

# Sample Chapter

*Fire Design of Steel Structure*

*EC1: Actions on structures. Part 1-2: Actions exposed to fire.*

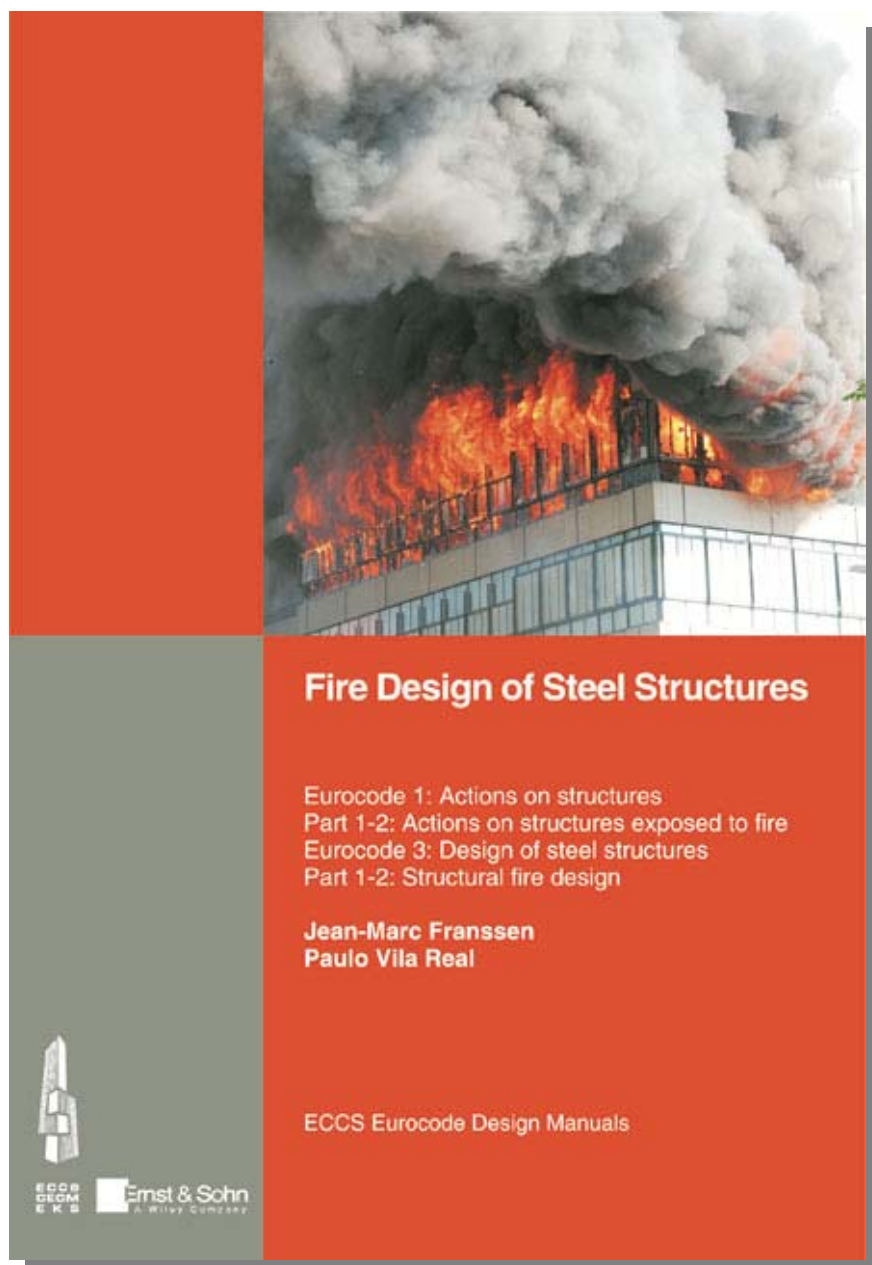
*EC3: Design of steel structures. Part 1-2: Structural fire design.*

Editor: ECCS – European Convention for Constructional Steelwork / Associacao

Portuguesa de Construcao Metalica e Mista

Copyright © 2010 Ernst & Sohn, Berlin

ISBN: 978-3-433-02974-9



Wilhelm Ernst & Sohn  
Verlag für Architektur und  
technische Wissenschaften  
GmbH & Co. KG  
Rotherstraße 21, 10245 Berlin  
Deutschland  
[www.ernst-und-sohn.de](http://www.ernst-und-sohn.de)

**Ernst & Sohn**  
A Wiley Company

---

# **FIRE DESIGN OF STEEL STRUCTURES**

---

## **ECCS EUROCODE DESIGN MANUALS**

### **ECCS EDITORIAL BOARD**

*Luís Simões da Silva (ECCS)*

*António Lamas (Portugal)*

*Jean-Pierre Jaspart (Belgium)*

*Reidar Bjorhovde (USA)*

*Ulrike Kuhlmann (Germany)*

### **DESIGN OF STEEL STRUCTURES**

*Luís Simões da Silva, Rui Simões and Helena Gervásio*

### **FIRE DESIGN OF STEEL STRUCTURES**

*Jean-Marc Franssen and Paulo Vila Real*

### **AVAILABLE SOON**

---

### **DESIGN OF COLD-FORMED STEEL STRUCTURES**

*Dan Dubina, Viorel Ungureanu and Raffaele Landolfo*

### **DESIGN OF PLATED STRUCTURES**

*Darko Beg, Ulrike Kuhlmann, Benjamin Braun and Laurence Davaine*

### **DESIGN OF CONNECTIONS IN STEEL AND COMPOSITE STRUCTURES**

*Jean-Pierre Jaspart*

### **FATIGUE DESIGN OF STEEL AND COMPOSITE STRUCTURES**

*Alain Nussbaumer, Luís Borges and Laurence Davaine*

### **INFORMATION AND ORDERING DETAILS**

---

For price, availability, and ordering visit our website [www.steelconstruct.com](http://www.steelconstruct.com).

For more information about books and journals visit [www.ernst-und-sohn.de](http://www.ernst-und-sohn.de).

---

---

# **FIRE DESIGN OF STEEL STRUCTURES**

**Eurocode 1: Actions on Structures**

**Part 1-2 – General actions – Actions on  
structures exposed to fire**

**Eurocode 3: Design of Steel Structures**

**Part 1-2 – General rules – Structural fire design**

**Jean-Marc Franssen**

**Paulo Vila Real**



**ECCS  
CECM  
EKS**



**Ernst & Sohn**  
A Wiley Company

# TABLE OF CONTENTS

<b>FOREWORD</b>	xiii
<b>PREFACE</b>	xv
<b>NOTATIONS</b>	xvii

## Chapter 1

<b>INTRODUCTION</b>	<b>1</b>
1.1. Relations between different Eurocodes	1
1.2. Scope of EN 1993-1-2	3
1.3. Layout of the book	3

## Chapter 2

<b>MECHANICAL LOADING</b>	<b>7</b>
2.1. General	7
2.1.1. General rule	7
2.1.2. Simplification 1	10
2.1.3. Simplification 2	10
2.1.4. Simplification 3	12
2.2. Examples	12
2.3. Indirect actions	14

## Chapter 3

<b>THERMAL ACTION</b>	<b>17</b>
3.1. General	17
3.2. Nominal temperature-time curves	18
3.3. Parametric temperature-time curves	21
3.4. Zone models	29

---

---

**TABLE OF CONTENTS**

---

3.5. CFD models	31
3.6. Localised fires	32
3.7. External members	39

**Chapter 4****TEMPERATURE IN STEEL SECTIONS** **45**

---

4.1. Introduction	45
4.2. The heat conduction equation and its boundary conditions	45
4.3. Advanced calculation model. Finite element solution of the heat conduction equation	47
4.3.1. Temperature field using the finite element method	48
4.4. Section factor	51
4.5. Temperature of unprotected steelwork exposed to fire	54
4.6. Temperature of protected steelwork exposed to fire	61
4.7. Internal steelwork in a void protected by heat screens	77
4.8. External steelwork	78
4.8.1. General principles	78
4.8.2. Example	80
4.9. View factors in the concave part of a steel profile	88
4.10. Temperature in steel members subjected to localised fires	91
4.10.1. Unprotected steel members	91
4.10.2. Protected steel members	93
4.11. Temperature in stainless steel members	94
4.11.1. Example	97

**Chapter 5****MECHANICAL ANALYSIS** **99**

---

5.1. Basic principles	99
5.2. Mechanical properties of carbon steel	104
5.3. Classification of cross-sections	109
5.4. Fire resistance of structural members	118
5.4.1. General	118

---

5.4.2. Tension members	120
5.4.3. Compression members	121
5.4.4. Shear resistance	124
5.4.5. Laterally restrained beams	127
5.4.5.1. <i>Uniform temperature distribution</i>	127
5.4.5.2. <i>Non-uniform temperature distribution</i>	128
5.4.5.3. <i>Bending and shear</i>	131
5.4.6. Laterally unrestrained beams	133
5.4.6.1. <i>The elastic critical moment for lateral-torsional buckling</i>	133
5.4.6.2. <i>Resistance to lateral-torsional buckling</i>	137
5.4.7. Members with Class 1, 2 or 3 cross-sections, subjected to combined bending and axial compression	140
5.4.8. Members with Class 4 cross-sections	143
5.4.9. Some verifications of the fire resistance not covered by EN 1993-1-2	143
5.4.9.1. <i>Shear buckling resistance for web without intermediate stiffeners</i>	147
5.4.9.2. <i>Cross section verification of a member subjected to combined bending and axial force (compression or tension)</i>	145
5.4.9.2.1. <i>Class 1 and 2 rectangular solid sections</i>	146
5.4.9.2.2. <i>Class 1 and 2 doubly symmetrical I- and H-sections</i>	147
5.4.9.3. <i>Bending, shear and axial force</i>	149
5.5. Design in the temperature domain. Critical temperature	149
5.6. Design of continuous beams	160
5.6.1. General	160
5.6.2. Continuous beams at room temperature	161
5.6.3. Continuous beams under fire conditions	164
5.7. Fire resistance of structural stainless steel members	166
5.8. Design examples	173

---

## TABLE OF CONTENTS

---

### Chapter 6

#### **ADVANCED CALCULATION MODELS** **235**

---

6.1. General	235
6.2. Thermal response model	237
6.3. Mechanical response model	244

### Chapter 7

#### **JOINTS** **251**

---

7.1. General	251
7.2. Strength of bolts and welds at elevated temperature	252
7.3. Temperature of joints in fire	254
7.4. Bolted connections	255
7.4.1. Design fire resistance of bolts in shear	255
7.4.1.1. <i>Category A: Bearing type</i>	255
7.4.1.2. <i>Category B (slip resistance at serviceability) and Category C (slip resistance at ultimate state)</i>	256
7.4.2. Design fire resistance of bolts in tension	256
7.4.2.1. <i>Category D and E: Non-preloaded and preloaded bolts</i>	256
7.5. Design fire resistance of welds	256
7.5.1. Butt welds	256
7.5.2. Fillet welds	257
7.6. Design examples	257

viii

### Chapter 8

#### **THE COMPUTER PROGRAM “ELEFIR-EN”** **267**

---

8.1. General	267
8.2. Brief description of the program	268
8.2.1. Available thermal calculations	268
8.2.2. Available mechanical calculations	273
8.3. Default constants used in the program	278
8.4. Design example	279

---

**Chapter 9****CASE STUDY 293**

9.1. Description of the case study	293
9.2. Fire resistance under standard fire	294
9.2.1. Thermal calculations	294
9.2.2. Structural calculation	295
9.2.2.1. <i>Loading</i>	295
9.2.2.2. <i>Fire resistance by the simple calculation model</i>	300
9.2.2.3. <i>Fire resistance by the general calculation model</i>	302
9.3. Fire resistance under natural fire	304
9.3.1. Temperature development in the compartment	304

**REFERENCES 311****Annex A****THERMAL DATA FOR CARBON STEEL AND STAINLESS  
STEEL SECTIONS 319**

A.1. Thermal properties of carbon steel	319
A.1.1. Specific heat	319
A.1.2. Thermal conductivity	320
A.1.3. Thermal elongation	321
A.2. Section factor $A_m/V$ [m <sup>-1</sup> ] for unprotected steel members	322
A.3. Section factor $A_p/V$ [m <sup>-1</sup> ] for protected steel members	324
A.4. Tables and nomograms for evaluating the temperature in unprotected steel members subjected to the standard fire curve ISO 834	325
A.5. Tables and nomograms for evaluating the temperature in protected steel members subjected to the standard fire curve ISO 834	331
A.6. Thermal properties of some fire protection materials	335
A.7. Thermal properties of stainless steel	336
A.7.1. Specific heat	336
A.7.2. Thermal conductivity	336
A.7.3. Thermal elongation	337

## TABLE OF CONTENTS

---

A.8. Tables and nomograms for evaluating the temperature in unprotected stainless steel members subjected to the standard fire curve ISO 834	339
A.9. Thermal properties of some fire compartment lining materials	345

### Annex B

#### **INPUT DATA FOR NATURAL FIRE MODELS** **347**

---

B.1. Introduction	347
B.2. Fire load density	347
B.3. Rate of heat release density	350
B.4. Ventilation control	354
B.5. Flash-over	358

### Annex C

#### **MECHANICAL PROPERTIES OF CARBON STEEL AND STAINLESS STEEL** **359**

---

C.1 Mechanical properties of carbon steel	359
C.1.1. Mechanical properties of carbon steel at room temperature (20°C)	359
C.1.2. Stress-strain relationship for carbon steel at elevated temperatures (without strain-hardening)	361
C.1.3. Stress-strain relationship for carbon steel at elevated temperatures (with strain-hardening)	370
C.1.4. Mechanical properties to be used with Class 4 cross-section and simple calculation models	372
C.2. Mechanical properties of stainless steel	374

### Annex D

#### **TABLES FOR SECTION CLASSIFICATION AND EFFECTIVE WIDTH EVALUATION** **383**

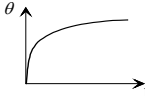

---

<b>Annex E</b>	
<b><u>SECTION FACTORS OF EUROPEAN HOT ROLLED</u></b>	
<b><u>IPE AND HE PROFILES</u></b>	<b><u>389</u></b>
 <b>Annex F</b>	
<b><u>CROSS-SECTIONAL CLASSIFICATION OF EUROPEAN</u></b>	
<b><u>HOT ROLLED IPE AND HE PROFILES</u></b>	<b><u>397</u></b>
F.1. Cross-sectional classification for pure compression and pure bending	398
F.2. Cross-sectional classification for combined compression and bending moment	404

## 5. MECHANICAL ANALYSIS

members should be considered to act together so that the interaction effect between them is directly taken into account (load redistribution from weak heated parts to cold parts outside the fire compartment). Advanced calculation methods, normally based on the Finite Element Method together with a global analysis provide more realistic models of mechanical response of structures in fire than tabulated data or simple models. More information about advanced calculation models is presented in Chapter 6.

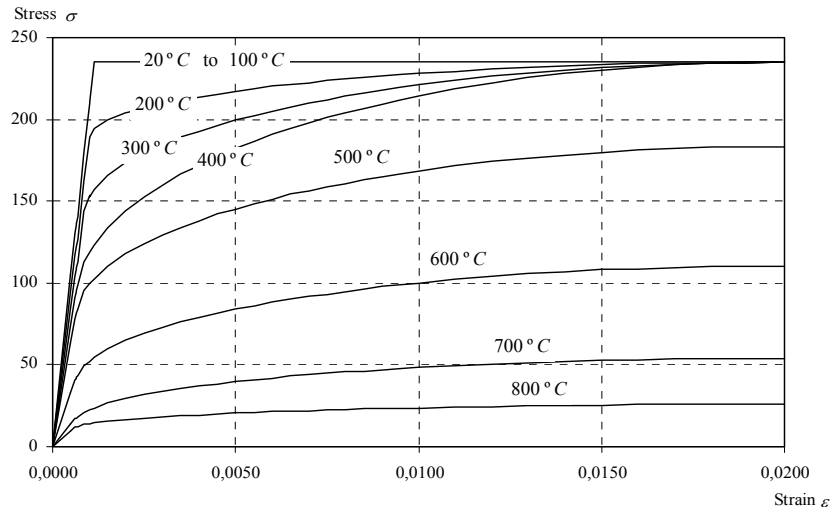
**Table 5.1:** Relation between calculation models, structural schematization and fire model

	Nominal Fires			Natural Fires		
						
Type of Analysis	Tabulated data	Simple Calculation Models	Advanced Calculation Models	Tabulated data	Simple Calculation Models	Advanced Calculation Models
Member analysis	Not available in EC3-1-2	Yes	Yes	No	Yes (if available)	Yes
Analysis of parts of the structure	No	Yes (if available)	Yes	No	No	Yes
Global structural analysis	No	No	Yes	No	No	Yes

### 5.2. MECHANICAL PROPERTIES OF CARBON STEEL

The strength of steel decreases as the temperature increases beyond 400°C. For S235 structural steel, Fig. 5.4 shows the strength as a function of temperature as well as the stress-strain relationships at elevated temperature. This figure also shows that the stiffness of steel also decreases with increasing temperature. At elevated temperature, the shape of the stress-

strain diagram is modified compared to the shape at room temperature. Instead of a linear-perfectly plastic behaviour as for normal temperature, the model recommended by EN 1993-1-2 at elevated temperature is an elastic-elliptic-perfectly plastic model, followed by a linear descending branch introduced at large strains when the steel is used as material in advanced calculation models. Detailed aspects from this behaviour can be seen in Fig. 5.5. More details on the stress-strain relationship for steel grades S235, S275, S355 and S460 are given in Annex C.



**Fig. 5.4:** Stress-strain relationship for carbon steel S235 at elevated temperatures

In an accidental limit state such as fire, higher strains are acceptable. For this reason Eurocode 3 recommends a yield strength corresponding to 2% total strain rather than the conventional 0.2% plastic strain (see Fig 5.5). However, for members with Class 4 cross sections, Eurocode 3 recommends a design yield strength based on the 0.2% proof strain.

The stress-strain relationship at elevated temperature is also shown in Fig 5.5 and is characterised by the following three parameters:

- The limit of proportionality,  $f_{p,\theta}$  :
- The effective yield strength,  $f_{y,\theta}$
- The Young's modulus,  $E_{a,\theta}$

The design values for the mechanical (strength and deformation) material properties in the fire situation  $X_{d,fi}$  are defined in Eurocode 3, as

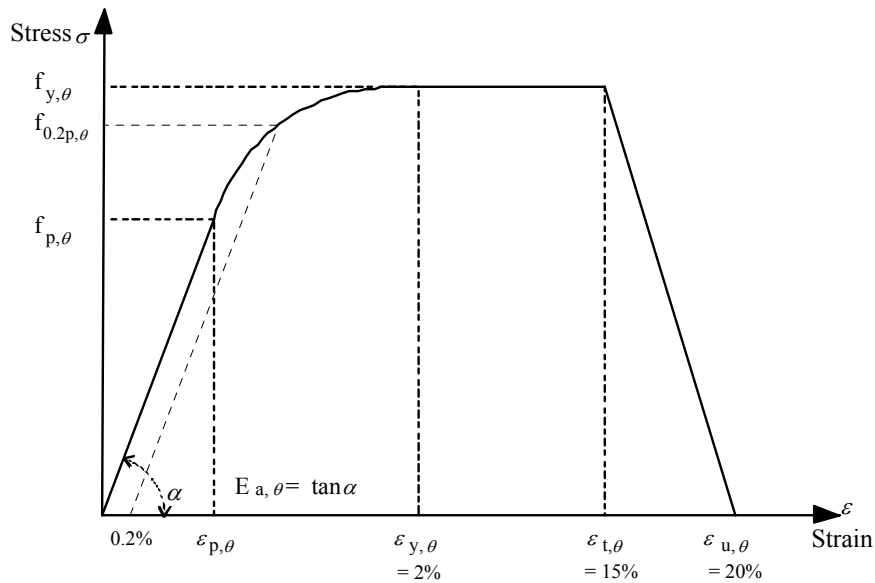
## 5. MECHANICAL ANALYSIS

follows:

$$X_{d,fi} = k_{\theta} X_k / \gamma_{M,fi} \quad (5.2)$$

where:

- $X_k$  is the characteristic value of a strength or deformation property (generally  $f_k$  or  $E_k$ ) for normal temperature design to EN 1993-1-1;
- $k_{\theta}$  is the reduction factor for a strength or deformation property ( $X_{k,\theta}/X_k$ ), dependent on the material temperature;
- $\gamma_{M,fi}$  is the partial safety factor for the relevant material property, for the fire situation, taken as  $\gamma_{M,fi} = 1.0$ , or other value defined in the National Annex.



**Fig. 5.5:** Stress-strain relationship for carbon steel at elevated temperatures

Following Eq. 5.2 the yield strength at temperature  $\theta$ , i.e.,  $f_{y,\theta}$ , is a function of the yield strength,  $f_y$ , at 20 °C, given by:

$$f_{y,\theta} = k_{y,\theta} f_y \quad (5.3)$$

The Young's modulus at temperature  $\theta$ , i.e.,  $E_{y,\theta}$ , is a function of the Young's modulus,  $E_a$ , at 20 °C, given by:

$$E_{a,\theta} = k_{E,\theta} E_a \quad (5.4)$$

In the same way the proportional limit at elevated temperature is given by:

$$f_{p,\theta} = k_{p,\theta} f_y \quad (5.5)$$

According to Annex E of EN 1993-1-2 for members with Class 4 cross section under fire conditions, the design yield strength of steel should be taken as the 0.2% proof strain and thus for this class of cross section the yield strength at temperature  $\theta$ , i.e.,  $f_{y,\theta}$ , is a function of the yield strength,  $f_y$ , at 20 °C given by:

$$f_{y,\theta} = f_{0.2p,\theta} = k_{0.2p,\theta} f_y \quad (5.6)$$

Table 5.2 presents the reduction factors for the stress-strain relationship of carbon steel at elevated temperatures and Fig. 5.6 is a graphical representation of these data. In this table the reduction factor (relative to  $f_y$ ) for the design strength of hot rolled and welded thin-walled sections (Class 4), given in Annex E of EN 1993-1-2, is also presented.

Table 5.2 shows that carbon steel begins to lose strength above 400 °C. For example, at 700 °C it has 23 % of its strength at normal temperature and at 800 °C it retains only 11% of that strength, and its strength reduces to 6% at 900 °C. Concerning the Young's modulus it begins to decrease earlier at 100 °C.

---

107

The reduction of the effective yield strength given by Table 5.2, which was obtained experimentally, can be approximated by the following equation:

$$k_{y,\theta} = \left\{ 0.9674 \left( e^{\frac{\theta_a - 482}{39.19}} + 1 \right) \right\}^{-1/3.833} \leq 1 \quad (5.7)$$


---

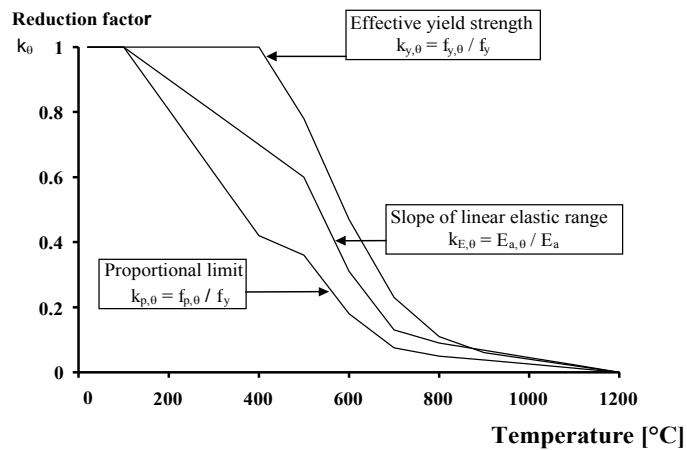
## 5. MECHANICAL ANALYSIS

**Table 5.2:** Reduction factors for carbon steel for the design at elevated temperatures

Steel Temperature $\theta_a$	Reduction factors at temperature $\theta_a$ relative to the value of $f_y$ or $E_a$ at 20°C			
	Reduction factor (relative to $f_y$ ) for effective yield strength $k_{y,\theta} = f_{y,\theta} / f_y$	Reduction factor (relative to $f_y$ ) for proportional limit $k_{p,\theta} = f_{p,\theta} / f_y$	Reduction factor (relative to $E_a$ ) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta} / E_a$	Reduction factor (relative to $f_y$ ) for the design strength of hot rolled and welded thin walled sections (Class 4) $k_{0.2p,\theta} = f_{0.2p,\theta} / f_y$
20 °C	1.000	1.000	1.000	1.000
100 °C	1.000	1.000	1.000	1.000
200 °C	1.000	0.807	0.900	0.890
300 °C	1.000	0.613	0.800	0.780
400 °C	1.000	0.420	0.700	0.650
500 °C	0.780	0.360	0.600	0.530
600 °C	0.470	0.180	0.310	0.300
700 °C	0.230	0.075	0.130	0.130
800 °C	0.110	0.050	0.090	0.070
900 °C	0.060	0.0375	0.0675	0.050
1000 °C	0.040	0.0250	0.0450	0.030
1100 °C	0.020	0.0125	0.0225	0.020
1200 °C	0.000	0.0000	0.0000	0.000

NOTE: For intermediate values of the steel temperature, linear interpolation may be used.

108



**Fig. 5.6:** Reduction factors for the stress-strain relationship of carbon steel at elevated temperatures (see Fig. 3.2 from EN 1993-1-2)

Fig. 5.7 shows the comparison between the values of the reduction of the effective yield strength,  $k_{y,\theta}$ , given by Table 5.2 and the ones obtained using Eq. 5.7. The two curves are very close.

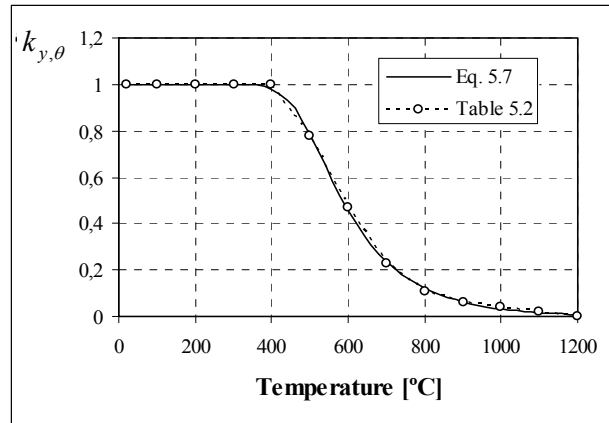


Fig. 5.7: Reduction factors for the yield strength,  $k_{y,\theta}$ , at elevated temperatures

### 5.3. CLASSIFICATION OF CROSS SECTIONS

Rolled or welded structural sections may be considered as an assembly of individual plate elements, some of which are internal elements like the webs of open sections or the flanges of hollow sections, and others are outstand elements like the flanges of open sections. Examples of internal and outstand elements are shown in Fig. 5.8. As the plate elements in structural sections are relatively thin compared with their width, when loaded in compression (as a result of axial loads applied to the whole section and/or from bending) they may buckle locally (see Fig 5.9).

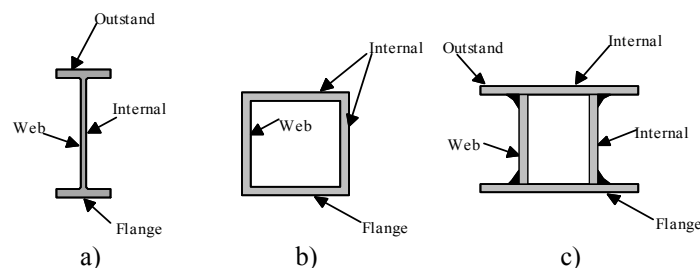
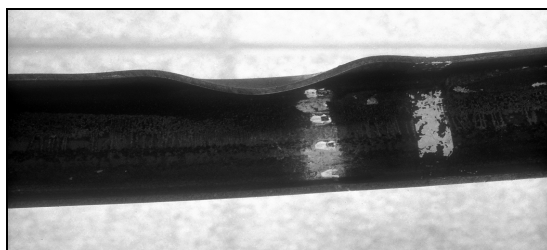


Fig. 5.8: Internal and outstand elements.  
a) Rolled section; b) Hollow section; c) Welded section

The tendency of a plate element within the cross section to buckle may limit the axial load-carrying capacity, or the bending resistance of the section, because collapse can occur before the section reaches its yield strength. Premature failure as a result of local buckling can be avoided by limiting the width-to-thickness ratio of the individual elements within the cross section. An approach which classifies sections according to their ability to resist local buckling is introduced in Eurocode 3 and this approach is described below.



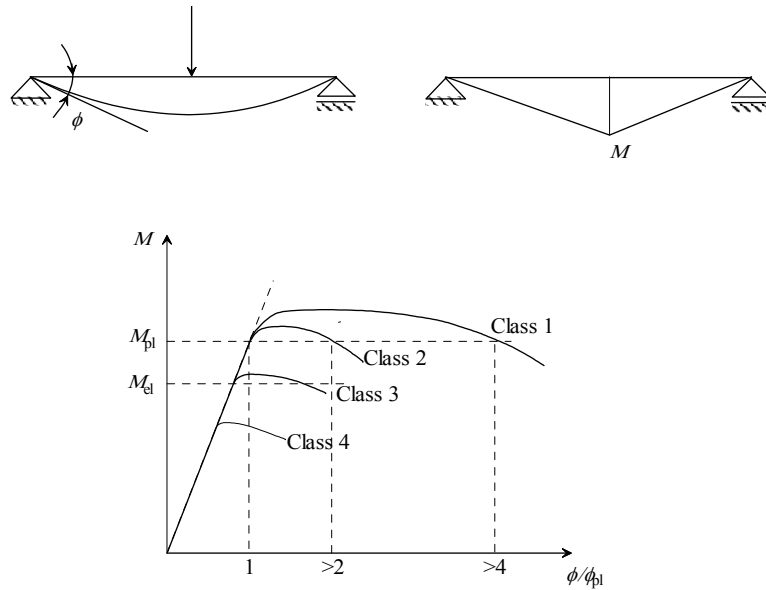
**Fig. 5.9:** Local buckling of the upper flange of a beam subject to bending (ESDEP, 1995)

Eurocode 3 defines four cross section classes depending on the slenderness of each constitutive plate (defined by a width-to-thickness ratio) and on the compressive stress distribution, i.e., uniform or linear:

- **Class 1** cross sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
- **Class 2** cross sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
- **Class 3** cross sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- **Class 4** cross sections are those in which local buckling will occur before reaching the yield strength in one or more parts of the cross section.

Fig. 5.10 shows the moment-rotation curves for each of the four classes, highlighting the strength and the rotation capacity that can be

reached before local buckling occurs. In this figure,  $\phi_{pl}$  is the rotation needed to form a full plastic stress distribution in the most loaded section of the beam, i.e., the rotation needed to form a plastic hinge in that section,  $M_{pl}$  is the plastic moment and  $M_{el}$  the elastic moment.



**Fig. 5.10:** Moment-rotation curves

A key parameter used when analysing plate buckling for I-sections girders and box girders is the normalised plate slenderness,  $\bar{\lambda}_p$ , given by (EN 1993-1-5):

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} \quad (5.8)$$

where  $\sigma_{cr}$  is the elastic critical buckling stress, which can be found in any textbook for stability analysis or in Annex A of EN 1993-1-5, given by:

$$\sigma_{cr} = \frac{k_\sigma \pi^2 E}{12(1-\nu^2)} \left( \frac{t}{b} \right)^2 \quad (5.9)$$

where

$k_\sigma$  is the plate buckling factor which accounts for edge support conditions and stress distribution;

## 5. MECHANICAL ANALYSIS

$\nu$  is Poisson's coefficient;  
 $E$  is the Young's modulus;  
 $t$  is the plate thickness;  
 $b$  is the width of the plate.

Substituting from Eq. (5.9) into Eq. (5.8) and rearranging gives:

$$\begin{aligned}\bar{\lambda}_p &= \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{f_y}{k_\sigma \frac{\pi^2 E t^2}{12(1-\nu^2)b^2}}} = \frac{b/t}{\sqrt{k_\sigma} \sqrt{\frac{\pi^2}{12(1-\nu^2)}}} \frac{1}{\sqrt{\frac{E}{f_y}}} = \\ &= \frac{b/t}{\sqrt{k_\sigma} \sqrt{\frac{\pi^2}{12(1-\nu^2)}}} \frac{1}{\sqrt{\frac{210000}{235} \frac{E}{f_y}}} = \\ &= \frac{b/t}{28.4\sqrt{k_\sigma}} \frac{1}{\sqrt{\frac{235}{f_y} \frac{E}{210000}}} = \frac{b/t}{28.4\sqrt{k_\sigma}} \frac{1}{\varepsilon} = \frac{b/t}{28.4\varepsilon\sqrt{k_\sigma}}\end{aligned}\quad (5.10)$$

where

$$\varepsilon = \sqrt{\frac{235}{f_y} \frac{E}{210000}} \quad \text{with } f_y \text{ and } E \text{ in MPa} \quad (5.11)$$

112

Introducing the parameter  $\varepsilon$  allows the expression for the normalised slenderness  $\bar{\lambda}_p$  to be defined independent of the steel grade. Eq. (5.11) is used in EN 1993-1-4 for stainless steel, which has several Young's modulus values depending on the steel grade. This is not the case for carbon steel where the Young's modulus can be considered as constant at room temperature,  $E = 210000$  MPa. Eurocode 3 defines the following for carbon steel:

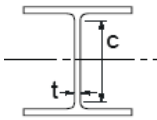
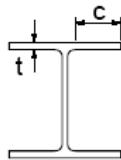
$$\varepsilon = \sqrt{235/f_y} \quad \text{with } f_y \text{ in MPa} \quad (5.12)$$

Eq. (5.11) and Eq. (5.12) are only applicable for carbon steel at room temperature. The benefit of using Eq. (5.11) for carbon steel will appear as soon as high temperatures have to be considered.

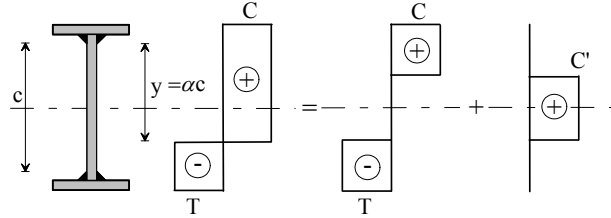
Table 5.3 summarizes the maximum width-to-thickness ratio (slenderness) limits for the constitutive plates of hot rolled profiles in compression or subject to bending about the strong axis, for Class 1, 2 and 3

cross sections. Complete information on hot rolled and welded section classification can be found in the Annex D. For elements with slenderness greater than the Class 3 limits, the cross section should be taken as Class 4. The various compression parts in a cross section (such as a web or flange) can, in general, be of different classes. A cross section is classified according to the highest class of its compression parts.

**Table 5.3:** Maximum slenderness for compression parts of cross section

			
Element	Class 1	Class 2	Class 3
Flange	$c / t = 9 \epsilon$	$c / t = 10 \epsilon$	$c / t = 14 \epsilon$
Web subject to compression	$c / t = 33 \epsilon$	$c / t = 38 \epsilon$	$c / t = 42 \epsilon$
Web subject to bending	$c / t = 72 \epsilon$	$c / t = 83 \epsilon$	$c / t = 124 \epsilon$

The procedure for evaluating the class of a cross section is relatively simple for the case of pure compression and pure bending as shown in Table 5.3. However, when the section is subjected to combined bending and compression ( $M+N$ ) a more laborious procedure is needed. For simplicity, section classification may initially be conducted under the most severe conditions of pure axial compression. If the result is a Class 1 section nothing is to be gained by conducting additional calculations and considering the actual pattern of stresses. However if the result is Class 2, Class 3 or Class 4, then it is normally advisable for economic reasons to repeat the classification calculation more precisely (SCI, 2005), using a parameter  $\alpha$  that defines the compressive part of the web in a I-cross section (see Fig. 5.11 and 5.12 for the case of bending about y-y) as presented in EN 1993-1-1 and reproduced in Table D.1.1. The procedure is illustrated below.



**Fig. 5.11:** Pattern of normal stresses for Class 1 or 2 I-section.  
Positive – Compression (C and C'); Negative – Tension (T)

According to Fig. 5.11, for equilibrium:

$$C = T$$

$$N = C'$$

C and T together resist to bending  $M$

Block C' must be symmetric about the geometrical axis, and therefore:

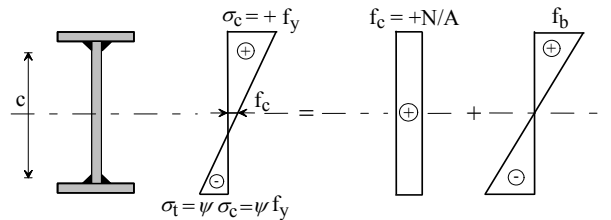
$$(y - \frac{c}{2})2t_w f_y = N \quad (5.13)$$

and the parameter  $\alpha$  is given by

$$\alpha = \frac{y}{c} = \frac{1}{2} + \frac{N}{2ct_w f_y} \quad (5.14)$$

114

If the web is not Class 1 or 2 under combined axial force and bending, the classification of the cross section is made using the ratio,  $\psi = \sigma_t / \sigma_c$ , (which is the ratio of the tensile and compressive stresses at the extreme fibres, as shown in Fig. 5.12). It is assumed that the pattern of normal stresses is the sum of the stresses due to axial force  $N$  and those due to bending, in which the maximum normal stress is equal to the yield stress.



**Fig. 5.12:** Pattern of normal stresses for a Class 3 or 4 I-section.  
Positive – Compression; Negative – Tension

## Fire Design of Steel Structures

EC 1: Actions on structures. Part 1-2: Actions on structures exposed to fire

EC 3: Design of steel structures. Part 1-2: Structural fire design



This book explains and illustrates the rules that are given in the Eurocode for designing steel structures subjected to fire. After the first introductory chapter, Chapter 2 explains how to calculate the mechanical actions (loads) in the fire situation based on the information given in EN 1990 and EN 1991. Chapter 3 presents the models to be used to represent the thermal action created by the fire. Chapter 4 describes the procedures to be used to calculate the temperature of the steelwork from the temperature of the compartment and Chapter 5 shows how the information given in EN 1993-1-2 is used to determine the load bearing capacity of the steel structure. The methods use to evaluate the fire resistance of bolted and welded connections are described in Chapter 7. Chapter 8 describes a computer program called "Elefir-EN" which is based on the simple calculation model given in the Eurocode and allows designers to quickly and accurately calculate the performance of steel components in the fire situation. Chapter 9 looks at the issues that a designer may be faced with when assessing the fire resistance of a complete building. This is done via a case study and addresses most of the concepts presented in the earlier Chapters. The concepts and fire engineering procedures given in the Eurocodes may seem complex those more familiar with the prescriptive approach. This publication sets out the design process in a logical manner giving practical and helpful advice and easy to follow worked examples that will allow designer to exploit the benefits of this new approach to fire design.

(428 pages with 134 figures. Softcover. Date of publication: May 2010)

**Please pass this order form to your local bookseller or**  
**Fax-No. +49 (0)30 47031 240 – Ernst & Sohn Berlin, Germany**

No.	Order-No.	Title	Unit price.*
	978-3-433-02974-9	Fire Design of Steel Structures	€ 70 / \$ 95
	904221	Publishing Index Verlag Ernst & Sohn	for free
	2488	Journal Steel Construction - Design and Research	1 sample copy for free

Delivery- and Invoice address: ☐private ☐business

Company			
Contact person			Telephone
UST-ID Nr./VAT-ID No.			Fax
Street//No.			E-Mail
Country	-	Zip code	Location

X

\_\_\_\_\_  
Date / Signature