate that the coefficient has the same symbol as in ISO 4355, however, with a different scaling.
In ENV 1991-2-3 no specifications are given for possible variation of C_e. The author's suggestion is to harmonize the use of C_e before the final EN is produced. If this is not done, misunderstandings may result.
The national codes of Canada and the United States have exposure coefficients in their formats. Since symbols are different, no misunderstandings are expected.
The ENV opens for national authorities to specify values of C_e.

3.2 Thermal transmittance effects

With the increasing use of glass roofs over the last decade the Working Group of ISO TC 98, SC3, felt that thermal transmittance effects should be introduced into the format, and developed a model for such reductions. This model is presented in Annex D of ISO 4355. However, it is only informative. It could be mentioned that the same approach to C_t is an integral part of the Norwegian standard NS3479 since 1990, and lately also incorporated in Swedish specifications on snow loads. Norwegian experience with the use of C_t ranging from approx. 0,35 - 1,0 is good.

It is felt that CEN should add such guidelines in the EN version.

3.3 Comparison of shape coefficients and snow drift effects for pitched roofs

A comparison of the variation of snow load on a pitched roof as a function of the roof angle β, is shown for the windward side on Fig. 2 and for the leeside on Fig. 3.

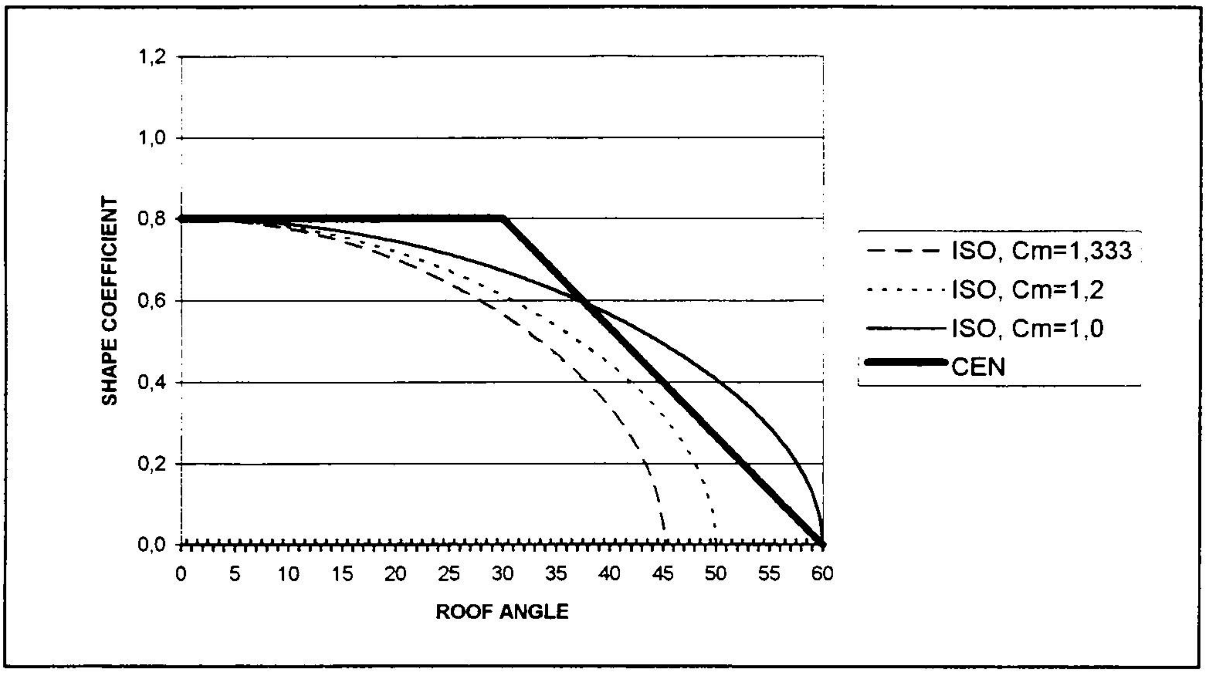

Figure 2

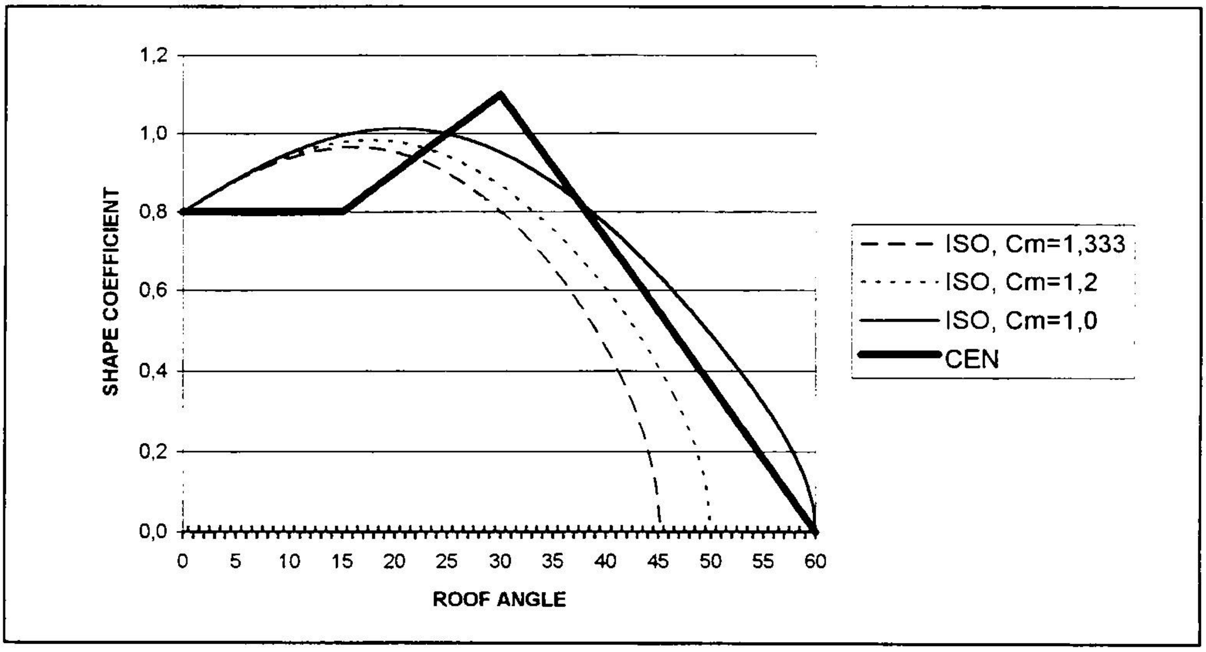

Figure 3

Taking account of new measurements of snow load on roofs, the ISO 4355 has reduced the maximum leeside load by almost 20 percent as compared with the old ISO 4355. Moreover, the maximum drift load as a function of the roof angle has been changed from $\beta = 30°$ to approximately $\beta = 20°$ in accordance with measurements and experience.
The ENV, on the other hand, has reduced the maximum leeside load by approximately 10 percent. However, the maximum is still for $\beta = 30°$.

These differences should be studied, and possibly eliminated, before the final EN is produced.

For monopitch roofs the new ISO 4355 has added a drift load part to the balanced load, leading to an increased load for monopitch roofs. The ENV 1991-2-3 has introduced an extra load case for monopitched roofs. It is hard to see that this load case will cause more unfavorable conditions than the ordinary load.

3.4 Comparison of shape coefficients for curved roofs

The old ISO 4355, 1981, gave two different load cases, which were based on Russian measurements and specifications. Clause 3.2 of the old ISO 4355 had a prescription about partial loading, which said that the load should be applied according to the shape coefficient distribution on any given portion of the roof area, and zero load on the remainder of the area. This led to particularly unfavorable conditions for arhces, which are very sensitive to asymmetrical loading.

In the new ISO 4355, 1995, it is recommended that only half of the snow load on arches shall be considered to be a variable free action, which leads to more favorable conditions for arches. The CEN ENV 1991-2-3 presents two different load cases. The load case II seems to yield larger bending moments than does the case 2 of the old ISO 4355. Since the ENV defines the snow load as a variable free action, the ENV may lead to much more severe conditions for arches than does the new ISO 4355. This problem should be studied thoroughly before the EN-document is finalized.

3.5 Comparison of shape coefficients for multilevel roofs

The ENV 1991-2-3 has prescribed the same shape coefficients as those used in the old ISO 4355, 1981.

The new ISO 4355, 1995, gives more prescriptions for multilevel roofs, which are based on new American research and load surveys, the results of which are felt to be more realistic under varying conditions than were the results of the old ISO 4355, 1981.

In Fig. 4 the load on the lower roof (apart from possible slide load), represented with the shape coefficient μ_w of the CEN ENV, is compared with the sum of balanced load and drift load according to the new ISO 4355.

It should be noted that, in accordance with American surveys, the shape coefficient diminshes with increasing ground load s_0, whereas the shape coefficient of the CEN ENV and the old ISO 4355 is independent of s_0.

It also should be noted that the new ISO 4355 yields larger loads on lower roofs having small differences in level between upper and lower roof, than does the ENV and the old ISO 4355. The cause for this increase is observations of snow load accumulation on the ground or on lower level roofs for arches or pitched roofs sloping down to the lower level, see clause 5.4.5.7 of the new ISO 4355.

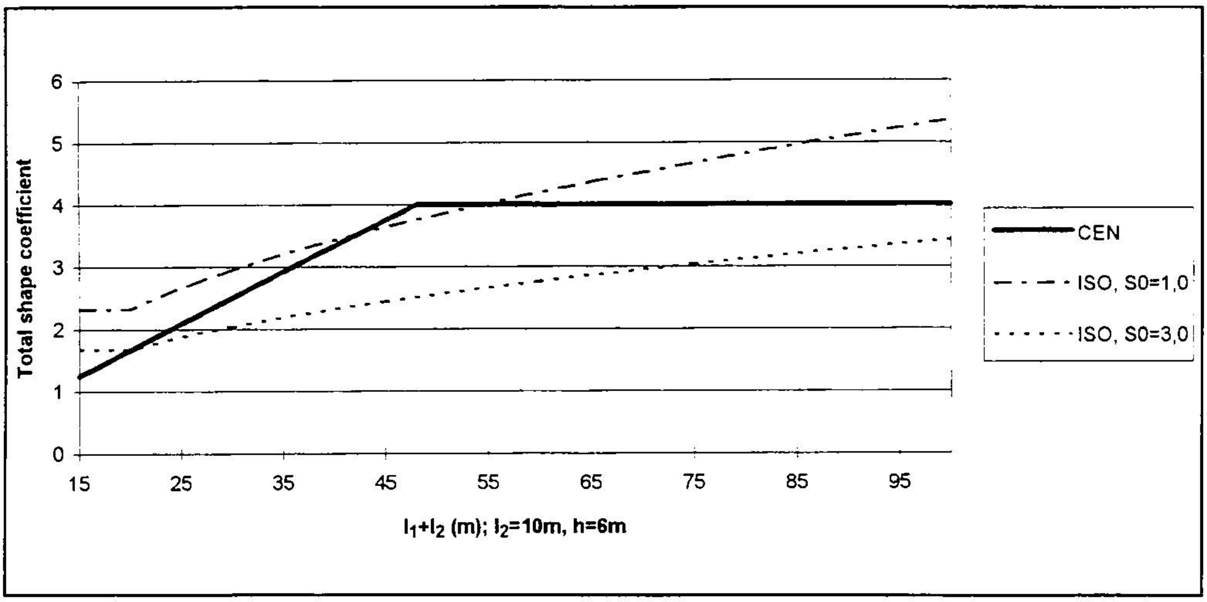

Figure 4

4. Action codes and reliability codes

When the design code is subdivided into governing reliability codes and action codes, it is the author's opinion that the action codes should be restricted to the prescription of the loads only, whereas the reliability codes should specify design situations and safety considerations.
The new ISO 4355, 1995, is as far as possible based on this principle.

However, the ENV 1991-2-3, classifies the snow load to be an accidental action under specified conditions. The author feels that this may lead to misunderstandings when applying the assembly of documents.

Firstly, classification of snow load as an accidental action was never suggested in ISO, and is not in accordance with the definition of the term. The consequences should be investigated carefully before a transformation to EN.

Secondly, if it is considered appropriate to treat snow load as an accidental action, this should be treated in the ENV 1991-1, Basis of design, rather than in ENV 1991-2-3.

5. Concluding remark

Several (important) differences, some of them with significant effects on the design snow load, are stated. It is felt that CEN ENV 1991-2-3: Actions on Structures - Snow loads should be examined as far as consequences are concerned, before the final EN-document is decided upon. In this connection some of the results of the new ISO 4355 should be studied in order to arrive at more harmonized documents in CEN and ISO..

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ICE RISK LEVELS - A PRACTICAL APPROACH TO DESIGN

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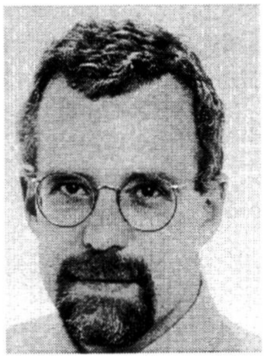

Ulrik Støttrup-Andersen, born 1950 got his degree in civil and structural engineering from the Technical University of Denmark in 1974. He is heavily engaged in national and international standardization, and he is chairman of the ISO Working Group for Atmospheric Icing of Structures.

Summary

The work with the elaboration of a new international standard for atmospheric iceloads on structures under the ISO have come up with a new term to be used world-wide for definition of atmospheric iceload on structure, namely "Ice Risk Level".
The ice risk levels are intendend to be used for instance for national mapping giving the ice load in various areas of a country.
The present paper briefly introduce the philosophy of the ISO Standard for Atmospheric Icing on Structures, the ice risk levels, examples of the ice accretion (shape and dimension) on various members and objects, wind drag coefficients for iced members, combination factors for wind and ice, etc.

1. Introduction

Atmospheric icing may have a great impact on the overall design and safety of various structures. The decisive load may be the increased vertical load due to the weight of accreted ice but quite often it is a combination of ice and wind, as the ice may increase the wind drag considerably. This is now commonly known and accepted in many national standards, but the big question is how much ice and how is it deposited onto various kind of structures and structural elements.
The work with elaboration of a new international standard for atmospheric iceloads on structures under the ISO (International Standardisation Organisation) have come up with a new term to be used for definition of atmospheric iceload on structure, namely "Ice Risk Level".
Two different categories of ice risk levels are defined, one for glaze and one for rime.
The ice risk levels are intended to be used for instance for national mapping giving the ice load in various areas of a country. A major problem faced by the ISO Working Group for Atmospheric Icing of Structures was primarily connected to rime ice accretion on non-circular, non-rotating objects and rime accretion on various shaped objects with large dimensions. After quite some reflection combined with full scale observations the Group have come up with a proposal for rime accretion on such members and objects.

The new ISO Standard will besides the ice risk levels and the models for ice accretion on various kind and shapes of objects also give rules for the ice accretion dependent on height above terrain, the shapes of ice vanes on fixed objects for the different ice risk levels, drag coefficients for wind drag on iced members, combination factors for combination of wind and ice, etc. necessary for the engineering design of structures exposed to atmospheric icing.
The present paper briefly introduce the philosophy of the ISO Standard for Atmospheric Icing on Structures, the ice risk levels, examples of the ice accretion (shape and dimension) on various members and objects, wind drag coefficients for iced members, combination factors for wind and ice, etc.

2. Types of Icing

Atmospheric icing is traditionally classified according to two different formation processes, "in-cloud icing" and "precipitation icing". Often is in-cloud ice called "rime" while precipitation ice often is devided into "glaze" and "wet snow". The physical properties and the appearance of the accreted ice will vary widely according to the variations of the meteorological conditions during the icing procedure. In Table 1 is given typical properties of atmospheric ice.

Type of ice	Density (kg/m^3)	Adhesion Cohesion	General Appearance	
			Colour	Shape
Glaze	900	Strong	Transept-rant	Evenly distributed/ icicles
Wet snow	300-600	Weak (forming) Strong (frozen)	White	Evenly distributed/ eccentric
Hard rime	600-900	Strong	Opaque	Eccentric, pointing windward
Soft rime	200-600	Low to Medium	White	Eccentric pointing windward

Table 1: Typical properties of accreted atmospheric ice.

Besides the properties mentioned in Table 1, other parameters such as compression strength, shear strength, etc. may be used to describe the nature of the accreted ice. For an engineering point of view it has been chosen to operate with two types of ice and simplified into "glaze" and "rime", and in the ISO Standard the practical application rules will be devided into rules for glaze and for rime.

3. Ice Risk Levels (IRL)

To be able to have a precise expression of the design ice load on structures at a certain location it is suggested to introduce the term "Ice Risk Level". Having the specific Ice Risk Level for a location the designer should have very valuable and important information for the assessment of the design atmospheric ice load on structures and combination of wind and ice. Ice Risk Levels are defined for glaze and for rime.
For glaze is the ice risk levels (IRL G) defined as the 50 years thickness of ice on a reference collector. In total is defined 5 levels, G1 to G5, starting with a thickness of 10 mm for level G1 and up to 40 mm for level G4. Ice risk level G5 is to be used for extreme glaze accretions, where the specific information should be given by a specialist.

IRL	Thick ness [mm]	Masses for Glaze on an object with diameters			
		10 mm	30 mm	100 mm	300 mm
G1	10	0,6 kg/m	1,1 kg/m	3,1kg/m	8,8kg/m
G2	20	1,7 kg/m	2,8 kg/m	6,8kg/m	18,1kg/m
G3	30	3,4 kg/m	5,1 kg/m	11,0kg/m	28,0kg/m
G4	40	5,7 kg/m	7,9 kg/m	15,8kg/m	38,5kg/m
G5	to be used for extreme ice accretions				

Table 2: Ice Risk Levels for Glaze (IRL G)

In Table 2 is given the definition of the five ice risk levels for glaze together with the weight of the glaze on a circular member with different diameter (density of the glaze: 900 kg/m^3).

IRL	Ice-mass [kg/m]	Ice diameter on an Ø30 mm collector Density of ice			
		300 kg/m^3	500 kg/m^3	700 kg/m^3	900 kg/m^3
R1	0,5	47 mm	7 mm	32 mm	28 mm
R2	0,9	63 mm	49 mm	42 mm	37 mm
R3	1,6	83 mm	65 mm	55 mm	49 mm
R4	2,8	109 mm	85 mm	72 mm	64 mm
R5	5,0	146 mm	113mm	96 mm	85 mm
R6	8,9	195 mm	151 mm	128 mm	113 mm
R7	16,0	261 mm	202 mm	171 mm	151 mm
R8	28,0	345 mm	267 mm	226 mm	199 mm
R9	50,0	461 mm	357 mm	302 mm	266 mm
R10	to be used for extreme ice accretions				

Table 3: Ice Risk Levels for Rime (IRL R)

The ice risk levels for rime (IRL R) are defined as the 50-years mass of ice per meter accreted on a standard collector. For rime is defined 10 levels starting with 0.5 kg/m for level R1 up to 50 kg/m for ice risk level R9. For extreme ice accretions to be treated individually by specialist level R10 should be used.

The standard collector is for both rime and glaze a 30 mm diameter cylinder rotating around its axis, and orientated perpendicular to the wind direction and 10 m above ground level. In Table 3 is given the defined 50 years ice masses for the ice risk levels for rime, as well as the outer diameter of the ice on a 30 mm cylinder for various densities of the rime.

4. Variation of Icing with Height above Terrain

The amount of atmospheric icing on a structure may vary with height above terrain, normally resulting in an increased ice load with increased height. If no site specific data are available the mass of rime dependent on the height above terrain may be found as

$$m_H = (H/10)^{0.25}\, m_{10}, \text{ where}$$

m_H is the mass at H meters above terrain
m_{10} is the mass at 10 meters above terrain

For glaze is normally assumed that the amount of accreted ice is independent of the height above terrain.

5. Icing on Members and Objects

The meteorological parameters together with the physical properties are influencing the sizes, shapes and weights of accreted ice on a given object.
The shape the size and the orientation of an object has especially a big influence on the accreted ice when it concerns rime, while glaze normally will have the same thickness independently on the shape of the member/object, see Figure 1.

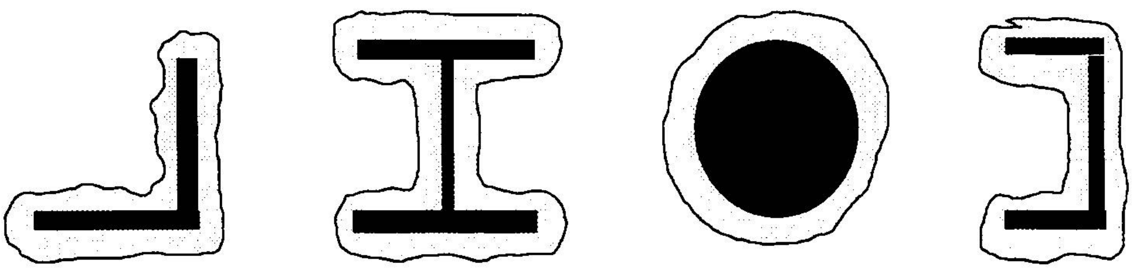

Fig. 1: Glaze on different members

The ISO Working Group for Atmospheric Icing on Structures has after quite some reflection combined with full scale observations come up with a proposal for rime accretion on various shaped members and objects.
For a fixed wind direction rime will accrete forming vaneshapes on profiles with a relatively small width perpendicular to the wind. On objects with larger widths the shapes will be more complex. Cylindrical accreted rime is normally only valid on slender elements with low

torrional stiffness and not sloping more than about 45° degrees to horizontal e.g. cables, mast guys - or on fixed nearly vertical members when the icing wind direction varies.
It is in the ISO Standard assumed that the mass of rime (50-years values) on objects with a width up to 300 mm is the same as defined by the ice risk level, i.e. independent on shape and width of the actual element, but the shape of the ice varies.
Besides theoretical reflections this has also been seen at full scale observations where vertical members of different shape and size has been observed during heavy icing periods. It was then seen that the amount of ice was nearly the same on all members independent on their shape and size.
When it comes to larger objects the constant mass is no longer valid. In the ISO Standard is given simple application rules for estimating the shape of the rime ice on various members and objects dependent on the ice risk levels.

Cross section shape of bars:		H		[		L	w		
Wind direction: →		L		L		L			
Object width [mm]		10		30		100		300	
IRL	Icemass	ICE VANES DIMENSIONS [mm]							
	G [kg/m]	L	W	L	W	L	W	L	W
R1	0,5	101	102	179	37	13	100	4	300
R2	0,9	124	104	197	42	23	100	8	300
R3	1,6	155	108	223	48	41	100	14	300
R4	2,8	196	113	260	57	102	113	24	300
R5	5,0	252	122	313	71	137	122	42	300
R6	8,9	327	135	385	89	191	135	76	300
R7	16,0	430	156	486	114	275	156	136	300
R8	28,0	560	185	614	146	390	185	321	343
R9	50,0	739	227	793	191	556	227	435	371
R10	*to be used for extreme ice accretions*								

Table 4: Ice dimensions for vane shaped rime on bars

In Table 4 is for typical cross sections of bars given the dimensions of the ice vanes for the various ice risk levels. The dimensions in the table are based on a density of the ice of 500 kg/m^3.
For larger objects the mass of ice per meter will be bigger than that on the standard collector.
In figures 2 and 3 is shown the principles for estimating the shape and the mass of rime on large objects.

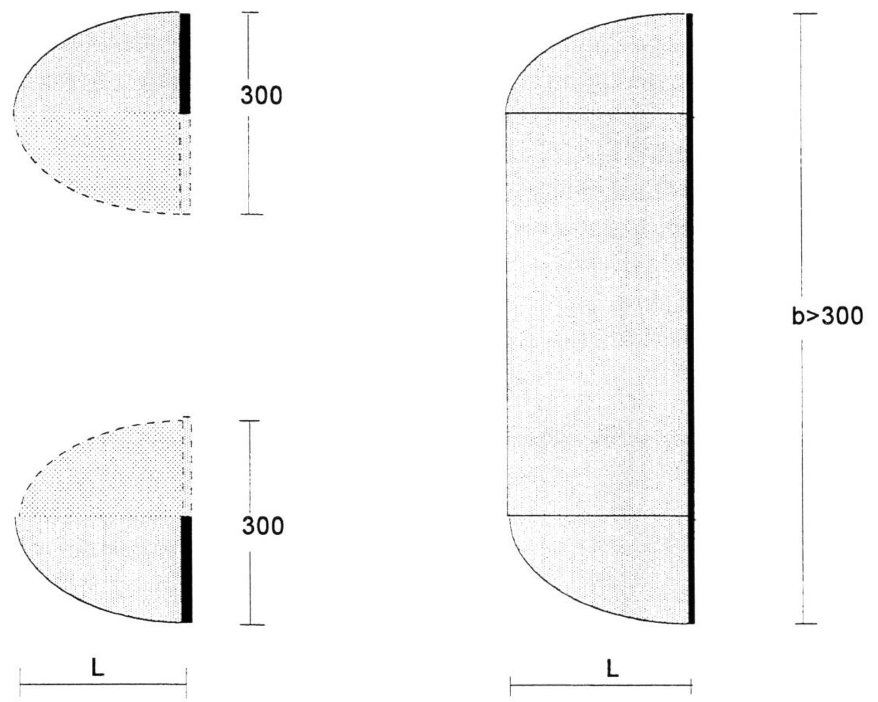

Fig. 2: Principle for estimation of ice on large flat objects

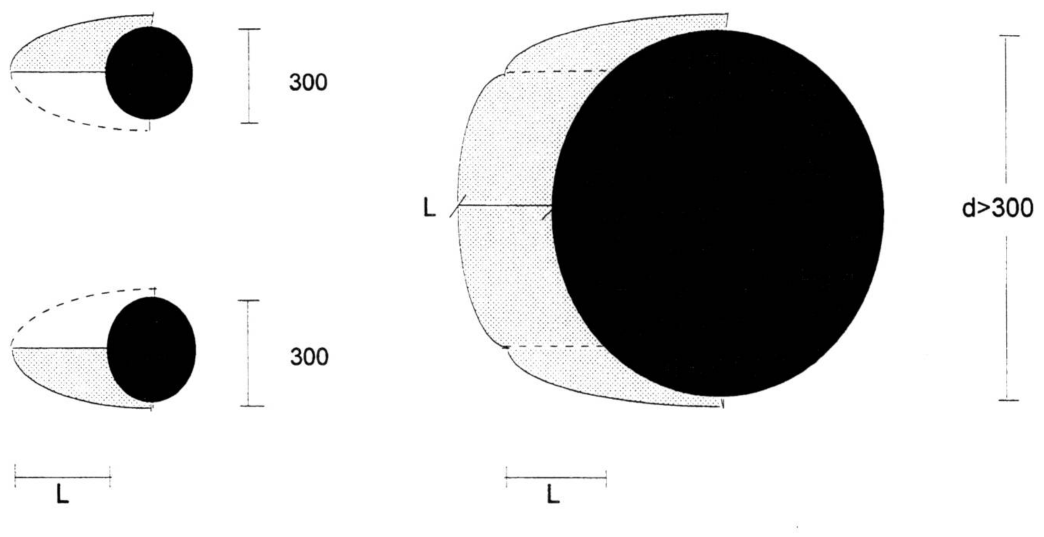

Fig. 3: Principle for estimation of ice on large round objects.

6. Wind and Ice

Besides the extra vertical loads due to the weight of the ice on the structure the icing will also increase the wind drag of the structure, and a combination of ice and wind may then govern the design. This is the case for instance for telecommunication masts and towers and for overhead transmission line towers in areas with reasonable ice loading.
In the ISO Standard is given recommendations for the estimation of wind drag coefficients for iced members and objects. The principle is based on factors to be used on the drag coefficient of the uniced members for the different ice risk levels. In Table 6 is as an example shown the factors for uniced members for rime.

IRL	Ice-mass [kg/m]	FACTOR ON THE DRAG COEFFICIENT					
		Drag coefficient without ice					
		1	1,2	1,4	1,6	1,8	2
R1	0,5	1,07	1,04	1,02	1,00	0,99	0,98
R2	0,9	1,13	1,07	1,03	1,00	0,98	0,96
R3	1,6	1,20	1,11	1,05	1,00	0,96	0,93
R4	2,8	1,27	1,15	1,06	1,00	0,95	0,91
R5	5,0	1,33	1,19	1,08	1,00	0,94	0,89
R6	8,9	1,40	1,22	1,10	1,00	0,93	0,87
R7	16,0	1,47	1,26	1,11	1,00	0,91	0,84
R8	28,0	1,53	1,30	1,13	1,00	0,90	0,82
R9	50,0	1,60	1,33	1,14	1,00	0,89	0,80
R10	to be used for extreme ice accretions						

Table 6: Drag coefficients for rime on bars.

When combining atmospheric iceload and wind load is normally combined the 50 year value of one load with a reduced value of the other load - for instance the one year value. Further the 50 year wind load is taken as the 50 year value of windload that may occur during icing periods. The reduction factor on the wind pressure to give the wind pressure in icing periods may be taken from Table 7 if no better information is available.

IRL G	Φ	IRL R	Φ
G 1	0,6	R 1	0,60
G 2	0,6	R 2	0,65
G 3	0,6	R 3	0,70
G 4	0,6	R 4	0,75
		R 5	0,80
		R 6	0,85
		R 7	0,90
		R 8	0,95
		R 9	1,00

Table 7: Reduction factor Φ on wind pressure to give the values in icing periods.

7. Concluding Remarks

The ISO Working Group for Atmospheric Icing on Structures plan to conclude the final draft for the standard in the Autumn 1996 so it will be ready for a voting by the Technical Committee TC 98 in the end of 1996.
Having hopefully finished the ISO Standard there still need to be undertaken a great job to implement the philosophy of the standard nationally as well as internationally. One of the most comprehensive tasks will be the creation of ice maps giving the actual Ice Risk Level for the various parts of the contries, in a similar way as for basic wind speeds and snow loads.
The members of the ISO Working Group contributes to the elabration of the standard without any financial support from ISO, and it is not realistic to include the elaboration of icemaps in the scope of the Working Group. The members of the Working Group may of course to a certain aspect assist national, European or other international organizations in the future implementation of the ISO Standard for Atmospheric Icing on Structures.

Revision of IS 2394
General Principles on Reliability for Structures

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Synopsis

The present IS 2394 which dates from 1986, has recently been rewritten and is ready for voting. A Table of Contents is presented here in Annex 1 of this paper.

It is of course interesting to compare this ISO-Draft with Eurocode 1, Basis of Design. Both documents have been written in the same period and, as far as Europe is concerned, partly by the same people (Gulvanessian, Leray, Ostlund and Vrouwenvelder were in both drafting panels).

The advantage of this panel overlap was that unnecessary and disturbing small differences between the two documents could be avoided. Some paragraphs even are completely identical.

Nevertheless there is a also a fundamental difference between the two documents. The main difference is that the ISO code is primarily of a conceptual nature where the Eurocode is more operational. As an example: the ISO code does not specify numbers for partial factors (γ factors) or load reduction factors (ψ factors).

A second typical distinction between the two documents is the explicit attention for probabilistic concepts in ISO. In this respect the new draft also differs from the 1986 version. In principle, all uncertainties and scatters encountered in the design process are basically considered from the probabilistic point of view. Topics like inherent versus statistical and model uncertainties and reliability targets are extensively discussed. In order to fulfil the reliability requirements two in principle equivalent design formats are presented:

- the probabilistic format, as discussed in chapter 6
- the partial factor format, as discussed in chapter 7

In the Eurocode only the partial factor method is presented. Only in the informative annex A the possibility of probabilistic methods as design method and as background for the partial factor method is mentioned.

One of the shortcomings of the ISO document, as mentioned before, is the lack of standardised data to help the designer to use the theoretical procedures. In this respect one might say that the present draft could not "replace" the present Eurocode 1, Basis of Design. However, this might only be a matter of time. The Joint Committee on Structural Safety is working on an operational Probabilistic Model Code, which exactly provides the missing information. In order to be prepared, it would be helpful if Eurocode 1 Basis of Design, would move already as far as possible into the direction of the new draft of IS 2394

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Annex A: Quality management and quality assurance
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