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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
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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

<table>
<thead>
<tr>
<th>Type of Analysis</th>
<th>Nominal Fires</th>
<th>Natural Fires</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tabulated data</td>
<td>Simple Calc. Models</td>
<td>Advanced Calc. Models</td>
</tr>
<tr>
<td>Member analysis</td>
<td>Not available in EC3-1-2</td>
<td>Yes</td>
</tr>
<tr>
<td>Analysis of parts of the structure</td>
<td>No (if available)</td>
<td>Yes</td>
</tr>
<tr>
<td>Global structural analysis</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

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.

![Stress-strain relationship for carbon steel S235 at elevated temperatures](image)

**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, \( E_{u,\theta} \).

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

\[ X_{d,fi} = k_\theta X_k / \gamma_{M,fi} \]  

(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 \( (X_k/\theta) \), 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:

\[ f_{y,\theta} = k_{y,\theta} f_y \]  

(5.3)

The Young’s modulus at temperature \( \theta \), i.e., \( E_{y,\theta} \), is a function of the Young’s modulus, \( E_y \), at 20 °C, given by:
5.2. MECHANICAL PROPERTIES OF CARBON STEEL

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

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 \(^\circ\)C given by:

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

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 \(^\circ\)C. For example, at 700 \(^\circ\)C it has 23 % of its strength at normal temperature and at 800 \(^\circ\)C it retains only 11% of that strength, and its strength reduces to 6% at 900 \(^\circ\)C. Concerning the Young’s modulus it begins to decrease earlier at 100 \(^\circ\)C.

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

\[
k_{y,\theta} = \left\{ 0.9674 \left( \frac{\theta - 482}{69.19} \right) + 1 \right\}^{0.833} \leq 1 \quad (5.7)
\]
### Table 5.2: Reduction factors for carbon steel for the design at elevated temperatures

<table>
<thead>
<tr>
<th>Steel Temperature $\theta_c$</th>
<th>Reduction factors at temperature $\theta_c$ relative to the value of $f_y$ or $E_a$ at 20°C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reduction factor (relative to $f_y$) for effective yield strength $k_{y,\theta} = f_{\theta,\theta} / f_y$</td>
</tr>
<tr>
<td>20 ºC</td>
<td>1.000</td>
</tr>
<tr>
<td>100 ºC</td>
<td>1.000</td>
</tr>
<tr>
<td>200 ºC</td>
<td>1.000</td>
</tr>
<tr>
<td>300 ºC</td>
<td>1.000</td>
</tr>
<tr>
<td>400 ºC</td>
<td>1.000</td>
</tr>
<tr>
<td>500 ºC</td>
<td>0.780</td>
</tr>
<tr>
<td>600 ºC</td>
<td>0.470</td>
</tr>
<tr>
<td>700 ºC</td>
<td>0.230</td>
</tr>
<tr>
<td>800 ºC</td>
<td>0.110</td>
</tr>
<tr>
<td>900 ºC</td>
<td>0.060</td>
</tr>
<tr>
<td>1000 ºC</td>
<td>0.040</td>
</tr>
<tr>
<td>1100 ºC</td>
<td>0.020</td>
</tr>
<tr>
<td>1200 ºC</td>
<td>0.000</td>
</tr>
</tbody>
</table>

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

**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)
5.2. MECHANICAL PROPERTIES OF CARBON STEEL

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.

![Graph showing reduction factors for the yield strength, $k_{y,\theta}$, at elevated temperatures.](image)

**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).

![Diagram of internal and outstand elements.](image)

**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.

![Local buckling of the upper flange of a beam subject to bending](ESDEP, 1995)

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.

\[ \phi_{pl} \]

\[ M_{pl} \]

\[ M_{el} \]

**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):

\[
\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} 
\]

(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:

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

(5.9)

where $k_\sigma$ is the plate buckling factor which accounts for edge support conditions and stress distribution;
5. MECHANICAL ANALYSIS

\[ \nu \text{ is Poisson’s coefficient; } \]
\[ E \text{ is the Young’s modulus; } \]
\[ t \text{ is the plate thickness; } \]
\[ b \text{ is the width of the plate.} \]

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

\[ \tilde{X}_p = \frac{f_y}{\sigma_{cr}} = \sqrt{\frac{f_y}{k_{\sigma} \frac{\pi^2 E t^2}{12(1-\nu^2)b^2}}} = \frac{b/t}{\sqrt{\frac{k_{\sigma} \frac{\pi^2 E t^2}{12(1-\nu^2)b^2}}} \frac{\pi^2 E t^2}{12(1-\nu^2)b^2}} \frac{1}{\sqrt{12(1-\nu^2)b^2}} = \frac{b/t}{\sqrt{k_{\sigma} \frac{\pi^2 E t^2}{12(1-\nu^2)b^2}}} \frac{1}{\sqrt{12(1-\nu^2)b^2}} \]

\[ = \frac{b/t}{\sqrt{210000 \frac{f_y}{235}}} \frac{235}{210000} \frac{E}{f_y} = \frac{b/t}{28.4\sqrt{k_{\sigma}}} \frac{1}{\sqrt{210000 \frac{f_y}{235}}} \frac{235}{210000} \frac{E}{f_y} = \frac{b/t}{28.4\sqrt{k_{\sigma}}} \frac{1}{\sqrt{210000 \frac{f_y}{235}}} \frac{235}{210000} \frac{E}{f_y} \]

\[ = \frac{b/t}{28.4\sqrt{k_{\sigma}}} \frac{1}{\sqrt{210000 \frac{f_y}{235}}} \frac{235}{210000} \frac{E}{f_y} \]

where

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

Introducing the parameter \( \varepsilon \) allows the expression for the normalised slenderness \( \tilde{X}_p \) 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:

\[ \varepsilon = \sqrt{\frac{235}{f_y}} \text{ with } f_y \text{ in MPa} \quad (5.12) \]

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
5.3. CLASSIFICATION OF CROSS SECTIONS

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

<table>
<thead>
<tr>
<th>Element</th>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange</td>
<td>(c/t = 9\varepsilon)</td>
<td>(c/t = 10\varepsilon)</td>
<td>(c/t = 14\varepsilon)</td>
</tr>
<tr>
<td>Web subject to compression</td>
<td>(c/t = 33\varepsilon)</td>
<td>(c/t = 38\varepsilon)</td>
<td>(c/t = 42\varepsilon)</td>
</tr>
<tr>
<td>Web subject to bending</td>
<td>(c/t = 72\varepsilon)</td>
<td>(c/t = 83\varepsilon)</td>
<td>(c/t = 124\varepsilon)</td>
</tr>
</tbody>
</table>

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.
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:
\[ (y - \frac{e}{2})2t_wf_y = N \quad (5.13) \]
and the parameter \( \alpha \) is given by
\[ \alpha = \frac{y}{c} = 1 + \frac{N}{2ct_wf_y} \quad (5.14) \]

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

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

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 see 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)

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