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Autor: Vandamme, M. / Janss, J.

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Buckling of Axially Loaded Steel Columns in Fire Conditions

Flambement de colonnes métalliques chargées axialement, sous l'effet d'incendies

Knicken von axial belasteten Stahlstützen unter Feuereinwirkung

M. VANDAMME
Scientific Cooperant
University of Ghent
Ghent, Belgium

J. JANSS
Chief Engineer
C.R.I.F.
Liège, Belgium

SUMMARY
A simple method for the design of axially loaded steel columns in fire conditions is presented. The method adopts the European buckling curves for the design of axially loaded bare metal sections as the basic design curves for steel columns in fire conditions.
The method has been compared with experimental results made recently in Belgium and in Denmark and the agreement is shown to be excellent and on the safe side.

RÉSUMÉ
Une méthode analytique simple est proposée pour le calcul des colonnes métalliques chargées axialement et soumises à des températures d'incendie. Dans cette méthode, les courbes européennes de flambement sont adoptées comme courbes de base pour le calcul des colonnes métalliques dans des conditions d'incendie.
La méthode a été comparée avec des résultats d'essai au feu récemment effectués en Belgique et au Danemark. Les charges expérimentales de flambement à haute température concordent très bien avec les valeurs calculées tout en restant du côté de la sécurité.

ZUSAMMENFASSUNG
Die Autoren beschreiben eine analytische Methode für die Berechnung von axial belasteten Stahlstützen unter Feuereinwirkung.
Bei dieser Methode werden die europäischen Knickspannungskurven als Basiskurven für die Berechnung von Stahlstützen unter Feuereinwirkung verwendet. Die Methode wurde mit kürzlich in Belgien und Dänemark durchgeführten Versuchsergebnissen verglichen. Die experimentell ermittelten Knicklasten stimmen sehr gut mit den berechneten Werten überein, welche auf der sicheren Seite bleiben.

1. INTRODUCTION

Up to now there have been few experimental investigations dealing with the stability of steel columns in fire conditions. This is mainly due first to the fact that few laboraties in Europe are equipped with testing apparatus allowing full scale tests; and second to the fact that the main effort has been put on the evaluation of the "fire resistance" of protected columns. Therefore the various existing results concern this last problem [1] and there is few information useful to the solution of the stability problem itself.

In the last years the structural fire engineering design came into existence, the aim of which is to supplement experimental data with analytical methods by which the structural behaviour of steel elements at elevated temperatures can be determined. [2][3][4]

The aim of this paper is to present a simple design method for axially loaded steel columns in fire conditions and to establish the validity of the approach by comparison with the results of an important test programme. The proposed design method is in accordance with the European Recommandations for Steel Construction at ambient temperature. [5]

2. BUCKLING OF STEEL COLUMN AT AMBIENT TEMPERATURE

Since Euler's historical approach the design bases of individual compression members have varied widely. The classical approach to the stability of axially loaded hinged steel columns was based on the assumption of a perfectly straight member of homogeneous material without any residual stresses, and of a perfectly centered axial load. In fact the idealized column does not exist and therefore high, more or less arbitrarily determined safety factors have been adopted to cover the decrease of resistance due to the imperfections.

More recently, analyses have been developed to calculate the stability limit of columns with known or assumed imperfections of geometry and loading.

In order to have a common approach to the buckling problem, the European Convention for Constructional Steelwork (E.C.C.S.) decided in the 1960's to carry out an extensive research action concerning compression members with all their geometrical and structural imperfections. The experimental program (more than 1.500 tests done in 7 european countries) and the theoretical investigation were based on statistic and probabilistic principles wherever possible and a computer simulation of the buckling tests using a Monte Carlo method supplemented the theoretical program.

The systematic theoretical investigation based on experimental data showed a wide scatter of column strength depending on the type of cross-section and the manufacturing procedures. It justified the selection of several representative column curves to which the strength of the most commonly used structural sections can be related. [5]

On the basis of this important research work, five new european non-dimensional buckling curves were proposed by E.C.C.S. (figure 1) [5] [6]

The following analytical expressions for the 5 non-dimensional buckling curves were adopted [7]

$$\overline{N} = \frac{1 + \alpha(\overline{\lambda} - 0,2) + \overline{\lambda}^2}{2\overline{\lambda}^2} - \frac{1}{2\overline{\lambda}^2}\sqrt{[1 + \alpha(\overline{\lambda} - 0,2) + \overline{\lambda}^2]^2 - 4\overline{\lambda}^2} \qquad (1)$$

with

Curve	a_o	a	b	c	d
α	0,125	0,206	0,339	0,489	0,756

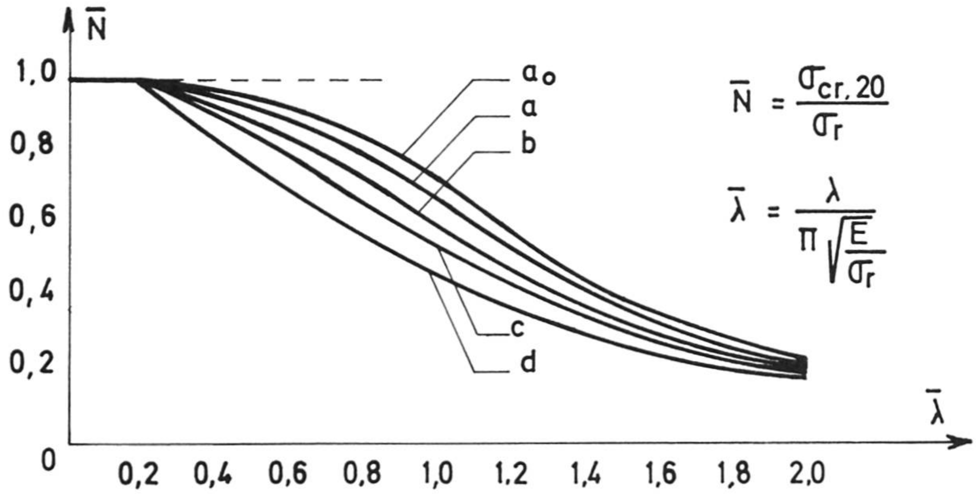

Fig.1. E.C.C.S. non dimensional buckling curves

In several countries, the european buckling curves were introduced in the national codes (for exemple in Belgium : NBN B 51-001 [8]).

3. PROPOSAL FOR A DESIGN METHOD FOR AXIALLY LOADED COLUMNS IN FIRE CONDITIONS

3.1. General

To deal with the behaviour of steel columns in fire conditions it seemed logical to follow the same theoretical procedure as the one developed by E.C.C.S. to establish the buckling curves at ambient temperature. However this proved impossible for several reasons. The simulation programme used in the theoretical investigation of E.C.C.S. [5] [6] requires the evaluation of the geometrical imperfections, the residual stresses, the scatter of the yield-stress within a cross-section, the stress-strain relationship for the steel, etc. All these values are well known for ambient temperature conditions but are presently difficult or impossible to evaluate under fire conditions. For example : how do the residual stresses behave ? Is the stress-strain relationship in compression the same as in traction and what kind of relationship should be choosen among the various solutions presented in the recent literature ?

It appears therefore hopeless, at the present time, to use a simulation programme, performant as it is, where the input data are affected by so much uncertainties.

For all these reasons, the authors switched to a simple analytical method directly connected to the E.C.C.S. buckling curves at ambient temperature. This method is described in chapter 3.2.

At the present time some countries as Sweden, France, The Netherlands, Denmark have also presented some simple methods which are described in references [2] [4] [3] [9].

3.2. Proposal of a simple design method in accordance with the Recommandations OF THE E.C.C.S.

The european non dimensional buckling curves at ambient temperature are represented by the well known RONDAL-MAQUOI [7] equation adopted by the E.C.C.S. (see § 2)

$$\overline{N} = \frac{1 + \alpha(\overline{\lambda} - 0{,}2) + \overline{\lambda}^2}{2\overline{\lambda}^2} - \frac{1}{2\overline{\lambda}^2} \sqrt{[1 + \alpha(\overline{\lambda} - 0{,}2) + \overline{\lambda}^2]^2 - 4\overline{\lambda}^2} \qquad (1)$$

This equation depends on the yield-stress σ_r, the YOUNG's modulus E and the slenderness ratio λ.

Several proposals for the variations of the yield-stress and the YOUNG's modulus with the temperature have been recently presented. [2] [3] [10], and the following relationships proposed by the E.C.C.S. [11] have been adopted :

$$\frac{\sigma_{r,\theta}}{\sigma_r} = 1 + \frac{\theta}{767 \ln\frac{\theta}{1750}} \qquad (0 \leqslant \theta \leqslant 600^\circ C) \qquad (2)$$

$$\frac{\sigma_{r,\theta}}{\sigma_r} = \frac{108(1 - \frac{\theta}{1000})}{\theta - 440} \qquad (600 \leqslant \theta \leqslant 1000^\circ C) \qquad (2bis)$$

$$E_\theta = E\,[-17{,}2.10^{-12}\theta^4 + 11{,}8.10^{-9}\theta^3 - 34{,}5.10^{-7}\theta^2 + 15{,}9.10^{-5}\theta + 1] \qquad (N/mm^2) \qquad (3)$$

In order to transform the general equation of the non dimensional buckling curves at ambient temperature into an equation fitted to temperature θ it seems logical to substitute $\sigma_{r,\theta}$ to σ_r and E_θ to E.

It appears immediately that in this transformation the influence of E_θ on the value of $\overline{N}$ is negligible so that the resulting equation is as follow

$$\overline{N}_\theta = \frac{\sigma_{r,\theta}}{\sigma_r}\left[\frac{1 + \alpha(\overline{\lambda} - 0{,}2) + \overline{\lambda}^2}{2\overline{\lambda}^2} - \frac{1}{2\overline{\lambda}^2}\sqrt{[1 + \alpha(\overline{\lambda} - 0{,}2) + \overline{\lambda}^2] - 4\overline{\lambda}^2}\right] \qquad (4)$$

For safety reasons it has been decided to cover presently all classes of profiles by one single curve corresponding to curve "c". This position could be modified in the future when more experimental results covering a wider range of sections and steel qualities are available.

Figure 2 shows the non dimensional buckling curves corresponding to different temperatures. Equation (4) and the curves of figure 2 have the great advantage to be independant of the steel quality and can thus be used to design columns witch different yield strengths in fire conditions.

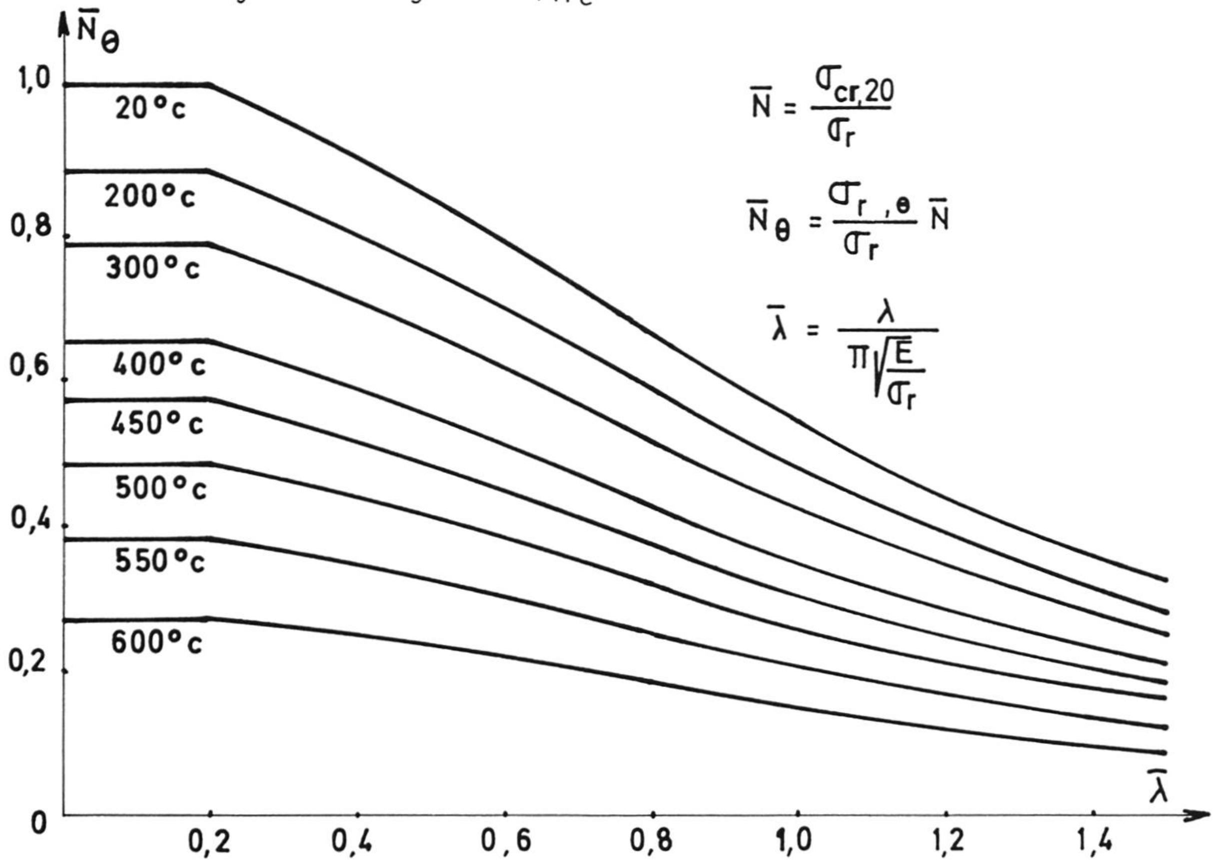

Fig. 2. Non dimensional buckling curves at elevated temperature.

4. EXPERIMENTAL VERIFICATION OF THE PROPOSED DESIGN METHOD

4.1. Test procedure

The buckling tests at elevated temperature were performed using the test equipment of the University of GHENT under leadership of Professor MINNE, director of the Fire Research Station.

In the test equipment and the tests an extreme care has been taken :

1° to achieve and to verify a correct axiality of the load;

2° to ensure high accuracy concerning the load level;

3° to avoid any initial bending moment which may be induced by small inclination of the supporting beams of the frame and the ends of the column itself. For this purpose special end blocks have been developed. These end fixtures provide a perfect rotational restraint at both ends. (figure 3) [12]

In order to verify the points mentionned, load tests at regular intervals at ambient temperature are carried out and the accuracy of the whole loading system is checked by deformation measurements on standard columns as well as some test elements.

Fig. 3 Special end fixture

4.2. Test results

4.2.1. Prelimmary tests

The test programme involved 33 columns (4 tests at ambient temperature and 29 tests in fire conditions) and the choosen parameters were the type of profile and the slenderness ratio λ.

Before testing, the profiles were submitted to the usual measurements in order to check the importance of the geometrical and structural imperfections and it was found that all columns were within the geometrical and structural tolerances adopted by E.C.C.S. [5] [6] for the study of buckling at ambient temperature.

4.2.2. Buckling tests at ambient temperature

In the furnace of the University of GHENT, the columns are placed in a vertical position and clamped in special end fixtures intended to provide a perfect rotational restraint at both ends.

To assess the actual end conditions of the columns - which constitute a very important point for all subsequent calculation and interpretation - four buckling tests have been made at ambient temperature.

Table 1 and 2 give the geometrical and mechanical properties of the columns tested at ambient temperature and the results of the buckling tests.

Table 1 Mechanical and geometrical properties of the columns tested a ambient temperature.

Column id.	Type of profile	Length of the column l en mm	Slenderness ratio λ (*)	Measured yield-stress σ_r N/mm^2	Actual area of the cross section (A_a) mm^2 (**)
0.1	HEB 200	3780	37,28	247,5	7787
0.2	HEB 200	3780	37,28	247,5	7787
0.3	HEB 200	3780	37,28	247,5	7787
0.4	HEB 140	3780	53,49	245,0	4300

* $\lambda = \frac{l}{2i}$ it is assumed that the columns have fixed end conditions

** values obtained by the weight method assuming a specific weight for steel of 7850 kg m^{-3}

Table 2 Results of the bucking tests at ambient temperature.

Column id.	Type of profile	λ	Critical load P_{cr} KN	Critical buckling stress $\sigma_{cr,20} = \frac{P_{cr}}{A_a}$ N/mm^2	$\bar{\lambda} = \pi\sqrt{\frac{E}{\sigma_r}}$	$\bar{N}_{test} = \frac{\sigma_{cr,20}}{\sigma_r}$	$\frac{\bar{N}_{test}}{\bar{N}_{theo}}$
0.1	HEB 200	37,28	1774,2	227,84	0,410	0,92	1,034
0.2	HEB 200	37,28	2236,4	287,20	0,410	1,16	1,303
0.3	HEB 200	37,28	2057,2	264,18	0,410	1,07	1,202
0.4	HEB 140	53,49	934,0	217,21	0,591	0,88	1,116

The average value of the ration $\left(\frac{\bar{N}_{test}}{\bar{N}_{theo}}\right)_{av.}$ between the experimental buckling stress and the theoretical stress obtained from the corresponding characteristic buckling curve of E.C.C.S. [5] for a fixed ends column is 1,16.
As the use of this curve with a confidence level of 97,5 % may seem too optimistic the average value of the ratio $\frac{\bar{N}_{test}}{\bar{N}_{theo}}$ was also computed between the experimental stress and the stress obtained from the corresponding mean curve of E.C.C.S. which has a confidence level of 50 % and the result obtained in this case is 0,995. These results confirm the assumption of fixed ends for the columns tested in the furnace of the University of GHENT.

4.2.3. Buckling tests at elevated temperatures

For fire testing each column is loaded axially and submitted to the thermal exposure according to the ISO 834 standard. The load is applied to the column at ambient temperature and kept constant for the whole duration of the fire test. The longitudinal expansion of the loaded column under fire is free.

a) Failure criterion of axially loaded columns at elevated temperature

Since for loaded columns no failure criterion received international acceptance, a choice had to be made.
Is considered as time of failure, the time at which the thermal elongation is annihilated by the shrinkage of the column. This shrinkage can be due to thermal degradation for concrete or thermal flow in the case of steel columns for example (figure 4).
This failure criterion shows some advantages over other criteria such as a limitation of the speed of deformation :

1° The criterion is easy to apply and to measure. It is not subject to (mis)interpretation.

2° Practice showed that the above defined time precedes very closely the full collapse, i.e. the instant when the loading system can not maintain the full load, because of the speed of deformation.
The later instant however depends on the caracteristics of the loading system.

3° The criterion is totally independent of the length of the column and practically independent of the pump caracteristics of the hydraulic system.

4° The point of zero deformation has some physical meaning in terms of a potential redistribution of the forces in the members of a structure.

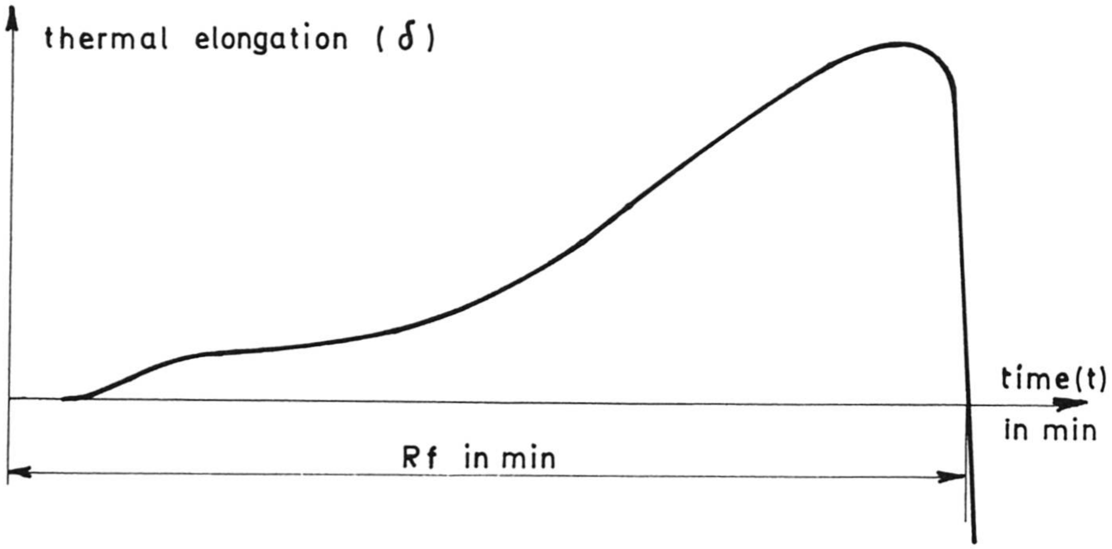

Fig. 4 Thermal elongation of a column in function of time.

b) Results of buckling tests at elevated temperature

The buckling tests at elevated temperature are divided in two series :

1° a series of eleven short columns with a slenderness ratio λ equal to 25. For this series no measurements of the actual yield stress were made and the guaranteed nominal yield stress was considered.

2° a series of eighteen columns with slenderness ratios comprised between 25 and 102. For this second series the actual yield stress of each column was measured.

The geometrical and mechanical properties of the 29 tested columns as well as the test results are summarized in table 3.

Table 3 Buckling tests in fire conditions
Mechanical and geometrical propecties of the test pieces
Test results

Column id.	Type of profile	Slenderness ratio λ (*)	Yield Strength N/mm^2	Area of the cross section A_a N/mm^2	Applied stress $\sigma_{cr,t}$ N/mm^2	R_f min	Critical temperature Θ (***) °C
1.1 **	HEA 300	25,23			137,3	17	610
1.2	HEA 300	25,23			137,3	115	553
1.3	HEA 300	25,23			137,3	157	541
1.4	HEA 300	25,23			137,3	185	559
1.5	HEB 300	24,93			157,0	123	492
1.6	HEB 300	24,93	Nominal value 235	Nominal value	176,6	110	444
1.7	HEB 300	24,93			157,0	146	510
1.8	HEB 400	25,54			157,0	135	578
1.9 **	HEB 300	24,93			157,0	17	498
1.10	HEB 300	24,93			157,0	59	582
1.11	HEB 300	24,93			150,5	160	560
2.1	HEB 300	24,93	274,0	14280	134,1	58	588
2.2	IPE 160	102,72	272,5	1997	56,5	97	564
2.3	IPE 160	102,72	272,5	1997	75,3	64	486
2.4	IPE 200	84,83	272,0	2812	69,9	92	559
2.5	IPE 200	84,83	272,0	2812	93,3	45	394
2.6	HEB 120	61,76	266,5	3327	104,7	55	519
2.7	HEB 120	61,76	266,5	3327	78,5	130	561
2.8	HEB 180	41,36	279,0	6120	92,3	90	616
2.9	HEB 180	41,36	279,0	6120	136,8	108	560
2.10	HEA 200	37,95	261,0	5301	125,8	85	565
2.11	HEA 300	25,23	267,5	10650	133,9	110	561
2.12	HEA 220	34,75	252,0	5990	162,2	116	502
2.13	HEB 200	37,77	218,0	7574	89,9	231	549
2.14	IPE 200	85,49	272,0	2765	117,0	102	250
2.15	HEB 140	53,49	247,0	4083	132,6	124	516
2.16	HEB 140	53,49	247,0	4083	90,9	115	576
2.17	IPE 220	72,22	273,0	3459	91,9	101	522
2.18	IPE 220	72,22	273,0	3459	118,5	82	508

(*) Column length exposed to fire : 3780 mm; buckling length : 1890 mm.

(**) Unprotected column.

(***) The critical temperature Θ corresponds to the failure time R_f defined in fig. 4

Most of the columns are insulated in order to have a slower temperature increase. With this method a more accurate temperature measurement is possible. Only two columns are unprotected (n° 1.1 and 1.9).

4.3. Comparison of the test results with the proposed design method

Knowing for each column the actual or nominal yield stress σ_r, the slenderness ratio λ and the critical temperature θ, it is easy to calculate the theoretical buckling stress at the temperature θ : $\sigma_{cr,\theta}$

$$\sigma_{cr,\theta} = \sigma_r . \overline{N}_\theta \text{ with } \overline{N}_\theta = \frac{\sigma_{r,\theta}}{\sigma_r} \overline{N}_{20} \quad \text{(equation (4))}$$

$$\text{and } \overline{N}_{20} = f(\overline{\lambda}_{20}) \quad \text{(equation (1))}$$

$$\overline{\lambda}_{20} = \lambda / \pi \sqrt{\frac{E}{\sigma_r}}$$

The analytical approach for the determination of $\sigma_{cr,\theta}$ is derived from the E.C.C.S. approach [5][6] which is based on the characteristic value of the mechanical properties of steel columns where the imperfection parameter is choosen in such way the buckling curves are characteristic curves with a 2,3 % confidence level.

A discrepancy arises when the fire resistance of a steel column is determined on one hand by a standard fire resistance test and on the other hand by an analytical approach based on characteristic values - Generally such an analytical method gives more conservative values than the test method.

PETTERSON and WITTEVEEN [13] have developed a correction procedure which leads to an improved consistency between an analytically and experimentally determined fire resistance. In this procedure, for steel columns, the calculated critical buckling stress $\sigma_{cr,\theta}$ is multiplied by a factor of magnification f which includes corrections with respect to representative deviations from the assumptions listed for the real structural element.

The corrected buckling stress

$$f.\sigma_{cr,\theta}$$

obtained in this way, can be considered as approximately consistent with the corresponding stress determine in a fire resistance test.

The factor of magnification f columns is given by the formulas [11]

$$f = 1 + \frac{1}{1500}\,\theta \qquad 0 < \theta < 300°\ C \qquad (5)$$

$$f = 1{,}2 \qquad 0 \geqslant 300°\ C \qquad (5\ bis)$$

Table 4 lists the comparative test ultimate stresses in fire conditions against the design stresses obtained from equation (4) or figure 2 and corrected with the magnification factor f.

It is noteworthy that a large majority of test buckling stresses in fire conditions are safely predicted by the design curves or by equation (4). It must be reminded however, that for the calculation of the theoretical limit load, the IPE profiles were in some way downgraded, as the curve "c" was used instead of "b", as usual at ambient temperature. Now, despite this apparently conservative measure, the IPE profiles do not show a substantially better behaviour that the HE shapes; two of the three test results which fall below the theoretical values correspond to IPE columns. It appears therefore reasonable at the present time to cover all classes of profiles by one simple curve corresponding to E.C.C.S. buckling curve c.

Table 4 Comparison between test results and the proposed design method

Column id.	$\bar{\lambda}$	σ_r N/mm^2	$\bar{N}_{20}$	θ °C	$\frac{\sigma_{r,\theta}}{\sigma_r}$	$\sigma_{cr,\theta}$ N/mm^2	$f.\sigma_{cr\theta}$ N/mm^2	Test result $\sigma_{cr,t}$ N/mm^2	(*) $\frac{\sigma_{cr,t}}{f.\sigma_{cr\theta}}$	(*)(*) $\frac{\sigma_{cr,t}}{f.\sigma_{cr\theta}}$
1.1	25,23		0,965	610	0,25	56,69	68,03	137,3		2,01
1.2	25,23		0,965	553	0,37	83,91	100,69	137,3		1,36
1.3	25,23		0,965	541	0,40	90,71	108,85	137,3		1,26
1.4	25,23		0,965	559	0,36	81,64	97,97	137,3		1,40
1.5 Serie 1	24,93	Nominal value 235	0,965	492	0,49	111,12	133,34	157,0		1,18
1.6	24,93		0,965	444	0,58	131,53	157,84	176,6		1,12
1.7	24,93		0,965	510	0,46	104,32	125,18	157,0		1,25
1.8	25,54		0,965	578	0,32	72,57	87,08	157,0		1,80
1.9	24,93		0,965	498	0,48	108,85	130,62	157,0		1,20
1.10	24,93		0,965	582	0,31	70,30	84,36	157,0		1,86
1.11	24,93		0,965	560	0,36	81,64	97,97	150,5		1,54
	Mean value									1,452
2.1	24,93	274,0	0,956	588	0,30	78,58	94,30	134,1	1,42	1,66
2.2	102,72	272,5	0,447	564	0,35	42,63	51,16	56,5	1,10	1,16
2.3	102,72	272,5	0,447	486	0,51	62,12	74,54	75,3	1,01	1,08
2.4	84,83	272,0	0,553	559	0,36	54,15	64,98	69,9	1,08	1,15
2.5	84,83	272,0	0,553	394	0,66	99,27	119,12	93,3	0,78	0,85
2.6	61,76	266,5	0,719	519	0,44	84,31	101,17	104,7	1,03	1,13
2.7	61,76	266,5	0,719	561	0,36	68,98	82,78	78,5	0,95	1,05
2.8	41,36	279,0	0,856	616	0,23	54,93	65,92	92,3	1,40	1,61
2.9 Serie 2	41,36	Actual value 279,0	0,856	560	0,36	85,98	103,18	136,8	1,33	1,54
2.10	37,95	261,0	0,884	565	0,35	80,75	96,90	125,8	1,30	1,42
2.11	25,23	267,5	0,956	561	0,36	92,06	110,48	133,9	1,21	1,38
2.12	34,75	252,0	0,911	502	0,48	110,20	132,20	162,2	1,23	1,32
2.13	37,77	218,0	0,905	549	0,38	74,97	89,96	89,9	1,00	0,93
2.14	85,49	272,0	0,547	250	0,83	123,49	144,48	117,0	0,81	0,87
2.15	53,49	247,0	0,795	516	0,45	88,36	106,40	132,6	1,25	1,31
2.16	53,49	247,0	0,795	576	0,32	62,84	75,40	90,9	1,21	1,24
2.17	72,22	273,0	0,635	522	0,44	76,28	91,53	91,9	1,00	1,11
2.18	72,22	273,0	0,635	508	0,46	79,74	95,69	118,5	1,24	1,34
	Mean value								1,13	1,23

(*) $\sigma_{cr,\theta}$ is calculated with the actual value of the yield stress
(**) $\sigma_{cr,\theta}$ is calculated with the nominal value of the yield stress

The average values of the ratio of the experimental buckling stress ($\sigma_{cr,t}$) to the corrected calculated stress ($f.\sigma_{cr,\theta}$) are respectively :

- for the first test series of 11 columns with a slenderness ratio equal to 25 and a nominal yield stress of 235 N/mm^2 :

$$\left(\frac{\sigma_{cr,t}}{f.\sigma_{cr,\theta}}\right)_{av} = 1{,}452;\ s = 0{,}307;\ V = 21\ \%\ \text{and a variance of } 0{,}0857$$

- for the second test series of 18 columns with slenderness ratio comprised between 25 and 102 and using the actual value of the yield stress of each column

$$\left(\frac{\sigma_{cr,t}}{f.\sigma_{cr,\theta}}\right)_{av} = 1{,}13;\ s = 0{,}187;\ V = 17\ \%\ \text{and a variance of } 0{,}030$$

- for this second series of 18 columns the same calculation is made with a nominal yield stress of 235 N/mm^2 :

  $(\frac{\sigma_{cr,t}}{f.\sigma_{cr,\theta}})_{av}$ = 1,23; s = 0,238 and V = 20 % and a variance of 0,0530

- for the 29 tests and considering a nominal yield stress of 235 N/mm^2 :

  $(\frac{\sigma_{cr,t}}{f.\sigma_{cr,\theta}})_{av}$ = 1,31; s = 0,284 and V = 22 % and a variance of 0,0770

  Figure 5 shows the results of the 29 tests against the theoretical 45° straight line representing perfect concordance between test and theory.

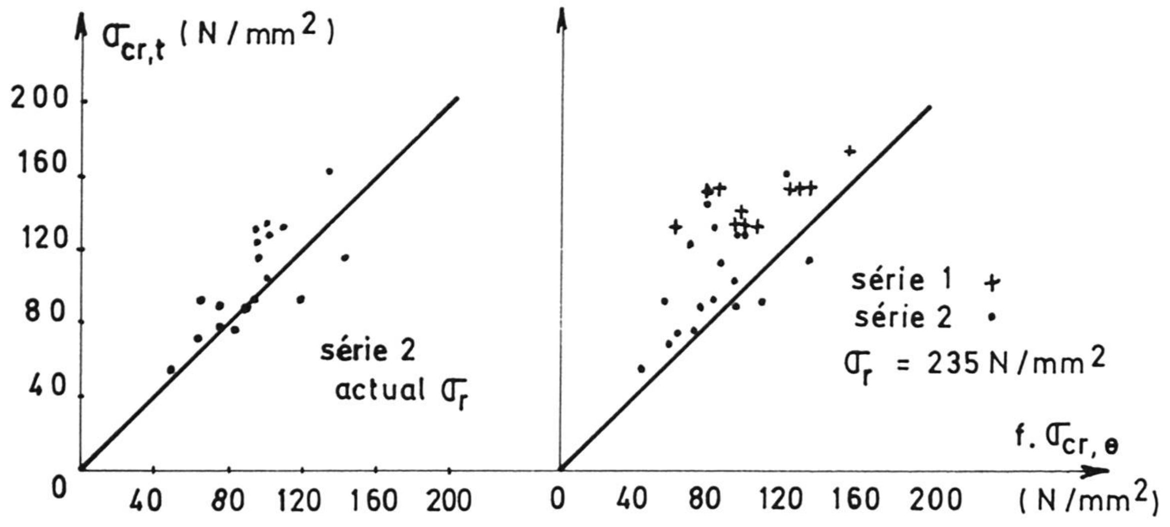

Fig. 5 Comparison between experimental and theoretical results

It can be concluded from this series of 29 tests, that the proposed simple design method for the critical loads of steel columns exposed to fire is in good agreement with the experimental results.

4.4. Comparison of the proposed method with tests made recently in Denmark

In a recent paper of the Institute of Building Technology and Structural Engineering of the Aalborg Universitetcenter a series of full scale tests on centrally loaded steel columns at elevated temperatures is described. [8]

The columns were tested in a horizontal position in a special furnace. In these tests the load was increased with a constant loading rate until buckling while the temperature of the furnace was kept constant (20° C; 200° C; 400° C and 500° C).

The results of these tests are summarized in table 5.

Table 5 Results of the danish buckling tests at elevated temperature

Column id.	Type of profile	λ	σ_r N/mm^2	$\sigma_{cr,t}$ N/mm^2	Θ °C	$\bar{N}_{20}$	$\frac{\sigma_{r,\Theta}}{\sigma_r}$	$\sigma_{cr,\Theta}$ N/mm^2	$\frac{\sigma_{cr,t}}{f.\sigma_{cr,\Theta}}$
H24.06	HEA 100	95	Nominal value [g] 240	108	400	0,5263	0,65	82,10	1,10
H24.07		95		175	200	0,5263	0,88	111,15	1,31
H24.09		94		60	550	0,5315	0,38	48,47	1,03
H24.10		95		110	500	0,5263	0,48	60,63	1,51
H36.02		144		86	200	0,3078	0,88	65,01	1,10
H36.03		143		67	400	0,3109	0,65	48,50	1,15
H36.04		143		36	550	0,3139	0,38	28,63	1,05
H36.05		143		62	440	0,3109	0,58	43,28	1,19
H42.01		167		29	550	0,2432	0,38	22,18	1,09
H42.02		166		58	400	0,2432	0,65	37,94	1,27
H42.03		167		50	460	0,2455	0,55	32,41	1,29
H42.05		167		60	200	0,2432	0,88	51,36	0,97
Mean value									1,17

The concordance of the danish test results with the proposed design method is also excellent. It can be seen that the average value of $\frac{\sigma_{cr,t}}{f.\sigma_{cr,\Theta}}$ for the 12 buckling tests on HEA profiles at elevated temperature is equal to 1,17 with a standard deviation of 0,16 and a coefficient of variation of 13 %.

5. CONCLUSIONS

A simple design method for steel columns under concentric loading in fire conditions is presented. The method is in accordance with the Recommandations of the European Convention of Constructional Steelwork (E.C.C.S.) for the design of steel columns at ambient temperature. The general equation of E.C.C.S. buckling curves is modified in order to take into account the effect of the temperature on the steel properties and the obtained design method is independant of the yield strength of the steel columns.The modification leads to the same expression as that used for bare steel columns and enables these curves to be used as the base design curves in fire conditions. For safety reasons, only curve "c" is recommended at the present time.

The method has been compared with a large number of experimental results obtained in Belgium and in Denmark, on steel columns in fire conditions with slenderness ratio varying between 25 and 167. A correction procedure has been introduced to achieve an improved consistency between the analytical and experimental fire resistance. The agreement is shown to be excellent on the safe side.

Due to the fact that up to now no criterion for buckling in fire conditions is available in national or international Standards, it is proposed to consider as time of failure, the time at which the thermal elongation is annihilated by the skrinkage of the column. This criterion has the advantage to be easy to apply and to measure and to be independent of the column length. It is not subject to interpretation and it is based on scientific experience.

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NOTATIONS

A : nominal area of cross section

A_a : actual area of cross section

E : modulus of elasticity

P_{cr} : column strength for axial load at ambient temperature

R_f : failure time of a column

V : coefficient of variation

s : standard deviation

t : time

Θ : critical temperature corresponding to the failure time R_f

σ_r : yield strength of steel at ambient temperature

$\sigma_{r,\Theta}$: yield strength of steel at temperature Θ

$\sigma_{cr,20}$: theoretical buckling stress at ambient temperature

$\sigma_{cr,t}$: experimental buckling stress at elevated temperature

$\sigma_{cr,\Theta}$: theoretical buckling stress at temperature Θ

λ : slenderness ratio

δ : thermal elongation

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