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Critical Temperatures of Steel Columns Exposed to Fire

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SUMMARY

This contribution deals with the interdependence between material properties of structural steel at elevated temperatures and critical temperatures of steel columns. A simple design method for estimating lower and upper bound values of critical temperatures is presented. Critical temperatures of steel columns can be determined by means of the related slenderness and the utilization factor. Both parameters can be calculated on the basis of design methods at normal temperatures.

INTRODUCTION

The analysis of the behaviour of steel columns exposed to fire can be separated into two steps solving two different subjects:

- the determination of the rise of steel temperature as a function of time;
- the load-bearing characteristics at elevated temperatures.

The first subject covers the relation between the fire exposure and the steel temperatures as a function of time. For structural fire design, the load-bearing characteristics of steel columns at elevated temperatures can be dealt with independently of the fire duration by considering the steel temperatures only.

Significant for the load-bearing behaviour of steel columns is the so-called critical temperature, crit T, which denotes the collapse temperature at elevated temperatures.

In recent years a considerable amount of research has been carried out studying the parameters affecting the critical temperatures of steel columns. Two parameters are of particular importance:

- the load level;
- the slenderness.

This contribution deals with the interdependence between material properties of structural steel at elevated temperatures and the critical temperatures of steel columns. A design method for the determination of the critical temperature is proposed in the following text.

BASIS OF THE EXACT METHOD OF ANALYSIS

For the determination of critical temperatures of steel columns, a numerical method has been developed in ref. 1. It is an incrementally formulated finite element method for structures based on the displacement method. Geometric non-linearities are taken into account using the 2nd-order theory, whereas material-induced and temperature-dependent non-linearities are taken into account by means of a cross-sectional analysis in the element nodes in the form of a fibre model.

In terms of stress and thermal history, the numerical procedures simulate reality. This means that after the external loads are applied at normal temperatures, the steel temperatures of the columns are gradually increased until failure occurs through loss of stability or reaching the plastic load-bearing capacity.

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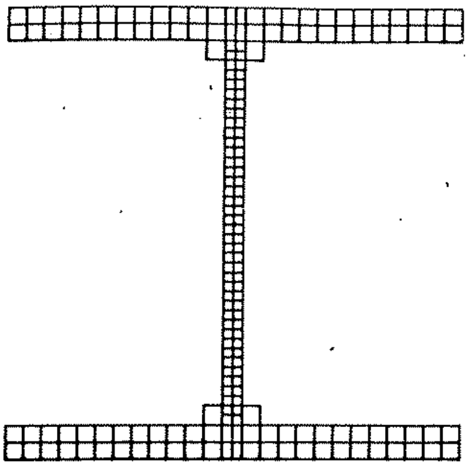

Fig. 1. Fibre model of an I-section.

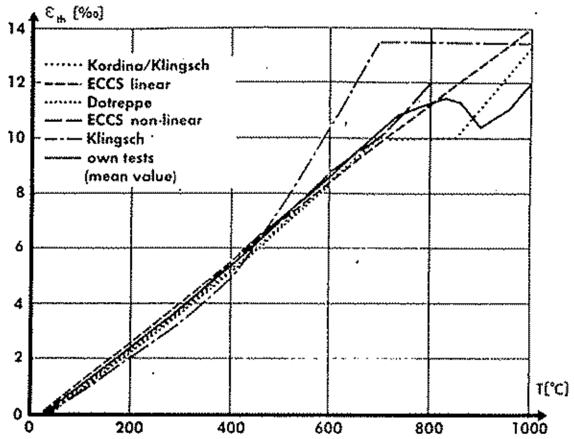

Fig. 2. The temperature-dependent thermal strains of structural steel (different authors).

This procedure corresponds to the standard fire test conditions (Iso-fire).

For numerical computations, the stress–strain relationship of structural steel is required in mathematical formulation. In general, the total strain ϵ_{total} at elevated temperatures can be described by the following expression:

$$\epsilon_{\text{total}} = \underbrace{\epsilon_{\text{th}}(T)}_{\text{thermal strain}} + \underbrace{\underbrace{\epsilon_{\sigma}(T, \sigma)}_{\text{stress-dependent strain}} + \underbrace{\epsilon_{\text{cr}}(t, T, \sigma)}_{\text{warm-creep term}}}_{\text{stress–strain relations}} \qquad (1)$$

The first term takes into account the pure thermal expansion of steel due to the elevated steel temperature. Figure 2 shows several relationships for the thermal strain ϵ_{th} as a function of the steel temperature. In the computations described in this paper, the latest theoretical proposal from Kordina and Klingsch [2] (dotted line) was adopted, which shows the best correlation with the authors' test results (continuous line) [3].

The third term in eqn. (1) takes into account the influence of warm-creep effects. It has been proved by the tests carried out by the authors [4, 5] that; in effect, the influence of the heating rate can be neglected. Hence, the second and the third terms can be combined. This combined term represents the temperature-dependent stress–strain relations of structural steel.

THE TEST PROCEDURE

Because of the particular importance of the stress–strain relationship with regard to the analytical results, the difference between the method used here and traditional test methods shall be pointed out. Up to now, the temperature-dependent behaviour of structural steel under simultaneous external and thermal loading has been studied mainly on the basis of uniaxial tensile tests using small-scale specimens. In contrast to this, transient-state tests on simply supported beams (IPE 80 and IPE 120 sections) with a single load being applied in the midspan were carried out by the authors. The beams, under constant load, were heated up very uniformly. The results of the 17 tests, as well as the general approach of the new testing method and the derivation of the stress–strain relationship, are published in detail in refs. 4 and 5.

Figure 3 shows the test procedures of uniaxial tensile tests and beam tests. In both cases, a constant load is applied to the specimen and then the temperature increased as a function of time. The difference between the test procedures becomes evident from the examination of the deformed specimens.

The elongation Δl measured in the tensile tests includes both the thermal strain and the strain due to the stress–strain relationship. The vertical deflections measured in the beam tests contain only that part of the strains required to derive stress–strain dependencies. Inaccuracies due to the combination of both

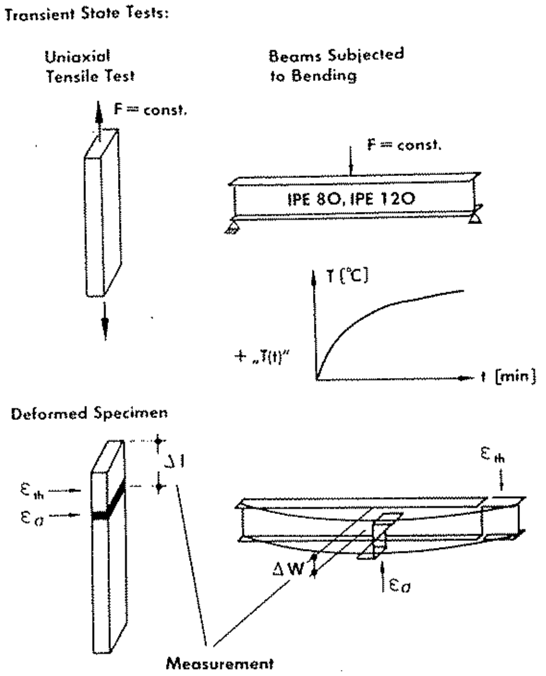

Fig. 3. Different test procedures of transient-state tests.

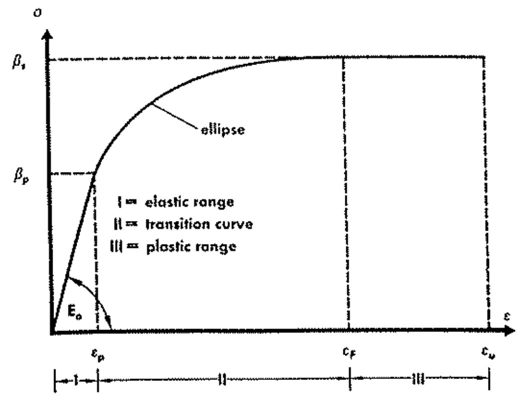

Fig. 4. The basic formulation of the stress–strain relationship of structural steel at elevated temperatures.

strains measured as total deformation is avoided using the transient-state beam tests.

The basic formulation of the stress–strain relationship of structural steel at elevated temperatures is given in Fig. 4. The analytical formulation of the stress–strain relations as well as the temperature-dependence of the three parameters $\bar{E}_0$, $\bar{\beta}_p$ and $\bar{\beta}_s$ are given in ref. 4.

It should be emphasized here that, for the first time, the so-called elastic limit, β_p, is derived and presented as a function of steel temperatures in ref. 4. It is an important parameter to simulate numerically the load–temperature deflection behaviour of beams and columns and the stability behaviour of columns and frames.

RESULTS OF THE PRECISE CALCULATION METHOD

In all cases, the computations take into account geometric eccentricities and residual stresses according to EUROCODE 3 [6], when determining the load-bearing capacity F_u at normal temperatures. This value is used for the determination of the load utilization factor, F/F_u, and when calculating the critical temperature.

As a typical example, Fig. 5 illustrates the interdependence between the slenderness and the critical temperatures of steel columns — buckling about the strong axis — at two different load levels. The curves are calculated for two hot-rolled I-sections according to the European buckling curves (a) and (b). The form of the curves is representative for all kinds of steel sections.

In particular, it can be seen that the critical temperatures of steel columns have a minimum in the middle range of slenderness at about $\bar{\lambda} = 1.0$. Crit T reaches its maximum for those steel columns the failure of which depends upon the ultimate load-bearing capacity of the cross-section $\bar{\lambda} = \,>\Rightarrow 0.0$

From this result, upper and lower limits can be presented for the critical temperatures of steel columns as a function of the load level. The relation between the material properties as limiting values on one hand and

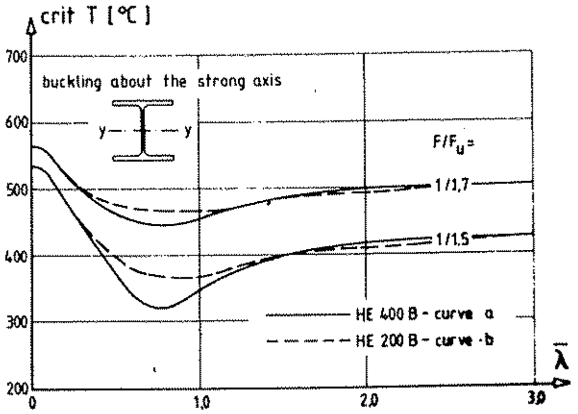

Fig. 5. Influence of the slenderness on the critical temperature of steel columns.

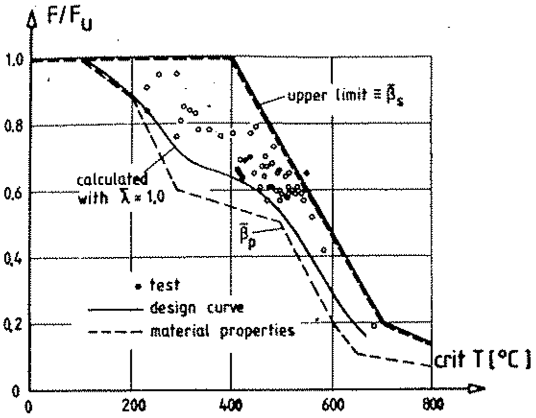

Fig. 6. Temperature-dependency of the collapse load of steel columns at elevated temperatures: comparison between German transient-state column tests and design curves.

the critical temperatures on the other hand now becomes obvious (see Fig. 6).

The upper limit, of course, is identical with the temperature-dependent yield point β_s of structural steel. A precise lower limit has to be calculated by using the numerical FEM-analysis. Here, the computation of the lower limit is based on an I-section, HE 200 B, buckling about the strong axis. Obviously, this lower limit is substantially affected by the temperature-dependent elastic limit β_p. Avoiding any temperature-dependent analysis, the use of the curve of β_p alone gives a safe approach determining the critical temperatures as a lower bound.

Figure 6 illustrates also the range between the upper and lower critical temperature limits for steel columns. For loads at the serviceability level below the maximum design load F (≈60% of the collapse load at normal temperature F_u), the difference is at maximum about 100 °C. The temperature-dependence in this range $0.2 \leqslant F/F_u \leqslant 0.6$ is almost linear.

THE PROPOSED DESIGN METHOD IN COMPARISON TO COLUMN TEST RESULTS

To compare test results in a very general way, the following generalized concept for steel columns is used:

(a) the geometrical and material properties of a steel column are summarized by use of the relative slenderness ratio

$$\bar{\lambda} = \left(\frac{F_{\rm pl}}{F_{\rm ki}}\right)^{1/2}$$

and denotes the square root of the plastic capacity of the cross-section $F_{\rm pl}$ versus the elastic buckling (Euler-) load $F_{\rm ki}$ at normal temperature (Fig. 7).

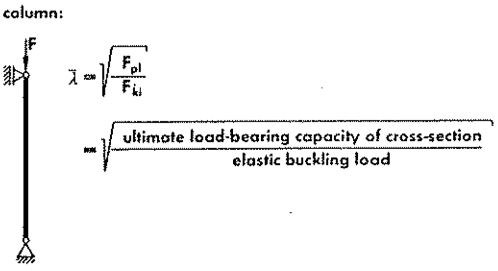

Fig. 7. The characteristic parameters of the proposed design method.

(b) the external load level is defined by the utilization factor

$$1/\nu_u = F/F_u$$

where F_u is the ultimate load-bearing capacity at normal temperature taking into account stability, and F is the actual load in the column. In the case of axially loaded columns, the load-bearing capacity F_u can be determined by using the European buckling curves as given in the EUROCODE 3. In Germany, the safety factor used for the determination of the design load at serviceability of steel columns is $\nu_u = 1.7$.

(c) given uniform or nearly uniform heating of the column, the critical temperature is a function of the slenderness ratio and the utilization factor:

$$\text{crit } T = \text{f}(1/\nu_u, \bar{\lambda})$$

Figure 6 shows the results of more than 50 German full-scale column fire tests with varying load utilization factors, slenderness ratios, load eccentricities, bending axes and section types in comparison with the upper and the lower limits. Only those tests are used where the real (measured) yield stresses β_s at normal temperatures are recorded. In the whole temperature range from about 200 °C up to 700 °C good correlations could be achieved between tests and the numerical simulation of individual tests [1]. All test results lie between both design curves. This

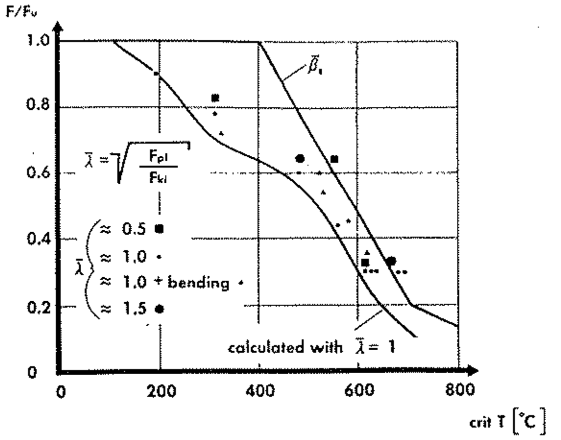

Fig. 8. Comparison between French transient-state column tests and design curves.

result does confirm in hindsight the material properties explained before.

Figure 8 shows the results of French column tests [8]. The utilization factors were calculated with the measured yield stresses. The slenderness parameters of the tests could be separated into special ranges. The tests with $\bar{\lambda}$ of about 1.0 show the lowest critical temperatures and are in good agreement with the calculated lower limit. It should be mentioned that the influence of the bending moments is enclosed in the utilization factor F/F_u.

The available Belgian transient-state tests [9] on axially loaded steel columns show the same good correlations (Fig. 9). Especially the test results on the basis of the actual yield stresses are in excellent agreement with the hypothesis of upper and lower limits. In general, the actual yield stress of hot-rolled steel members is higher than the nominal value. Therefore, the results of the columns with the slenderness ratio $\bar{\lambda} = 0.27$, which are based on nominal yield stress, are partly ranging above the theoretical limit.

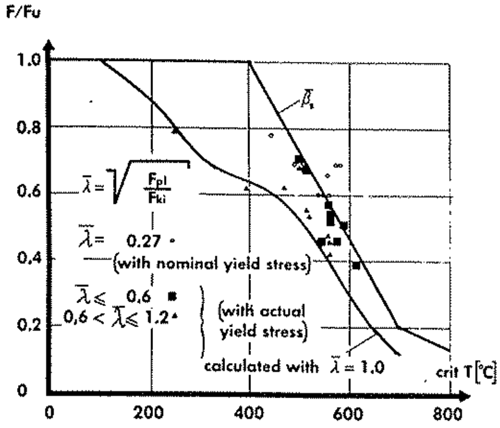

Fig. 9. Comparison between Belgian transient-state column tests and design curves.

Due to the special Danish test conditions [10], the experimental results differ from the calculation results for two important reasons (Fig. 10). The tests were carried out under steady-state conditions, which means that the temperatures in the columns were increased to a certain level and the temperatures then were held constant, while the axial load was increased until the failure occurred. The actual yield stresses of the test specimen were not recorded. So the ultimate load-bearing capacity F_u cannot be calculated exactly. As a consequence, the real load utilization factor F/F_u is incorrect.

It can be shown by theoretical investigations and it is proved by the Danish tests and demonstrated in Fig. 10 that, with an increasing load utilization factor F/F_u (or with decreasing critical temperatures) in connection with an incorrect value of $\bar{\lambda}$, the theoretical prediction of critical temperatures becomes more inaccurate. Only when the actual yield stress of the specimen is well known and the test was carried out under transient-state conditions, do the theoretical and experimental results show good correla-

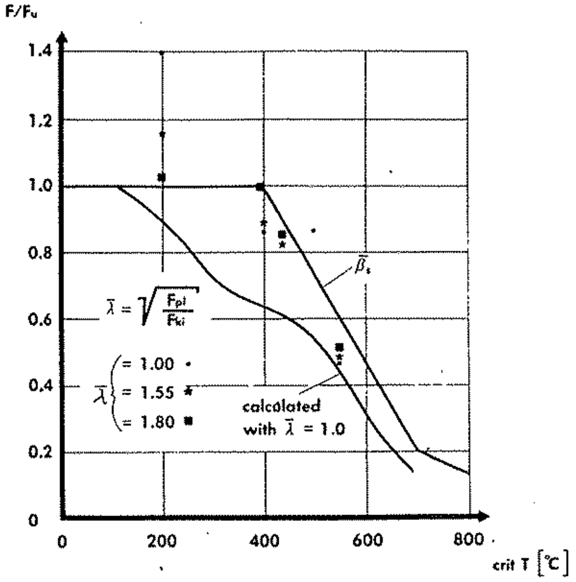

Fig. 10. Comparison between Danish steady-state column tests with nominal yield stress and design curves.

tion. On the other hand — also in this case — the calculated lower limit remains valid.

CONCLUSIONS AND PRACTICAL SIGNIFICANCE

Considering the results given here, a design method for the critical temperature of steel columns is presented by using the slenderness ratio and the utilization factor only. These two parameters can be determined on the basis of design methods at normal temperatures.

The critical temperatures of steel columns at serviceability level range between about 450 °C and 550 °C, depending on their related slenderness. There is a minimum of critical temperatures at values around $\bar{\lambda} \approx 1.0$. For related slenderness ratios between $\bar{\lambda} = 0$ and $\bar{\lambda} = 1.0$, the critical temperature can be obtained by linear interpolation between the upper and the lower limit.

A very direct approach can be made for estimating the critical temperature: per one percent of decreasing load level the critical temperature of the column rises about 3.5 °C. That means, for example, if the critical temperature of a steel column under design load is 500 °C, the critical temperature under half design load is about 650 °C. This approach is valid in the range of utilization factors between 0.2 to 0.6.

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LIST OF SYMBOLS

T	steel temperature (°C)
crit T	critical temperature (°C)
F	actual load (kN)
F_u	ultimate load-bearing capacity (kN)
F_{pl}	ultimate load-bearing capacity of cross-section (kN)
F_{ki}	elastic buckling load (kN)
A	cross-section (cm^2)
$E_0, \bar{E}_0$	initial Young's modulus (kN/cm^2) and specified initial Young's modulus (/)
σ	stress (kN/cm^2)
ϵ	strain (/)
ϵ_σ	stress-induced strain (/)
ϵ_{th}	thermal strain (/)
ϵ_u	ultimate strain (/)
$\beta_p, \bar{\beta}_p$	elastic limit stress (kN/cm^2) and specified elastic limit stress (/)
$\beta_s, \bar{\beta}_s$	yield stress (kN/cm^2) and specified yield stress (/)
$\bar{\lambda}$	related column slenderness ratio (/)
$1/\nu_u$	load utilization factor (/)

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