Reprint of paper, presented at the International Seminar on Steel Structures in Fire, Tongji University, Shanghai, P.R. China, November 1-3, 2001.
See note on page 7 for adjustment to final version of EC3-1.2.

THE NEW EUROCODE ON FIRE DESIGN OF STEEL STRUCTURES

Leen Twilt

Convenor of the Project Team on the conversion of ENV1993-1.2 to prEN1993-1.2
TNO, The Netherlands

ABSTRACT

In 1995 the part of Eurocode 3: “Design of Steel Structures” dealing with the fire design was published as a voluntary European standard: ENV1993-1.2. By the middle of 1999, work has started to convert this ENV into a harmonized technical specification (EN). Basis of the work is formed by the comments received on the ENV version from the various EU member states. A substantial part of the comments refers to technical issues, other comments are of an editorial nature. This paper concentrates on the technical comments. As a result of these technical comments, various modifications of ENV1993-1.2 which are proposed. Important topics in this respect are:

- deformation criteria under fire conditions
- material properties at elevated temperature
- fire design of connections
- classifcation of cross sections at elevated temperatures
- buckling curves and M-N interaction at elevated temperatures
- effect of non uniform temperature distribution on the mechanical resistance of beams
- calculation rules for thermal response of bare steel beams

The modifications regarding each of the above topics will globally be reviewed. For details, refer to the final draft of prEN1993-1.2 which is available by now and to background documents. References to these background documents are included in the paper.

KEYWORDS

Eurocode, steel structures, fire, thermal response, mechanical response, deformation criteria, shadow effect, buckling curves, M-N interaction, non uniform temperature distribution

1. INTRODUCTION

Part 1.2 of Eurcode 3 deals with the fire design of steel structures and was issued in September 1995 [1]. This part was, as all the other parts of the Eurocodes, published as an ENV, i.e. application within the European Union is on a voluntary basis only. The various EU Member States can, by means of so-called National Application Documents (NAD's), specify modifications, which then hold within the legislative system of that Member State. Many EU Member States have used this opportunity. Hence, the aim of the Eurocodes, i.e. harmonizing the rules for structural (fire) design is far from being achieved by their introduction.

By the middle 1999, the activities have started to convert the present Eurocodes from ENV's into EN's. This work is under responsibility of CEN TC250 and is more in particular guided by various Sub Committees, which have delegated the actual drafting activities to Project Teams. For the design of steel structures Sub Committee 3 (SC3) is in charge; preparation of the conversion text of the fire part (prEN1993-1.2) is undertaken by Project Team 2 (SC3/PT2)[1].

In the period between mid 1995 and mid 1999, practical experience has been obtained with the application of EC3-1.2. On the basis of this experience, 207 comments have been received from 19 different Member Sates and international branch organizations. About half of the comments refer to technical aspects of the ENV version of EC3-1.2; the other half to editorial matters. For the distribution of the technical comments over the main issues of the EC3-1.2, refer to Fig. 1.

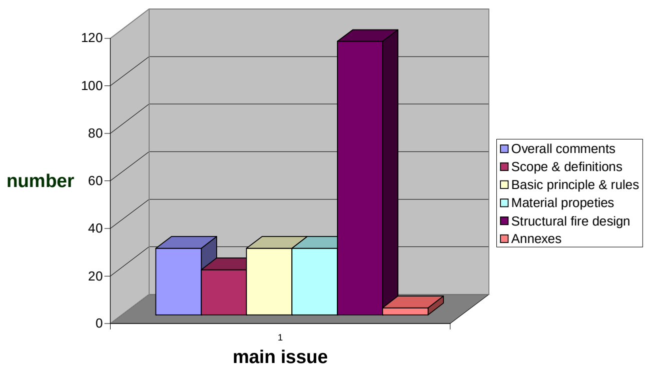

Figure 1: Comments per main issue

In this figure a separate category has been included for comments which are not related to one single aspect, the so-called "overall comments". One may conclude from Fig. 1, that by far most comments refer to the rules, specifically given for the Structural Fire Design (chapter 4 of ENV1993-1.2). Within this category, most attention (i.e. 85 %) is on the so-called "simple calculation rules" (section 4.2 of ENV1993-1.2). Important topics in this category are:

- connections;
- classification of cross sections;

---

[1] Membership of SC3/PT2: N. Andersen (DK), M. Fontana (CH), J-M. Franssen (B), D. Moore (UK), G. Sedlacek/ C. Heinemeyer – secretariat - (D), L. Twilt – convenor – (NL).

- buckling curves and M-N interaction;
- effect of non uniform temperature distribution on the mechanical resistance of beams (κ_1-factor);
- calculation rule for the thermal response of bare steel beams.

Topics of a more general nature, which have been strongly commented upon, include:
- deformation criteria;
- material properties.

In this paper, the main modifications of the ENV version of EC3-1. will be discussed in a global manner. For details, refer to the draft of the EN version [2], which will become available by the end of 2001 and to the accompanying background documents. In the text of the paper reference will be made to these background documents, where appropriate.

The discussion on the above topics is embedded in the structure chosen for EC3-1.2:
- **General**, dealing with formal aspects such as scope, definitions etc.
- **Basic principles and rules**, dealing with more general aspects such as deformation criteria etc;
- **Material properties**, both thermal and mechanical;
- **Simple calculation models**, as "main stream" for the structural fire design, dealing with component design (beams, columns etc.)

The above subdivision corresponds - by and large - with the various sections of EC3-1.2. With a view to provide some overall insight in the set up of EC3-1.2, the intent of each of the sections will be reviewed in general terms, before commenting on the anticipated modifications resulting from the conversion process

2. GENERAL

Section 1 of EC3-1.2 (General) contains:
- scope
- normative references
- basic assumption
- definitions
- symbols

Most of the clauses given are so-called "model clauses", i.e. they are identical in the various fire parts of Eurocode (concrete, steel, timber etc.). The Horizontal Group Fire (HGF)[2] is responsible for harmonizing the model clauses. As far as EC3-1.2 is concerned, the main deviation from the model clauses is due to the fact that for the fire design of steel structures the separating function is not taken into account.

Compared to the ENV version of EC3-1.2 no significant modifications are forseen.

---

[2] Membership HGF: J. Kruppa – convenor - (F), together with the convenors of the various Project Teams, responsible for the conversion of the Eurcode parts "Fire"

3. BASIC PRINCIPLES AND RULES

Section 2 of EC3-1.2 (Basic principles and rules) contains:

- performance requirements;
- actions;
- design values of material properties;
- assessment methods.

Again, most of the clauses in this chapter are “model clauses”, i.e. clauses which are identical for all the “fire” parts of the Eurocode (see paragraph 2 above).

The main modification in the EN compared to the ENV version concerns the deformation criteria. In the ENV version, a check on deformations was required for steel members with fire insulation and/or for members, the deformation of which might endanger the functioning of adjacent separating elements (e.g. partitions), if “the deformation of the load bearing structure requires consideration”. When using – in simple calculation models - a reduced strength reduction factor ($k_{x,\theta}$, based on a strain of 0.5%), it is not necessary to check the deformation criteria explicitly.

From the comments received, it became clear that there was major concern with this approach. Important arguments against were (1) the open end formulation (2) lack of functional background and (3) exceptional position for steel, since for other structural materials (concrete, timber etc.) such rules do not exist. Therefore, SC3-PT2 paid due attention to the deformation criteria. As a first step, a more functional definition for $k_{x,\theta}$ was derived, based on a maximum strain 1% [3]. In Fig. 2, the strength reduction factor $k_{y,\theta}$, to be applied if deformation criteria need not to be satisfied as well as the new $k_{x,\theta}$ factor are presented, together with the $k_{x,\theta}$ as specified in ENV1993-1.2.

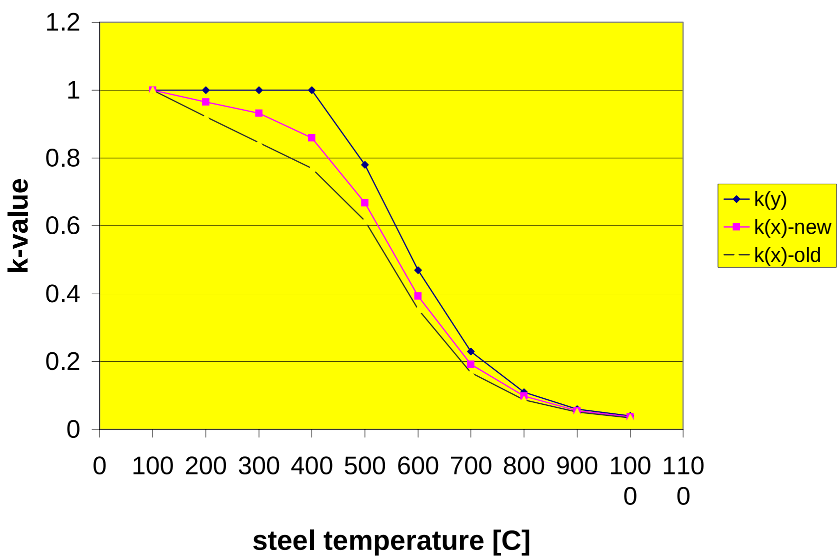

Figure 2: Strength reduction factors as function of the steel temperature

It can be shown that the effect of changing from $k_{y,\theta}$ (no deformation criteria to be satisfied) to $k_{x,\theta\text{-new}}$ (deformation criteria to be satisfied) in terms of fire resistance time is only marginal [4]. For this reason – and also considering the other arguments mentioned above – it has been decided to refrain from a check on deformation if the separating elements and/or the protection system (if any) are designed according to the CEN classification system & rules.

The other modifications mainly concern "stream lining" of the text and do not actually have a big technical impact. This means, for example, that the EN version of EC3-1.2 still offers three levels for assessing the structural behaviour of fire exposed steelwork:
- member analysis;
- analysis of the part of the structure;
- global structural analysis.

However, in view of the fact that member analysis is most often used in practice, emphasis is on this option.

4. MATERIAL PROPERTIES

In section 1 of EC3-1.2 both the mechanical and thermal properties of steel to be used in the structural fire safety design are specified. As far as the properties of the fire protection materials are involved, reference is made to the relevant CEN standard [5] (as in the ENV version of EC3-1.2).

New - compared to the ENV version of EC3-1.2 - is that the mechanical properties are now not only given for carbon steel, but for stainless steel as well. The latter information is presented in a separate Annex and is based on an extensive European research project, which was recently completed [6].

The mechanical properties of carbon steel are the same as specified earlier in the ENV version of EC3-1.2. In here, the factors for strength reduction and Young's modulus reduction are presented by both tabulated data (derived from tests) and equations. The latter are approximations of the tabulated data and consequently there is no exact match. This is illustrated in fig. 3, where the error, when using the equation is plotted against the steel temperature. Especially for steel temperatures beyond 700 °C the error is rather large. Since unambiguous formulations are not allowed in code text, it has been decided to delete the equations. The strength reduction factor for satisfying deformation criteria is still there, however – as explained earlier – on an informative basis only.

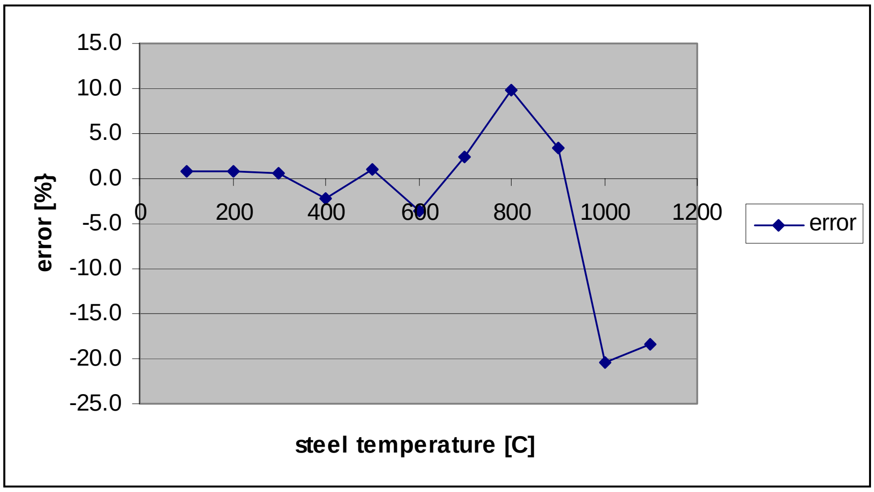

Figure 3: Error when for the stress reduction factor the equation according to [1] is used in stead of the tabulated data

Also the thermal properties of carbon steel are unchanged compared the ENV version of EC3-1.2. However, the time independent design values for the thermal elongation, the specific heat and the thermal conductivity have been deleted, since the widespread use of computers makes this simplification unnecessary.

5. SIMPLE CALCULATION MODELS

5.1 Introduction

The simple calculation models are dealt with in section 4.2 of EC3-1.2 (Structural fire design), and form by far the most extensive part of this code. Prerequisite for the simple calculation models is standard fire exposure and member analysis. The following topics are covered:

- general guidance, e.g. with respect to the **fire design of connections**;
- **classification of cross-sections**
- mechanical resistance of
  - tension members;
  - **compression members** (cross section classes 1, 2, 3);
  - beams, including the **effects of non-uniform temperature distribution** (cross section classes 1, 2, 3);
  - members under **combined bending and axial compression** (cross section classes 1, 2, 3);
  - members with cross section class 4**;**
- steel temperature development for:
  - **unprotected internal steelwork;**
  - internal steelwork insulated by fire protection material;
  - internal steelwork that is protected by heat screens;
  - external steelwork.

Topics which are subject to substantial modifications compared to the ENV version of EC3-1.2 are printed in bold in the above list. They will briefly be discussed hereafter.

5.2 Fire design of connections

The ENV version of EC3-1.2 specifies that the resistance of connections need not to be checked, provided the thermal resistance of the fire protection of the connection is not less than the minimum value of the thermal resistance of the fire protection of any of the steel members joint to that connection. The thermal resistances are defined by the ratio between the appropriate values of the insulation thickness (d_f) and the thermal conductivity (λ_f).

In the EN version this rule is - for reasons of consistency - extended with an additional condition: utilization should be less than the maximum value of any of the connected members. As a simplification, the comparison of the level of utilization within a connection may be performed as for room temperature.

As an alternative for the fire design based on the above simple rules, the fire resistance of connections may be determined using a method specified in Annex D (informative only) of the EN version. This calculation method holds both for bolted (shear/tension) and welded connections (butt & fillet welds) and is based on an assessment of the mechanical resistance of the connection and its temperature response, when exposed to fire. The method is based on [7].

5.3 Classification of cross sections

In the ENV version of EC3-1.2 cross sections of beams are classified by means of the parameter:

$$\varepsilon = [(235/f_y)(k_{E,\theta}/\ k_{y,\theta})]^{0.5}$$

with:

f_y	is	yield stress at room temperature;
$k_{y,\theta}$, $k_{E,\theta}$	is	strength reduction and Young's modulus reduction factor respectively.

The parameter ε was simply obtained by substituting elevated temperature values for yield strength and Young's modulus in the equations used for room temperature classification of the cross section. The variation of $(k_{E,\theta}/\ k_{y,\theta})^{0.5}$ as function of temperature shows an extremely irregular behaviour. See Fig. 4. As a result, the ENV classification rule gives strange result: starting with class 1, a cross section can move into class 2 or 3 and back. Since there is no evidence on the accuracy of the ENV rule and – also - the application of such a rule hampers the practical design procedure significantly, SC3/PT2 proposes to retain classification as in cold design, i.e. independent of the temperature.[3]

[3] Note: Further to discussions within SC3, borderline values for b/t are based on $\varepsilon = 0{,}85(235/f_y)^{0{,}5}$, thus creating a "safety margin" for classifying cross sections of beams at elevated temperature. See EC3-1.2: 4.2.2 (1).

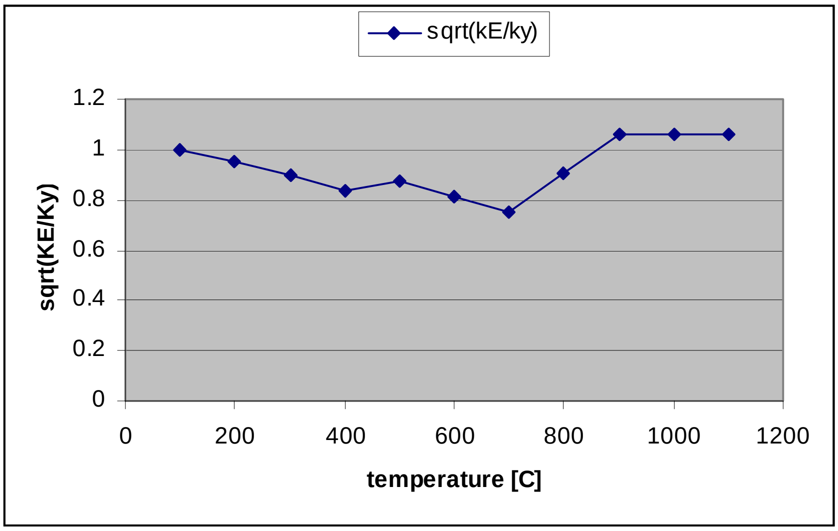

Figure 4 : Variation of the factor $(k_{E,\theta}/ k_{y,\theta})^{0.5}$ as function of the steel temperature

5.4 Mechanical resistance of compression members

The present EC3-1.2 rules for the mechanical resistance of compression members (buckling curves at elevated temperatures), are based on buckling curve c at room temperature (irrespective of the type of cross section) in combination with a reduced yield stress, by applying the strength reduction factor $k_{y,\theta}$. As input parameter, a non dimensional, temperature dependent slenderness ratio ($\bar{\lambda}_\theta$) is used, following from:

$$\bar{\lambda}_\theta = \bar{\lambda}\,[k_{y,\theta} / k_{E,\theta}]^{0,5}$$

with:

$\bar{\lambda}$	is	non dimensional slenderness ratio at room temperature
$k_{y,\theta}$, $k_{E,\theta}$	is	strength reduction and Young's modulus reduction factor respectively

In the mean time extensive research, sponsored by the European Commission for Coal and Steel (ECSC) has been carried out in this field [8], [9]. As a result, new buckling curves at elevated temperatures could be established. In this approach, the dimensionless slenderness ratio, as specified in the ENV version of EC3-1.2, is maintained, i.e. temperature dependent. An important advantage of the new rule is that it is consistent with the room temperature design. Also, somewhat less conservative results are obtained, when compared to the ENV version of EC3.1. Refer to Fig. 5 for a comparison between the "old" and the "new" buckling curves. For background information, see [8].

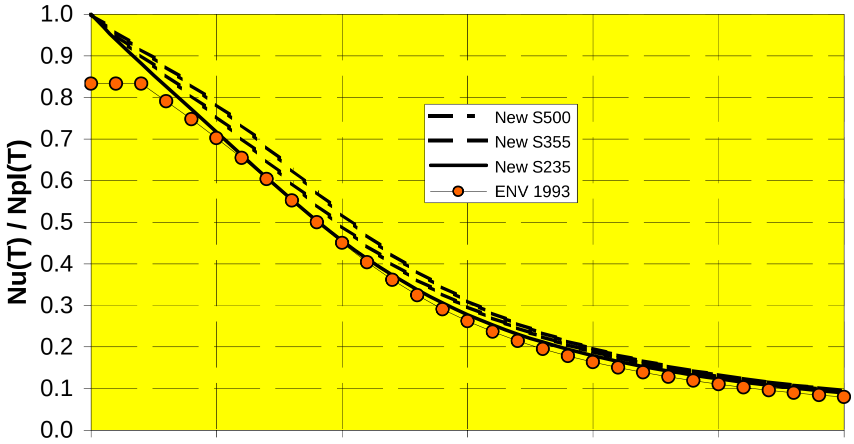

Figure 5: Comparison between the present buckling curves at elevated temperature and the new proposal

5.5 Mechanical resistance of beams with a non uniform temperature across the cross section of the beam

When using simple calculation models for the determination of the mechanical resistance of beams, the ENV version of EC3-1.2 offers the possibility to assume a uniform temperature distribution across the cross section of the beam. Especially for three side exposed beams (e.g. beam with a concrete floor above) the temperature distribution will – in reality - be far from uniform. This is corrected for by introducing an adaptation factor κ_1 smaller than unity, by which the resistance has to be divided[4]. The value of κ_1 depends on the way of exposure. In ENV1993-1.2 the following is specified:
for a beam exposed on all four sides: $\kappa_1 = 1.0$
for a beam exposed on three sides, with a slab (concrete or composite) above: $\kappa_1 = 0.7$

A wide variety of comments have been received with regard to the κ_1-value. Suggested values vary from 0.7 (as in ENV1993-1.2) to 1.0 (completely ignoring the effect of the non-uniform temperature distribution).

The Project Team has studied this issue in detail, basing itself on both experimental and theoretical work, carried out in the past [10], [11], [12]. From this study, it became clear the experimental evidence supporting a κ_1-value of 0.7 only exists for bare steel beams. See Fig. 6.

[4] To account for a possible non-uniform temperature distribution along a beam, an adaptation factor κ_2 is introduced in EC3-1.2. Since κ_2 will not be modified as result of the conversion process, it will not be discussed here.

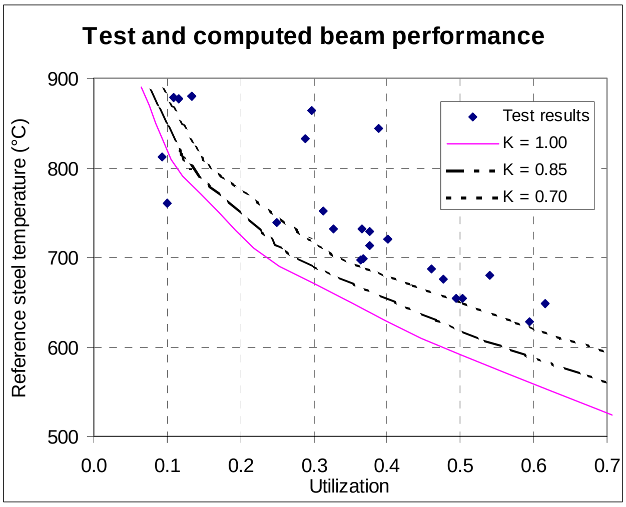

Figure 6: Test results and various options for the κ_1 factor for unprotected beams

For fire insulated steel beams, no such evidence is available. However, the results of theoretical studies suggest for these cases a κ_1-value of 0.85 for practical load levels. See Fig. 7. For this reasons SC3-PT2 proposes to change the κ_1-values in the EN version of EC3-1.2 for 3 side exposed beams accordingly. For 4 side exposed beams the situation remains the same, i.e. $\kappa_1 = 1.0$. See also [13].

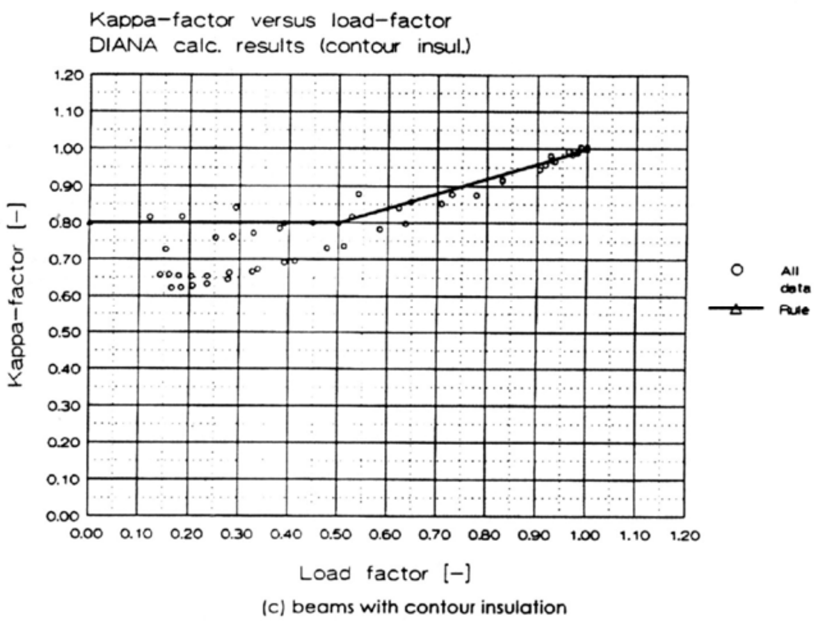

Figure 7: The κ_1-factor as function of the utilization factor for steel members protected by contour insulation (calculation by means of an advanced computer model) [4]

5.6 Combined bending and axial compression

The rules on combined bending and axial compression (M-N interaction) included in the ENV version of EC3-1.2 are based on research carried out in mid nineties [14]. There is no reason to doubt its validity [14]. The rules are formulated in conjunction with the rules for room temperature conditions specified part 1.1 of EC3 [15]. The complication now is that the latter rules will be modified, but that – by the middle of 2001 – it was not yet known which of the available options would be chosen. In view of the tough time schedule, and also for budgetary reasons, the Project Team has decided to stick to the room temperature rules according to ENV 1993-1.1. Since these rules will not be available anymore in the EN version of part 1.1, they will be incorporated in part 1.2 of the EN version. Note that the this modification has no effect what so ever in terms of calculation results.

5.7 Calculation rules for temperature development in unprotected internal steelwork

Calculation rules for the temperature development in bare (i.e. unprotected) steelwork specified in the ENV version of EC3-1.2 are based on conventional values for the coefficients of both the radiative and convective heat transfer. These values are chosen such that reasonable agreement with test results is obtained, leading however to assumptions which are – from a physical point of view – not very convincing. This particularly holds for the radiative heat transfer, where rather unrealistic values for the emissivity coefficients for both the steel surface and the fire environment have to be assumed: a value for the resultant emissivity as low as 0.5 is necessary in order to achieve a reasonable match with test results. This problem has become even more explicit when introducing the so-called Plate Thermometer (in stead of the common thermocouples) as measuring device for controlling the gas temperature during standard fire resistance testing [16], [17].

With a view to arrive at more realistic and consistent calculation rules for the temperature development in bare steel members and also to stay in line with future standard fire resistance testing practice, the Project Team proposes to use more realistic values for the emissivity coefficients of steel ($\epsilon_a = 0.7$; being a low, but realistic value) and the fire environment ($\epsilon_{fi} = 1.0$; as direct consequence of using the Plate Thermometer for furnace control [16]) .

The "uplifting effect" in terms of calculated temperatures of these modifications is – by and large - compensated by taking into consideration the so-called "shadow effect", which is not explicitly taken into account in the present calculation rules. Assuming fully embedded members (as in the case of simple calculation models), the shadow effect is caused by local shielding of the radiation, due the shape of the steel profile. It plays a role for profiles with a concave shape, such as I-sections; for profiles with a convex shape, such as tubes, it does not exist (no local shielding).

The increase of temperature $\Delta\theta_{a,t}$ in an unprotected steel member during a time interval Δt may then be determined from:

$$\Delta\theta_{a,t} = k_{shadow} \frac{A_m / V}{c_a \rho_a} \dot{h}_{net,d} \Delta t$$

with:

k_{shadow}	is	correction factor for the shadow effect
A_m / V	is	the section factor for unprotected steel members;

A_m	is	the surface area of the member per unit length;
V	is	the volume of the member per unit length;
c_a	is	the specific heat of steel [J/kgK];
$\dot{h}_{net,d}$	is	the design value of the net heat flux per unit area, with $\epsilon_a = 0.7$ and $\epsilon_{fi} = 1.0$ [W/m^2];
Δt	is	the time interval [seconds];
ρ_a	is	the unit mass of steel, from section 3 [kg/m^3].

New in the expression – compared to the ENV version of EC3-1.2 – is the correction factor k_{shadow} for the shadow effect[5]. It can be shown that the shadow effect is reasonably well described by taking

$$k_{shadow} = 0.9\ [A_m/V]_{box}/[A_m/V]$$

with

$[A_m/V]_{box}$ is box value of the section factor

In Fig. 8, temperatures, calculated on the basis of the above new rule (including the shadow effect & realistic values for the heat transfer coefficients) have been compared with values calculated on the basis of the "old" rule (i.e. no shadow effect & heat transfer coefficients according to the ENV version of EC3-1.2). The temperatures have been calculated for various times of standard fire exposure and for all listed European I sections (IPE, HEA, HEB etc.).

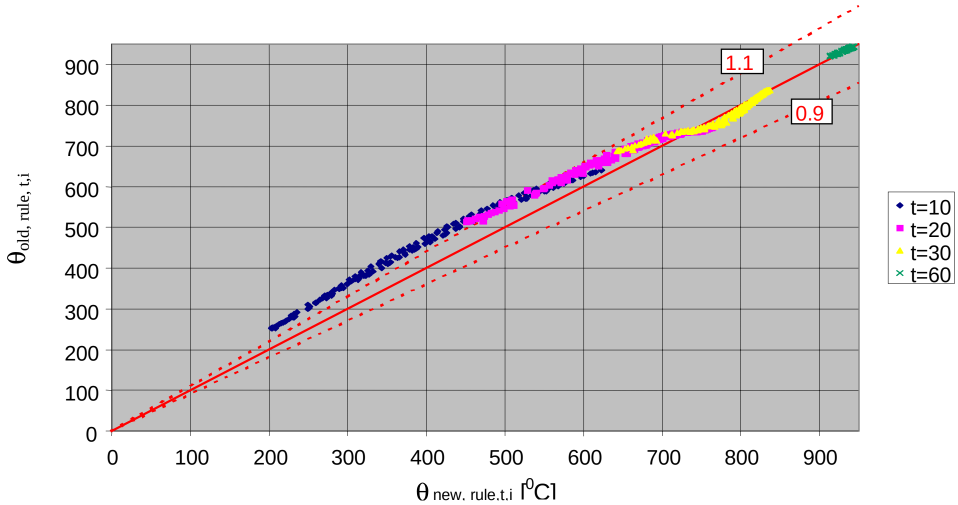

Figure 8: Steel temperatures in bare internal steelwork, calculated according to the present rule of EC3-1.2, compared with calculation results according to the proposed new rule.

[5] The correction factor makes no distinction between radiative and convective heat flux. It is clear that convective heat transfer is less affected by a shadow effect than radiation is, This is ignored since convection plays only a minor role over the heating period.

It follows from the definition of k_{shadow} that for tube profiles, the shadow effect is not activated, since $[A_m/V] = [A_m/V]_{box}$

Note that for protected members, the shadow effect is less relevant since the effect of the fire insulation overrules that of the heat transfer at the surface of the member. In EC3-1.2 the latter effect is – for protected members – even completely ignored.

6. CONCLUSIONS

Under responsibility of Sub Committee 3 of CEN TC250, Project Team 2 has worked on the conversion of the Eurocode on the Fire Design of Steel Structures (ENV1993-1.2) from an ENV to an EN. In the scope of this work more than 100 comments with regard to technical aspects of ENV1993-1.2 have been evaluated and – where appropriate – implemented. A similar number of editorial comments have been dealt with.

The conversion process has lead to various modifications with respect to the technical contents of ENV1993-1.2. Many of these modifications were possible because, since the publication of ENV1993-1.2 in 1995, results of new, mainly European research, came available. Also a better harmonisation between the various fire parts of Eurocode (concrete, steel, timber etc.) has been achieved.

The work on the conversion is nearly completed: a draft version of prEN1993-1.2 is available and will shortly be discussed within Sub Committee 3. The formal voting is expected in the course of 2002.

ACKNOWLEDGEMENT

The author wishes to acknowledge the significant contributions of his fellow members in Project Team 2, responsible for preparing the conversion of ENV1993-1.2 into an EN. Without these contributions the conversion process would not have reached its present stage.

REFERENCES

[1] ENV1993-1.2: "Eurocode 3 – Design of steel structures – Part 1.2: General rules – Structural fire design".
CEN, September 1995.

[2] prENV1993-1.2: "Eurocode 3 – Design of steel structures – Part 1.2: General rules – Structural fire design", draft for voting.
CEN, September 2001.

[3] Andersen, N.: "Background for the deformation criteria and $k_{x,\theta}$".
SC3/PT2 doc. N024 (rev. 1), January 200.

[4] Twilt, L.: "Deformation criteria for fire exposed steelwork".
SC3/PT2 doc. N063(NL), November 2000.

[5] prENV13381-4: "Test method for determining the contribution to the fire resistance of structural members: by applied protection to structural steel elements".

CEN TC127, september 2001.
[6] Zhao, B.: "Development of the use of stainless steel in construction; Work Package 5.1: Material behaviour at elevated temperatures".
ECSC contract 7210-SA/842 etc., March 2000.
[7] Latham, D.J. and Kirby, B.R.: "Elevated Temperature Behaviour of Welded Joints in Structural Steelwork".
ECSC contract 7210.SA/824 (F6.3/90).
[8] Talamona, D., Franssen, J.M., Schleich, J.B. and Kruppa, J.: "Stability of Steel Columns in Case of Fire: Numerical Modelling".
Journal of Structural Engineering, June 1997.
[9] Franssen, J.M., Talamona, D., Kruppa, J. and Cajot, L.G: "Stability of Steel Columns in Case of Fire: Experimental Evaluation".
Journal of Structural Engineering, February 1998.
[10] Wainman, D.E. and Kirby, B.R.: "Compendium of UK Standard Fire Test Data: Unprotected Structural Steel – 1 & 2".
British Steel Corporation, Swinden Laboratories, 1988.
[11] Twilt, L. and Both, C.: "Technical Notes on the Realistic Behaviour and Design of Fire Exposed Steel and Composite Steel-Concrete Structures".
ECSC project SA112, 1991.
[12] Kruppa, J., Twilt, L., Wesche, J. and Cooke, G.: "Fire Protection of Structural Steelwork".
ECSC project SA 314, 1995.
[13] Twilt, L.: "Value for κ_1 to be taken into account in prEN1993-1.2".
SC3/PT2 doc. E085, April 2001.
[14] Talamona, D.: "Flambement de poteaux méttalique sous charges excentreés à haute température".
PhD thesis, Univ. Blaise Pascal, Clermont Ferrand, France, 1995.
[15] ENV1993-1.1: Eurocode 3– Design of steel structures – Part 1.1: General rules – General rules and rules for buildings".
CEN, 1995.
[16] Leur, P.H.E. and Twilt, L.: "Thermal Exposure in Fire Resistance Furnaces".
Proceedings of 6th IAFSS conference held in Poitier, June 5-9, 1999.
[17] Twilt, L., Leur, P.H.E. v.d., and Both, C.: "Characteristics of the heat transfer for calculating the temperature development in structural steelwork exposed to standard fire conditions under plate thermocouple control".
Proceedings of the first international workshop "Structures in Fire", hold in Copenhagen June 19 and 20, 2001, September 2000.