Stability of Steel Columns in Case of Fire: Experimental Evaluation

By J. M. Franssen,[1] D. Talamona,[2] J. Kruppa,[3] and L. G. Cajot[4]

Abstract: Two new series of experimental full-scale tests have been performed on steel columns at elevated temperature, either under small or large eccentricities. Those results have been introduced in a database, together with results from previous tests found in the literature. An analytical formula for the buckling coefficient presented as shown in an earlier paper has been calibrated here by comparison with the results of tests made with no or with small eccentricity. The analytical proposal for a P-M interaction curve presented in the companion paper has been compared in this paper with all the results from the database, and proved to provide a satisfactory agreement.

INTRODUCTION

The behavior of steel columns at elevated temperatures has been analyzed numerically by the writers [see Franssen et al. (1995); Talamona et al. (1997)]. This work has led to the proposal of an analytical formula for the buckling coefficient, defined as the ratio between the ultimate load and the plastic load, in the case of central loading and an analytical expression for the P-M interaction curve in the case of eccentric loading. The expression for the buckling coefficient contains one scalar parameter, the severity factor β, which has to be calibrated to ensure an appropriate safety level. It is also desirable that the interaction formula is validated against a large set of experimental results before any confidence can be placed in the proposed method. Those verifications and calibrations are necessary for various reasons.

- There is always the possibility that an error has been made in the formulation or numerical implementation of the theories that form the basis of the analysis, although the probability of occurrence of this kind of problem has been reduced by a comparison exercise (Franssen et al. 1994) between different computer programs, including the two codes used in the present study.
- Any analytical, numerical, or theoretical model is much more likely to be accepted by authorities if it is backed and supported by a set of experimental tests—although it can be questioned that a furnace test is closer to real life than is a numerical simulation.
- The numerical simulations have been made with characteristic values for initial out-of-straightness ($H/1{,}000$) and residual stresses (0.30 or 0.50×235 N/mm^2), which are likely to produce excessively severe results.
- The material properties of steel at elevated temperatures are linearly interpolated between values given every 100°C in EC3-1.2 (*Eurocode* 1995). It would be amazing if Mother Nature had provided us with a material having such a peculiar variation of its parameters. The model is therefore an approximation of reality.

Some experimental test results are available in the literature and will be considered in the comparison. To avoid possible bias in the conclusions, it is desirable to obtain an experimental base as wide as possible, in relation to the total number of tests as well as the number of different independent sources, i.e., different laboratories. As it was not possible to find all the necessary experimental situations, it was decided to perform the following two new series of experimental tests:

- One series on columns with a small eccentricity of load—The same cross section was used for all the tests and the length of the column was changed in order to analyze the effect of the buckling length. In most of the available test results, the dimensions of the furnace fix, the length of the column, and the slenderness are changed by means of changes of the end supports and/or the section type.
- One series on columns with very large eccentricities of load—This situation seldom has been analyzed experimentally in the past.

EXPERIMENTAL TEST RESULTS

New Series of Tests

Twenty-one fire tests were performed in Spain in the LABEIN laboratory [see Azpiazu and Unanue (1993)] and eight others at the Fire Station of CTICM (France).

The specimens were electrically heated by means of ceramic mat elements at a rate of

- 5°C/min for the tests made by LABEIN
- 10°C/min up to 400°C and 5°C/min beyond 400°C for the tests made by CTICM

Automatic control of separate heating elements was used in order to ensure a uniform temperature distribution along the length of the elements. The temperature was measured with thermocouples welded on the specimens at different cross sections, from three sections in the shortest specimens to seven in the longest ones. The number of measurement points on each column varied from 17 to 35 depending on the length of the column. In the case that somewhat lower temperatures were recorded near the supports, at 100 mm from the support for the shortest columns and 100 and 230 mm for the longer ones, the failure temperature of the element was estimated as the mean temperature of the thermocouples located in the central part of the column. Failure time was the time when the load could not be maintained by the hydraulic system. The load, axial elongation, and horizontal displacements at midheight were monitored during the test. Table 1 is a summary of the tests made in LABEIN (first part) and CTICM (second part).

A note to the reader follows:

[1]Res. Assoc., N.F.S.R. Belgium, Univ. of Liège, 6, Quai Banning, B-4000, Liège, Belgium.
[2]PhD, 11, rue de la tuilerie, 68.390 Sausheim, France.
[3]Dept. Head, CTICM, 78.470 St-Rémy-les-Chevreuse, BP 64, France.
[4]Res. Engr., ProfilARBED Recherches, Route de Luxembourg, Esch/Alzette, G. D. de Luxembourg.

Note. Associate Editor: Amde M. Amde. Discussion open until July 1, 1998. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on February 26, 1996. This paper is part of the ***Journal of Structural Engineering***, Vol. 124, No. 2, February, 1998. ©ASCE, ISSN 0733-9445/98/0002-0158–0163/$4.00 + $.50 per page. Paper No. 12695.

TABLE 1. Results of Tests Calculated by LABEIN and CTICM

Number[a] (1)	H (mm) (2)	θ_{cr} (°C) (3)	Buckling axis[b] (4)	e (mm) (5)	P (kN) (6)	f_{y_w} (N/mm²) (7)	$f_{y_{fl}}$ (N/mm²) (8)	Section (9)	b (mm) (10)	h (mm) (11)	t_w (mm) (12)	t_{fl} (mm) (13)	$i2$[c] (mm) (14)	$i3$[d] (mm) (15)	$i4$[e] (mm) (16)
AL1	513	20	W	5	537	300	280	HEA100	101.92	99.20	6.10	7.80	—	0.00	—
BL1	513	532	W	5	362	300	286.5	HEA100	101.85	98.85	5.92	7.61	—	0.00	—
CL1	513	694	W	5	110	316	292.5	HEA100	101.78	99.07	6.43	7.80	—	0.00	—
DL1	513	863	W	5	40	309	282.5	HEA100	102.28	99.12	6.13	7.68	—	0.00	—
AL3	1,270	20	W	5	490	300	280	HEA100	101.95	99.08	5.97	7.67	—	0.00	—
BL3	1,272	390	W	5	292	300	286.5	HEA100	101.93	98.90	5.97	7.64	—	0.20	—
CL3	1,271	474	W	5	251	316	292.5	HEA100	101.90	99.25	6.13	7.82	—	0.40	—
DL3	1,269	749	W	5	24	309	282.5	HEA100	102.15	99.15	6.02	7.73	—	0.30	—
SL40	2,020	525	W	5	170	286	280	HEA100	—	—	—	—	—	—	—
SL41	2,026	509	W	5	174	286	280	HEA100	101.84	98.97	5.73	7.58	0.60	0.70	0.70
SL42	2,020	485	W	5	171	286	280	HEA100	101.82	99.04	5.76	7.61	0.90	1.70	0.90
SL43	2,021	20	W	5	366	286	280	HEA100	101.84	98.89	5.80	7.57	−0.40	0.00	0.00
SL44	2,023	495	W	5	173	286	280	HEA100	101.68	99.17	5.73	7.60	0.50	1.10	0.60
AL5	2,770	457	W	5	127	300	280	HEA100	101.94	99.06	5.78	7.68	−0.07	−0.04	0.60
BL5	2,772	587	W	5	73	300	286.5	HEA100	101.76	98.95	5.76	7.62	0.30	1.00	0.80
CL5	2,771	587	W	5	34	316	292.5	HEA100	102.03	99.25	5.98	7.76	0.70	0.80	0.80
DL5	2,772	886	W	5	7.7	309	282.5	HEA100	102.15	99.16	5.96	7.72	0.80	1.60	0.80
AL6	3,510	20	W	5	176	300	280	HEA100	101.99	99.08	5.79	7.66	−0.70	−0.40	0.60
BL6	3,510	446	W	5	105	300	286.5	HEA100	101.88	98.93	5.93	7.63	0.80	1.00	0.30
CL6	3,510	493	W	5	90	316	292.5	HEA100	102.05	99.12	5.94	7.71	0.70	0.80	0.80
DL6	3,510	727	W	5	11.5	309	282.5	HEA100	101.68	99.17	5.73	7.60	0.80	1.60	0.80
*P*1	4,000	664	W	100	100	314	275	HEB200	200.2	201.4	9.04	15.04	−0.5	−0.5	−0.5
*P*2	4,000	575	W	300	100	314	275	HEB200	200.3	201.4	9.04	15	−1	−3	−3
*P*3	2,000	599	S	650	100	314	275	HEB200	200.3	201.3	9.04	14.96	−0.5	−0.5	−0.5
*P*4	2,000	537	W	300	150	314	275	HEB200	200.3	201.4	9.04	15	1.5	0.5	0.5
*P*5	2,000	753	S	250	100	344	271	HEM160	163.4	180.2	14	22.68	0.5	0	0
*P*6	5,000	572	S	500	100	344	271	HEM160	163.5	180.4	14	22.62	2	3	1.5
*P*7	2,000	539	S	100	160	304	260	HEA140	141.5	137.5	5.61	8.96	0	0	−0.5
*P*8	5,000	507	S	100	100	304	260	HEA140	139.8	133.8	5.61	8.28	1	0.5	0.5

[a]Number of the test in the test report.
[b]S stands for major axis; W stands for minor axis.
[c]Imperfection at $H/4$.
[d]Imperfection at $H/2$.
[e]Imperfection at $3H/4$.

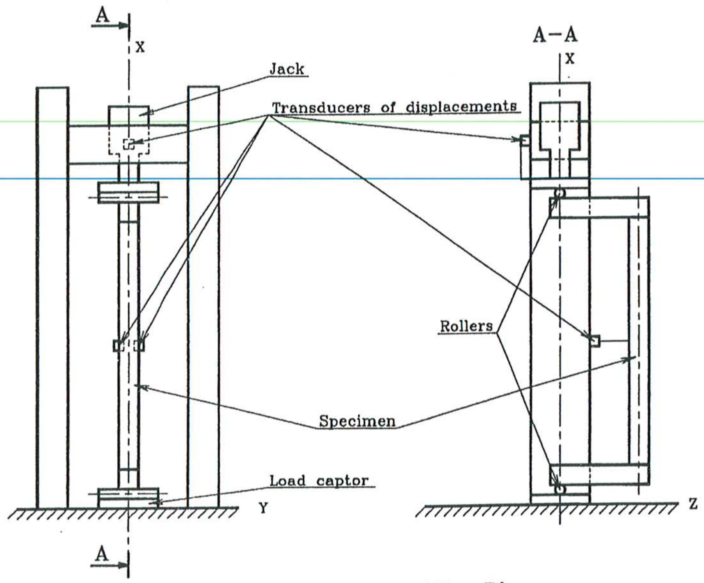

FIG. 1. Experimental Test Rig

- The values given for the failure temperature θ_{cr}, the yield strengths f_{y_w} and $f_{y_{fl}}$, and the dimensions of the section (b, h, t_w, and t_{fl}) result from the average between several measured values.
- The elements were placed vertically. They were turned in such a way that the effect of the imperfection was added to the effect of the load eccentricity if the value of $i2$, $i3$, or $i4$ has a positive sign in Table 1.
- The residual stresses in the profiles were measured by cutting some short pieces of profiles in small longitudinal bars and measuring the elongation in the bars when they are separated from the rest of the section. They were found to be rather low when compared to what is usually reported. The maximum observed values were on the order of magnitude of 0.10×235 N/mm².

Geometrical dimensions of the section and geometrical imperfections, i.e., out of straightness, of the column were measured for each tested element. Residual stresses and yield strengths were measured from coupons belonging to the same production as the tested element.

The load was applied before the test and was kept constant during the heating. It was applied with a well-defined eccentricity through a very sharp knife support (or rollers at CTICM). The end rotation around this axis was free, and the rotation around the other axis can be regarded as restrained by the action of the support (see Fig. 1). Measurements of lateral deflections in both directions as well as the examination of the deformed columns after the tests confirmed that no rotation occurred in the direction of the major axis. Different buckling lengths H from 510 to 5,000 mm were considered, with different load levels.

Due to the fact that tests AL1, AL3, SL43, and AL6 were performed at ambient temperature; that test SL40 was a preliminary test performed to verify the heating equipment on an element that had not been accurately measured; and that some technical difficulties led to uncertainties in tests DL3 and CL5,

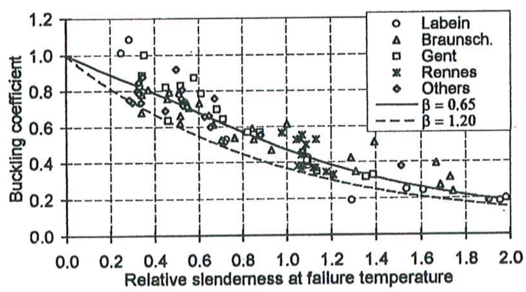

FIG. 2. Experimental Buckling Coefficients

14 tests from those performed by LABEIN at elevated temperature with a small eccentricity remain for consideration in the database. Results of these tests are plotted distinctly in Fig. 2.

Results from Literature

In addition to these tests, a survey of the available literature allowed the formulation of a database of compression tests conducted at elevated temperatures on steel elements. Tests were not considered if the actual yield strength of the specimen has not been measured [see Olesen (1980); Vandamme and Janss (1981)]. Nominal values were taken for geometrical properties when they had not been measured. The influence of the yield strength has been found to be overwhelming when compared to the influence of the geometrical properties (Talamona et al. 1995). Some tests also have been eliminated because of the presence of a very nonuniform temperature distribution (Aasen 1985). These tests are nevertheless valuable for the validation of numerical tools, provided time is spent to introduce the measured temperature distribution (Poh and Bennetts 1995). Finally, 78 experimental test results on columns with small eccentricities were obtained: One from Borehamwood (BSC 1988), 16 from Gent (Janss and Minne 1982), three from Stuttgart (Cajot et al. 1992), and 25 from Braunschweig (Technical 1977); 14 from Rennes (Aribert and Randriantsara 1980); and, finally, 19 from Berlin (Knublauch et al. 1974a,b). With the 14 new tests performed in Bilbao, the database thus comprises 92 test results on columns loaded with a small eccentricity.

PROPOSAL OF ANALYTICAL FORMULA

The analytical proposal resulting from the numerical analysis has been presented in Talamona et al. (1997). It is repeated here for clarity.

The proposed formulas have been established for the temperature range 400–800°C. They can safely be applied for temperatures higher than 800°C or lower than 400°C. For the latter case, however, it is preferable and more logical to interpolate linearly on the temperature between ultimate load given at 100°C—similar to the value at 20°C—by ambient temperature recommendations and ultimate load at 400°C given by the following formulas. This is due to the fact that the following formula, being derived from observations made at elevated temperatures, does not yield the same critical load as the formula established for ambient temperature when applied for a temperature of 20°C.

Centrally Loaded Column

$$P_u(\theta) = \chi(\theta)K_{fy}(\theta)f_y\Omega \qquad (1)$$

where $P_u(\theta)$ = ultimate load of axially loaded column at elevated temperature; Ω = cross-sectional area of profile; and f_y = yield strength at room temperature.

$$\chi(\theta) = \frac{1}{\varphi(\theta) + \sqrt{\varphi^2(\theta) - \bar{\lambda}^2(\theta)}} \qquad (2)$$

$$\bar{\lambda}(\theta) = K_\lambda(\theta)\bar{\lambda} \qquad (3)$$

$$K_\lambda(\theta) = \sqrt{K_{fy}(\theta)/K_E(\theta)} \qquad (4)$$

$$\bar{\lambda} = \frac{H/i}{\pi\sqrt{E/f_y}} \qquad (5)$$

where i = radius of gyration of relevant plane of buckling; and E = Young's modulus at room temperature.

$$\varphi(\theta) = \frac{1}{2}[1 + \alpha\bar{\lambda}(\theta) + \bar{\lambda}^2(\theta)] \qquad (6)$$

where α = imperfection factor; and β = severity factor, a parameter to be determined in order to ensure appropriate safety level.

$$\varepsilon = \sqrt{235/f_y} \quad (f_y \text{ in N/mm}^2 \text{ in this equation}) \qquad (7)$$

$K_{fy}(\theta)$ and $K_E(\theta)$ describe the decrease of yield strength and Young's modulus with elevation of temperature. Their values are taken from EC3-1.2 (*Eurocode* 1995) and are listed in Table 2 for temperature steps of 100°C. Linear interpolation is allowed for $K_{fy}(\theta)$ and $K_E(\theta)$, and gives a very good approximation for $K_\lambda(\theta)$ if used instead of (4).

TABLE 2. Parameters of Stress-Strain Relationship

Temperature (1)	$K_{fy}(\theta)$ (2)	$K_E(\theta)$ (3)	$K_\lambda(\theta)$ (4)
20	1.00	1.00	1.000
100	1.00	1.00	1.000
200	1.00	0.90	1.050
300	1.00	0.80	1.118
400	1.00	0.70	1.195
500	0.78	0.60	1.140
600	0.47	0.31	1.231
700	0.23	0.13	1.330
800	0.11	0.09	1.106
900	0.06	0.0675	0.943

Eccentrically Loaded Columns

$$\frac{P}{P_u(\theta)} + k\frac{M}{M_u(\theta)} \le 1 \qquad (8)$$

where P = axial load; M = bending moment, or maximum bending moment in case of nonuniform moment distribution; and $P_u(\theta)$ is taken from (1).

$$M_u(\theta) = W_p K_{fy}(\theta)f_y \qquad (9)$$

where W_p = plastic modulus of section for relevant plane of bending.

$$k = 1 - \mu\frac{P}{P_u(\theta)} \le 3 \qquad (10)$$

$$\mu = -0.84\bar{\lambda}(\theta)(\psi + 1) - 0.50\psi + 1.00 \le 0.80, \quad \text{buckling around the minor axis} \qquad (11)$$

$$\mu = -1.40\bar{\lambda}(\theta)(\psi + 1) - 0.31\psi + 1.09 \le 0.80, \quad \text{buckling around the major axis} \qquad (12)$$

$$\bar{\lambda}(\theta) = K_\lambda(\theta)\bar{\lambda} \qquad (13)$$

with $\bar{\lambda} \le 1.10$ in the case of buckling around the major axis.

$$\psi = \frac{M_2}{M_1} \quad \text{with} \quad |M_1| \ge |M_2| \qquad (14)$$

As given in (14), M_1 and M_2 = bending moments at supports.

CALIBRATION

One scalar parameter is present in the proposed formula to calibrate the model, the severity factor β, which has an influence on the buckling coefficient χ, i.e., on the ultimate load under central compression [see (2), and (6)]. Fig. 3 shows the evolution of the buckling curve at elevated temperature for different values of β and for a yield strength at 20°C of 355 N/mm^2.

Strictly speaking, the severity factor should be experimentally evaluated only from tests made on centrally loaded columns. As the number of such tests is rather limited, results from tests made with a small eccentricity also have been con-

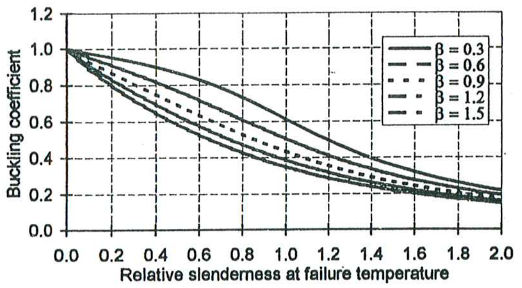

FIG. 3. Different Shapes of Analytical Buckling Curve

sidered. The criterion to decide whether the eccentricity is small or not was based on the interaction formula of EC3-1.1 (*Eurocode* 1992), which has a form similar to (8). The eccentricity was judged as small if the term comprising P is at least equal to three times the term comprising M in the interaction formula. Now that a new interaction formula has been proposed, (8)–(14), consideration can be given to reassessing the value of β with this new formula for evaluation of tests with small eccentricities. This has not been done because it has been shown that if results from purely centrally loaded columns are plotted distinctly from tests made with a small eccentricity, all the results form only one group (see Fig. 4). In other words, the choice of the interaction formula for the evaluation of β is not very important, provided tests with a small eccentricity are considered.

For each experimental test i, it is possible to calculate the experimental buckling coefficient as follows:

$$\chi_i^{\text{exp}} = \frac{P_i^{\text{exp}}}{P_{p,i}(\theta)} \quad (15)$$

where P_i^{exp} = load applied during test; and $P_{p,i}(\theta)$ = squash load at failure temperature θ, evaluated by

$$P_{p,i}(\theta) = K_{fy}(\theta) f_y \Omega \quad (16)$$

The results are the points represented in Fig. 2, where the distinction is made between the results of tests performed in different laboratories.

It appears that no test series has produced results that are systematically higher or lower than the others, although for results from Braunschweig, for example, results are higher for long columns and lower for short columns. The opposite tendency seems to characterize tests from Gent.

For the whole set of results, a significant variation around the mean value is observed. For each relative slenderness, the variation is about 0.15. Some tests even result in an experimental buckling coefficient greater than unity. This can be explained by uncertainties in experimental values of the cross-sectional area, of yield strength at room temperature, of failure temperature, of applied load, by a small undetected degree of rotational restraint at the supports, or by the fact that the decrease of yield strength with temperature may exhibit some variation from the function $K_{fy}(\theta)$ considered for the evaluation of the tests.

The variations around the mean values, plus the different pattern shown by the results from different laboratories, explain that it is possible to derive very different conclusions from a relatively small number of results coming from one source.

The tests made in Rennes were performed on columns with the same length, and therefore the same relative slenderness calculated at ambient temperature. If Fig. 2 were plotted with the relative slenderness at 20°C on the horizontal axis, (5), the representative points would fall on the same vertical line. When the relative slenderness is evaluated at the failure temperature, (3), it appears that the highest buckling coefficients correspond to the lowest slenderness and the lowest coefficients to the highest slenderness (see Fig. 2). It can be seen that the points are closer to the average line. This is an experimental confirmation of what has been observed in the numerical analysis—that the variation is reduced when the results are calculated as a function of the relative slenderness calculated at the failure temperature.

The two curves drawn in Fig. 2 are not derived from the experimental results, but calculated according to (1). The value of 1.20 for β, which safely covered the numerical results obtained with characteristic imperfections in Talamona et al (1997), also covers almost all of the experimental results. Those results are, in general, more favorable than the analytical results because (1) real specimens seldom have characteristic imperfections; (2) even if one of the 2 imperfections (either residual stresses or out-of-straightness) may have a characteristic value, it is VERY SELDOM that both imperfections have simultaneously a characteristic value.

Once a value has been chosen for the severity factor β, it is possible to calculate for each test a ratio between the analytical and experimental buckling coefficient—i.e., between the calculated and the applied load—plus an average value and a standard deviation for the whole distribution. The value of β = 0.65 leads to an average of 0.99, slightly on the safe side, and a coefficient of variation of 16.7%.

The analytical curve has been drawn in Fig. 2 with those two values of the imperfection factor and for a yield strength equal to 277 N/mm^2, the average value for the 92 tests. It must be mentioned that Fig. 2 gives an excessive impression of variation around the mean value because the analytical curve could be drawn for one single value of the yield strength. In fact, when calculating the ratio of the analytical and experimental tests, the actual value of the yield strength is considered for each test, which tends to reduce the scatter. This unfortunately cannot be put on a single graph.

Fig. 4 shows the experimental results when discrimination is made between tests where the nominal eccentricity of the load was 0 and those tests where there was a small but acceptable eccentricity. It can be seen that the general trend is the same, which is a justification of the fact that tests with small eccentricities were taken into account in the calibration of the formula established for centrally loaded columns.

Fig. 5 is an experimental validation of the fact that increasing nominal yield strength favorably influences the buckling coefficient. Tests are plotted in Fig. 5 only if the yield strength at ambient temperature differs by more than 10% from the average value of the complete test series. It can be seen that

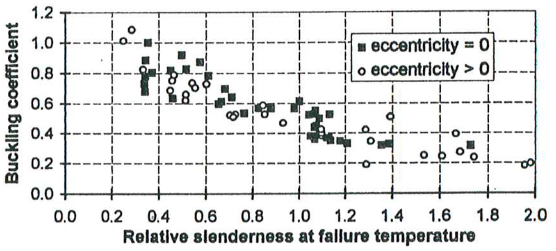

FIG. 4. Influence of Small Eccentricity

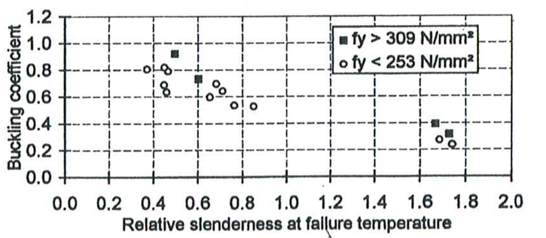

FIG. 5. Influence of Yield Strength

the four tests with a high yield strength have higher buckling coefficients than do the 12 tests with a low yield strength.

VERIFICATION OF PROPOSAL

The calibration has been made on the central load only. The global formula, including the bending term, has to be compared with experimental evidence. When all admissible results are considered, no matter the eccentricity of the load, the database comprises 156 results of full-scale experimental tests. For each test, the failure temperature corresponding to the applied load has been estimated by the new analytical proposal. Each test is represented by one point in Fig. 6. Of the evaluated failure temperatures, 112 are within 10% of the experimental failure temperature (expressed in degrees centigrade).

Fig. 7 is another presentation of the comparison. The histogram shows that most of the points are on the safe side and that only 12 of them overestimate the failure temperature by more than 10%. This is another reason why the value of 0.65 could be accepted for the severity factor. When bending moments are introduced (and they are practically always present in real situations), a new safety margin appears compared to the cases of centrally loaded columns. The average of all temperature ratios, including the 92 centrally loaded test results, is now 94.5%. This means that the average value of the temperature ratio, when only eccentrically loaded columns are considered, is 88%. This figure is obtained from the following equation:

$$156 \text{ tests} \times 94.5\% = 92 \text{ centrally loaded tests} \times 99\% + 64 \text{ eccentrically loaded tests} \times 88\% \quad (17)$$

APPLICATION

As an application example, it is shown how the axial load $P1$ and the eccentrical load $P2$ of the columns defined in Fig. 8 are calculated in the case of a failure temperature of 538°C (1,100°F).

1. The material properties at 538°C are interpolated from the values given in Table 2

$$K_{fy}(538) = 0.662 \quad \text{and} \quad K_E(538) = 0.490$$

2. Eq. (4) yields

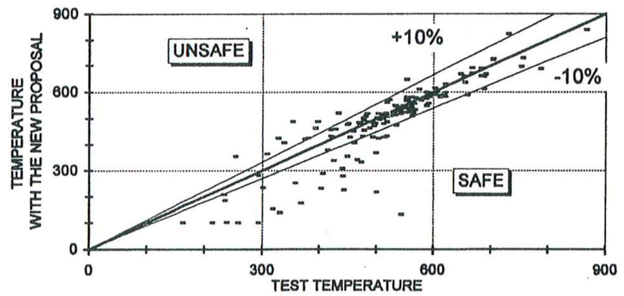

FIG. 6. Comparison of All Available Tests

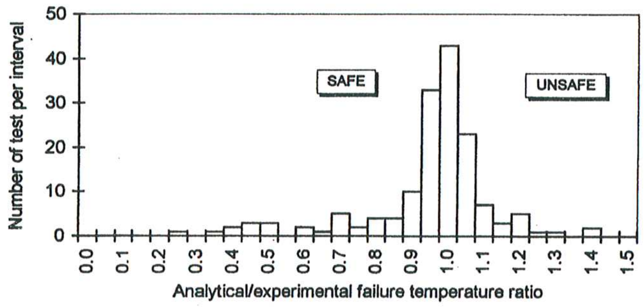

FIG. 7. Histogram of All Available Tests

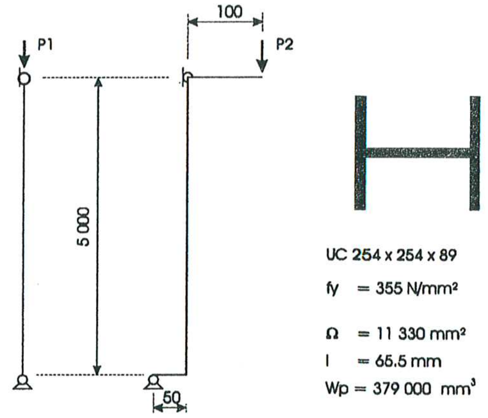

FIG. 8. Application Example

$$K_\lambda(538) = (0.662/0.490)^{0.5} = 1.162$$

3. The relative slenderness at room temperature is given by (5)

$$\bar{\lambda} = (5{,}000/65.5)/[\pi(210{,}000/355)^{0.5}] = 76.3/76.4 = 0.999$$

4. The relative slenderness at 538°C is given by (3)

$$\bar{\lambda}(538°\text{C}) = 1.162 \times 0.999 = 1.163$$

5. Eqs. (6) and (7) yield the imperfection factor

$$\alpha = 0.65 \times (235/355)^{0.5} = 0.65 \times 0.814 = 0.529$$

6. Eqs. (6) and (2) yield the buckling coefficient

$$\varphi(538°\text{C}) = 0.5 \times (1 + 0.529 \times 1.163 + 1.163^2) = 1.484$$

$$\chi(538°\text{C}) = 1/[1.484 + (1.484^2 - 1.163^2)^{0.5}] = 0.416$$

7. The ultimate axial load is calculated according to (1)

$$P1(538°\text{C}) = 0.416 \times 0.662 \times 355 \text{ N/mm}^2 \times 11{,}330 \text{ mm}^2 = 1{,}108 \text{ kN}$$

8. In case of eccentrical loading, the ratio between the two moments is given by (14), or by the ratio between the eccentricities

$$\psi = -50/100 = -0.50$$

9. Parameter μ is given by (11)

$$\mu = -0.84 \times 1.163(1 - 0.50) - 0.50 \times (-0.50) + 1.00 = 0.762$$

10. The ultimate bending moment is given by (9)

$$M_u(538°\text{C}) = 379{,}000 \times 0.662 \times 355 = 89{,}000 \text{ kN}\cdot\text{mm}$$

11. Eq. (8) has to be solved, taking (10) into account, to find the ultimate load

$$\frac{P}{1{,}108} + \left(1 - 0.762\frac{P}{1{,}108}\right)\frac{100P}{89{,}000} = 1$$

12. The solution is $P2 = 659$ kN.

CONCLUSION

A proposal has been made for evaluating the ultimate load bearing capacity or the failure temperature of steel columns subjected to fire. This proposal, based on extensive numerical study of the problem, has been calibrated against 92 experimental tests made on centrally loaded columns in order to obtain, on average, the same results with the formula as those observed in the tests. When test results from eccentrically loaded columns are also considered, an additional safety margin appears to be provided by the analytical proposal.

The parameters that were identified as significant by the numerical study have been confirmed by the comparison with experimental tests.

ACKNOWLEDGMENTS

This work was sponsored by the European Convention for Coal and Steel. This research project involves L. Twilt from TNO in The Netherlands, who is investigating the effect of nonsymmetric heat exposure. The writers acknowledge his contribution to the discussions on the subject reported in the present work.

APPENDIX I. REFERENCES

Aasen, B. (1985). "An experimental study on steel columns behaviour at elevated temperatures." The Norwegian Inst. of Technol., Trondheim, Norway.

Aribert, J. M., and Randriantsara, C. (1980). "Etude du flambement à des températures d'incendie. Action du fluage." *Constr. Métallique*, CTICM, 4, 3–22.

Azpiazu, W., and Unanue, J. A. (1993). "Buckling curves for hot rolled *H* profiles submitted to fire." *Rep. 97.798-0-ME/V*, LABEIN; Bilbao, Spain.

BSC Swinden Laboratories. (1988). *Compendium of UK standard fire test data: Unprotected structural steel—1.* Rotherham.

Cajot, L. G., Chantrain, P., Mathieu, J., and Schleich, J. B. (1992). "REFAO-III, practical design tools for unprotected steel columns submitted to ISO-fire." *Res. 7210-SA/505, Final Rep. Eur 14348 EN*, C.E.C., Luxembourg.

Eurocode 3: Design of steel structures: Part 1.1. General rules and rules for buildings. (1992). Draft ENV 1993-1-1, Commission of the Eur. Communities, Brussels, Belgium.

Eurocode 3: Design of steel structures: Part 1.2. General rules. Structural fire design. (1995). Draft ENV 1993-1-2, Commission of the Eur. Communities, Brussels, Belgium.

Franssen, J. M., et al. (1994). "A comparison between five structural fire codes applied to steel elements." *Proc., 4th Int. Symp. Fire Safety Sci.*, T. Kashiwagi, ed., IAFSS, Gaithersburg, 1125–1136.

Franssen, J. M., Schleich, J. B., and Cajot, L. G. (1995). "A simple model for the fire resistance of axially-loaded members according to Eurocode 3." *J. Constr. Steel Res.*, 35, 49–69.

Janss, J., and Minne, R. (1982). "Buckling of steel columns in fire conditions." *Fire Safety J.*, 4(4), 227–235.

Knublauch, E., Rudolphi, R., and Stanke, J. (1974a). "Theoretische ermittlung der feuerwiderstandsdauer von stahlstützen und vergleich mir versuchen. 1. Teil: Berechnung der stahltemperatur von stahlstützen." *Der Stahlbau*, 6, 175–182.

Knublauch, E., Rudolphi, R., and Stanke, J. (1974b). "Theoretische ermittlung der feuerwiderstandsdauer von stahlstützen und vergleich mir versuchen. 2. Teil: Bestimmung der kritischen stahltemperatur." *Der Stahlbau*, 8, 249–254.

Olesen, F. B. (1980). "Fire tests on steel columns." Inst. of Build. Technol. and Struct. Engrg., Aalborg, Denmark.

Poh, K. W., and Bennetts, I. D. (1995). "Behavior of steel columns at elevated temperatures." *J. Struct. Engrg.*, ASCE, 121(4), 676–684.

Talamona, D., Franssen, J. M., and Recho, N. (1995). "Buckling of eccentrically loaded steel columns submitted to fire." *Proc., Int. Conf. on Fire Res. and Engrg.*, P. Lund, ed., SFPE, Boston, 533–538.

Talamona, D., Franssen, J. M., Schleich, J. B., and Kruppa, J. (1997). "Stability of steel columns in case of fire: Numerical modeling." *J. Struct. Engrg.*, ASCE, 123(6), 713–720.

Technical University of Braunschweig. (1977). "Sonderforshchungsbereich 148: Brandverhalten von bauteilen." Tech. Univ. of Braunschweig.

Vandamme, M., and Janss, J. (1981). "Buckling of axially loaded steel columns in fire conditions." *IABSE Periodica*, 3/1981, 82–95.

APPENDIX II. NOTATION

The following symbols are used in this paper:

b = width of cross section;
E = Young's modulus at room temperature;
e = eccentricity of load;
f_y = yield strength at ambient temperature;
$f_{y_{fl}}$ = yield strength in flange at ambient temperature;
f_{y_w} = yield strength in web at ambient temperature;
H = total length of column;
h = depth of cross section;
i = radius of gyration of cross section;
$K_E(\theta)$ = ratio between $E(\theta)$ and E(—);
$K_{fy}(\theta)$ = ratio between $f_y(\theta)$ and f_y(—);
$K_\lambda(\theta)$ = defined by (6) as function of $K_E(\theta)$ and $K_{fy}(\theta)$(—);
M = applied bending moment (N/mm);
M_u = ultimate bending moment of column without axial load (N/mm);
P = applied axial load (N);
P_p = plastic load of cross section (N);
P_u = ultimate load of centrally loaded column (N);
t_{fl} = thickness of flange (mm);
t_w = thickness of web (mm);
W_p = plastic modulus of cross section (mm^3);
α = imperfection factor (—);
β = severity factor (—);
θ = steel temperature (°C);
$\bar{\lambda}$ = relative slenderness of column, evaluated at room temperature (—);
$\bar{\lambda}(\theta)$ = relative slenderness of column, evaluated at failure temperature (—);
$\chi(\theta)$ = buckling coefficient calculated at failure temperature (—);
ψ = ratio between lowest and highest bending moment (—); and
Ω = cross-sectional area of section (mm^2).