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An approach for calculating the failure loads of unprotected concrete filled steel columns exposed to fire

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Abstract. This paper deals with the development of an approach for evaluating the squash load and rigidity of unprotected concrete filled steel columns at elevated temperatures. The current approach of evaluating these properties is reviewed. It is shown that with a non-uniform temperature distribution, over the composite cross-section, the calculations for the squash load and rigidity are tedious in the current method. A simplified approach is proposed to evaluate the temperature distribution, squash load, and rigidity of composite columns. This approach is based on the model in Eurocode 4 and can conveniently be used to calculate the resistance to axial compression of a concrete filled steel column for any fire resistance time. The accuracy of the proposed approach is assessed by comparing the predicted strengths against the results of fire tests on concrete filled circular and square steel columns. The applicability of the proposed approach to a design situation is illustrated through a numerical example.

Key words: fire resistance calculation; failure loads; HSS columns; concrete filled; steel columns; squash load evaluation; high temperature properties.

1. Introduction

The advantages of concrete filled steel columns are well recognised in view of their high load carrying capacity, fast construction, small cross-section, and high fire resistance (Klingsch and Weurker 1995, Lie and Kodur 1996). When properly designed, the use of concrete filled steel columns may eliminate the need for external fire protection to steel and this will lead to aesthetically pleasing construction of exposed steelwork.

The traditional method to assess the fire resistance of composite columns is based on the results of standard fire resistance tests (ASTM 1990, UL 1982), which can be time consuming and expensive.

In recent years, the use of calculation methods for fire resistance evaluation is gaining wide

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acceptance. These calculations follow well accepted engineering principles. Generally, a composite column is assumed to be exposed to the standard fire. The temperature distribution in the composite column is determined from a heat transfer analysis. Using the material stress-strain relationships at high temperatures, a structural analysis is then carried out to determine the reduced load carrying capacity of the composite column. The fire resistance of the column is the standard fire exposure time at which the column load carrying capacity decreases to the level of the applied load. These calculations may be performed at different levels of complexity ranging from detailed finite element analysis (Kodur and Lie 1996a, Lie and Chabot 1990, Wang and Moore 1995), to give a complete history for the stress, strain and deflection in the column, to a design procedure in which only the column load carrying capacity is calculated (CEN 1994).

Some of the recent studies (Lie and Kodur 1996) have focused on developing simple design equations for evaluating the fire resistance of concrete filled steel columns. However, these methods are based on empirical relationships and have some limited applicability.

In this paper the current approach of evaluating the resistance to axial compression of a concrete filled steel column is reviewed by comparing the predictions from this approach with test data. A simplified approach is proposed for calculating the column squash load and rigidity at elevated temperatures.

2. Current approach of evaluating column resistance to axial compression

2.1. Description of the method

According to the recommendations given in Clause 4.3.6 of Eurocode 4 Part 1.2 (CEN 1994), the column resistance to axial compression at elevated temperatures is expressed as:

$$N_T = \chi_T N_{u,T} \tag{1}$$

where N_T and $N_{u,T}$ are the resistance to axial compression and squash load of the column at elevated temperatures, respectively. χ_T is the column strength reduction coefficient and is a function of the relative slenderness of the column, $\bar{\lambda}_T$, which is defined as:

$$\bar{\lambda}_T = \sqrt{\frac{N_{u,T}}{N_{cr,T}}} \tag{2}$$

in which $N_{cr,T}$ is the column Euler load at elevated temperature and is defined by:

$$N_{cr,T} = \frac{\pi^2 (EI)_T}{L^2} \tag{3}$$

where $(EI)_T$ is the column rigidity at elevated temperature and L its effective length.

The three steps associated with the above method are:

(1) adopting a column buckling curve.

(2) determining the temperature distribution in the column cross-section.

(3) calculating the column squash load and rigidity.

The assumptions and procedure associated with the above steps play a significant role in the calculations and hence they are discussed in detail.

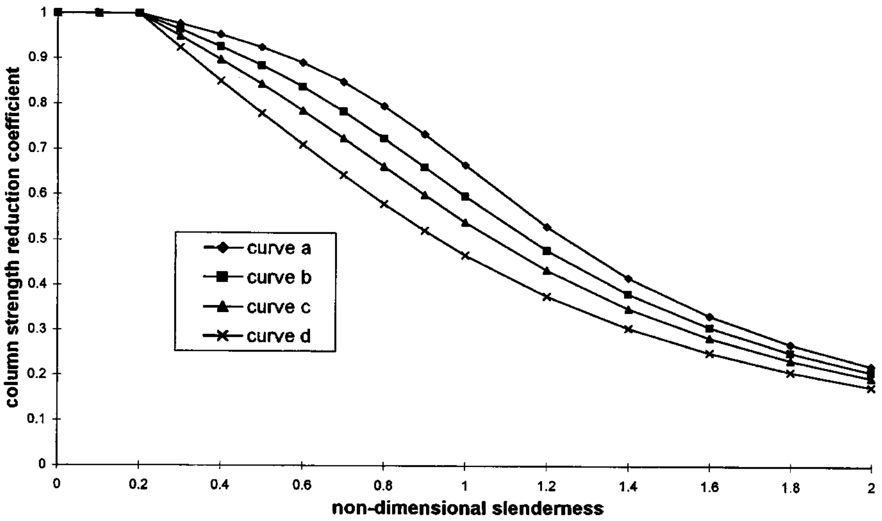

Fig. 1 Column strength reduction coefficient as a function of non-dimensional slendemess

2.1.1. Column buckling curve

The relationship between the strength reduction coefficient of the column (χ_T) and its relative slenderness ($\bar{\lambda}_T$) is dependent on residual stress and initial imperfections in the column. A column buckling curve is usually used to express the relationship between these two parameters. Eurocode 3 Part 1.1 (CEN 1993) gives four column buckling curves for the design of different types of steel columns at ambient temperature. These four column buckling curves are adopted in Eurocode 4 Part 1.1 (CEN 1992) for the design of composite columns and are presented in Fig. 1. For fire resistant design of all composite columns, Eurocode 4 Part 1.2 (CEN 1994) recommends the use of column buckling curve "*c*". This is in contrast to the use of column buckling curve "*a*" for ambient temperature design in Eurocode 4 Part 1.1 (CEN 1992) for concrete filled steel columns. The reason for using column buckling curve "*c*", which gives lower column strength, is attributed to the more severe influence of imperfection and thermal bowing on the column resistance to axial compression when exposed to fire.

2.1.2. Temperature distribution

When using the method in Eurocode 4 Part 1.2 (CEN 1994) for fire resistance design of a concrete filled steel column, the temperature distribution in the composite cross-section is required. Concrete is a very good insulating material and its temperature rise, when exposed to fire, is slow. This creates a highly non-uniform temperature distribution in the composite cross-section. To obtain the exact temperature distribution, numerical techniques (Lie and Chabot 1990, Wickstrom 1983) have to be adopted.

2.1.3. Column squash load and rigidity

The column squash load may be calculated by dividing the composite cross-section into a number of sub-areas and summing up the contribution from each sub-area as:

$$N_{u,T} = \sum \sigma_{s,T} A_s + \sum \sigma_{c,T} A_c + \sum \sigma_{r,T} A_r \tag{4}$$

where σ is the sub-area material design strength at temperature T and A its area. Subscripts "s", "c" and "r" refer to the steel, concrete and reinforcement components respectively. For structural steel and reinforcement, the design strength is equal to the yield stress. For concrete, its design strength is its cylinder strength (CEN 1994). Because the temperature distribution in the composite cross-section is highly non-uniform, it is often necessary to divide the composite section into many sub-areas.

Similarly, the rigidity of the composite cross-section may be expressed as:

$$(EI)_T + \sum E_{s,T} I_s + \sum E_{c,T} I_c + \sum E_{r,T} I_r \tag{5}$$

where E is the sub-area material Young's modulus at temperature T and I its second moment of inertia.

It is clear from Eqs. (4) and (5) that the calculations for column squash load and rigidity are very lengthy and tedious.

2.2. Discussion

The current Eurocode approach (CEN 1994) uses column buckling curve "c" for the fire resistance design of a concrete filled steel column. This needs to be confirmed by experimental results. Also the evaluation of exact temperature distribution in a composite cross-section requires considerable skill and effort. Furthermore the calculations for the exact column squash load and rigidity using Eqs. (4) and (5) can be tedious and time consuming. To validate the Eurocode approach and to make it simpler to use, it is desirable to verify the assumption of adopting column buckling curve "c", by comparing the predicted results from the design method with test data and to have a simplified method for calculating the temperature distribution in the composite cross-section and the column squash load and rigidity.

2.3. Experimental studies

This paper uses data from the experimental studies carried out at the National Research Council of Canada (NRCC) to validate the approach in Eurocode 4 Part 1.2 (CEN 1994). These experimental studies were undertaken to investigate the influence of three types of concrete-filling; namely, plain concrete (PC), bar-reinforced concrete (RC), and fibre-reinforced concrete (FC), on the fire resistance of concrete filled steel columns.

Fifty eight concrete-filled steel columns were tested to failure by exposing the columns to the standard fire. The columns had circular and square cross-sections and were filled with one of three types of concrete. No external fire protection was provided for the steel.

All columns were 3810 mm long. The outside diameter of the circular columns varied from 141 mm to 406 mm while the width of the square columns varied from 152 mm to 305 mm. The wall thicknesses varied from 4.8 mm to 12.7 mm. The test variables were column sectional dimensions, wall thickness, load intensity, end conditions, concrete strength, aggregate and reinforcement type. Fig. 2 shows elevation and cross-sectional details of a typical concrete filled steel columns.

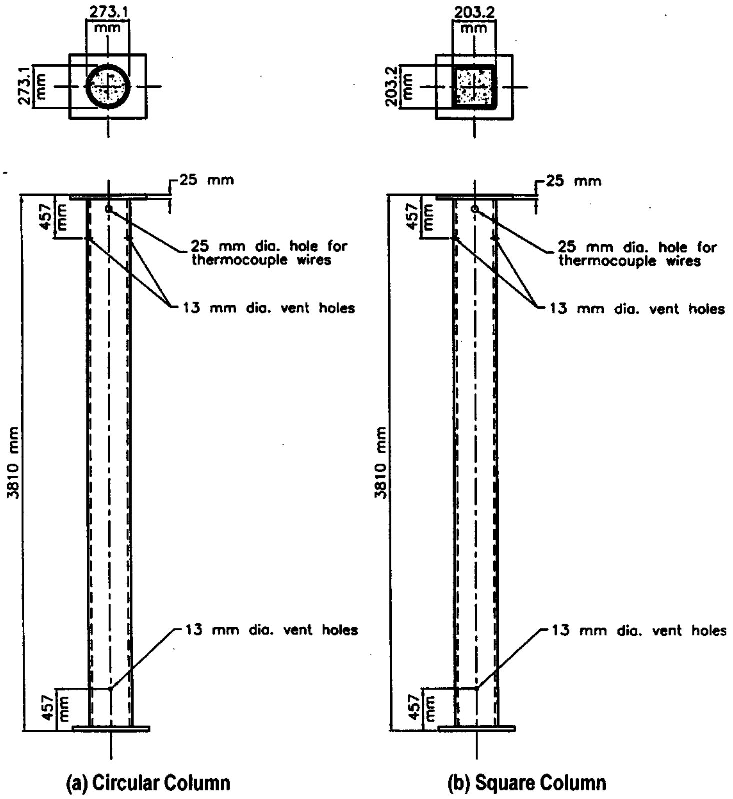

Fig. 2 Elevation and cross section of concrete-filled steel columns

The average 28-day cylinder strength of the concrete varied from 24 to 49 MPa, while the corresponding strength on the test day, which was at least four months after construction, varied from 24 to 59 MPa. For the RC-filling, the reinforcing bars were tied together to form a steel cage, which was placed inside the column. For FC-filling, steel fibres, 1.77 percent by mass, were mixed with the concrete.

The concrete was poured into the column through the top opening and vibrators were used to consolidate the concrete. Thermocouples, with a thickness of 0.91 mm, were installed at the mid-height of the column to measure temperatures at different locations in the cross section.

The tests were carried out by exposing the concrete-filled columns to heat in a furnace especially built for testing loaded columns (Lie 1980). The test furnace was designed to produce conditions such as temperature, structural loads and heat transfer, to which a member might be exposed during a fire. It consists of a steel framework with the furnace chamber inside it. The furnace facility includes a hydraulic loading system with a capacity of 1,000 t.

Most of the columns were subjected to constant concentric loads during testing. The applied load on the columns varied from about 60 to 140% of the factored compressive resistance of the concrete core or about 10 to 45% of the factored compressive resistance of the composite column, calculated according to the specifications of Canadian Standard CSA/CAN3-S16.1-M89 (CSA 1989).

During the test, the column was exposed, under a load, to heating controlled in such a way that the average temperature in the furnace followed, as closely as possible, the ASTM E119-88

Table 1 Comparison of predicted failure load with test data for concrete filled steel columns

Column	Column size	Failure mode	Failure load from test (KN)	Predicted load as per Eurocode: Buckling curve "a" Predicted test (kN)	Buckling curve "a" Test/predicted	Buckling curve "c" Predicted load (kN)	Buckling curve "c" Test/predicted
C02	141.3×6.55	B	110	165	0.67	134	0.82
C04	141.3×6.55	B	131	161	0.81	130	1.01
C05	168.3×4.78	B	150	198	0.76	160	0.94
C08	168.3×4.78	B	218	297	0.73	241	0.90
C09	168.3×6.35	B	150	147	1.02	124	1.21
C11	219.1×4.78	B	492	429	1.15	353	1.39
C13	219.1×4.78	B	384	313	1.23	260	1.48
C17	219.1×8.18	B	525	499	1.05	422	1.24
C20	273.1×5.56	B	574	879	0.65	757	0.76
C21	273.1×5.56	B	525	632	0.83	541	0.97
C22	273.1×5.56	B	1000	958	1.04	845	1.18
C23	273.1×12.7	B	525	409	1.28	379	1.39
C28	355.6×6.35	C	1050	1375	0.76	1274	0.82
C29	355.6×12.7	C	1050	924	1.14	861	1.22
C30	406.4×12.7	C	1900	3107	0.61	3007	0.63
C31	141.3×6.55	B	80	135	0.59	110	0.73
C32	141.3×6.55	B	143	144	0.99	116	1.23
C34	219.1×4.78	B	500	452	1.11	368	1.36
C35	219.1×4.78	B	560	486	1.15	394	1.42
C37	219.1×8.18	B	560	515	1.09	434	1.29
C40	273.1×6.35	C	1050	1539	0.68	1311	0.80
C41	273.1×6.35	C	1050	1689	0.62	1431	0.73
C44	273.1×6.35	B	715	750	0.95	597	1.20
C45	273.1×6.35	C	712	800	0.89	671	1.06
C50	323.9×6.35	C	820	1464	0.56	1218	0.67
C51	323.9×6.35	C	1180	2303	0.51	2024	0.58
C53	355.6×6.35	C	1335	2043	0.65	1780	0.75
C55	355.6×12.7	C	965	1191	0.81	1043	0.93
C57	406.4×6.35	C	1400	2554	0.55	2177	0.64
C59	406.4×12.7	C	1900	3096	0.61	2913	0.65
C60	406.4×12.7	C	1900	3063	0.62	2778	0.68
SQ1	152.4×6.35	B	376	276	1.36	224	1.68
SQ2	152.4×6.35	B	286	308	0.93	249	1.15
SQ7	177.8×6.35	B	549	653	0.84	531	1.03
SQ17	254.0×6.35	C	1096	2028	0.54	1711	0.64
Ave.1		-	-	-	0.96	-	1.16
S.D.1		-	-	-	0.22	-	0.25
Ave.2		-	-	-	0.68	-	0.77
S.D.2		-	-	-	0.17	-	0.18
Ave.3		-	-	-	0.85	-	1.01
S.D.3		-	-	-	0.24	-	0.30

Failure mode: B=Buckling, C=Compression, C=circular, SQ=square
1 Buckling failure, 2 Compression failure, 3 all tests.

(ASTM 1990) standard temperature-time curve. The furnace, concrete and steel temperatures as well as the axial deformations and rotations were recorded until failure of the column.

A summary of the results, as obtained from tests, is presented in Table 1 for plain concrete filled columns. All the columns in Table 1 are of fixed end conditions. Full results of the fire tests on all columns, filled with PC, RC and FC, can be found in Lie and Chabot (1992), Lie *et al.* (1992), Kodur and Lie (1996a), Kodur and Lie (1996b).

2.4. Comparison with test results

Table 1 compares the predicted column resistance to axial compression with the results of the fire tests carried out at NRCC on plain concrete filled steel columns (Lie and Chabot 1992). The total number of reinforced columns from the NRCC test series is small to make a meaningful statistical comparison and these tests are not included in this study. Also, in Table 1, fire resistance time is not compared for the reason that in cases when the predicted fire resistance time was higher than the test fire resistance time, the predicted fire resistance time could not be calculated accurately due to the lack of recorded temperature data after the test failure time. However, for columns whose predicted fire resistance time was lower than the test fire resistance time, the ratio of test to predicted fire resistance time was found to be in close agreement with the ratio of test to predicted load at the test fire resistance time. This suggests that the accuracy in predicting the column strength at the test fire resistance time may be used to represent the accuracy in predicting the fire resistance time.

The predicted column resistance to axial compression were calculated using the design approach in Eurocode 4 Part 1.2 (CEN 1994). The concrete component in each composite section was divided into 20 sub-areas and their temperatures were calculated by interpolation from measured temperatures.

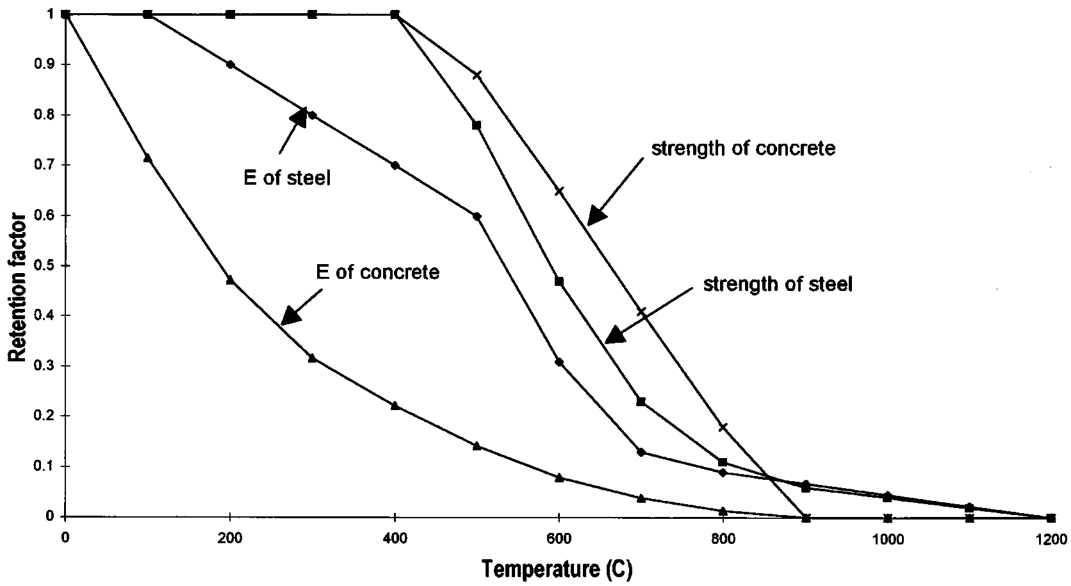

Fig. 3 Strength and elasticity retention factors for steel and concrete

2.4.1. Material properties

The calculation method in Eurocode 4 Part 1.2 (CEN 1994) uses the yield stress and Young's modulus for steel and reinforcement, and the cylinder strength and Young's modulus for concrete. Various high temperature models for these material properties have been proposed in the literature. Each model gives the retention factor, which is the ratio of the value of the specific material property at high temperature to that at ambient temperature. For steel, different models give very similar results and the material property model in Eurocode 4 Part 1.2 (CEN 1994) was used. For concrete, the difference between different models can be quite large. Since this study uses the test results of Lie and Chabot (1992), the concrete model proposed by Lie and Chabot (1990) was adopted. It should be pointed out that the concrete model of Lie and Chabot (1990) gives much higher strength retention factor than Eurocode 4 Part 1.2 (CEN 1994). Fig. 3 presents the retention factors used in this study for steel and concrete at different temperatures.

2.4.2. Results and discussion

Two values of the column resistance to axial compression were calculated. They were obtained using column buckling curves "*a*" and "*c*" respectively. The ratios of the test load to the predicted column resistance to axial compression are given in Table 1.

Results in Table 1 show that overall, the predictions are acceptable if column buckling curve "*c*" is used (predictions being equal to test results on average). The use of column buckling curve "*a*" leads to higher column resistance to axial compression (over prediction being 15% on average), therefore being unsafe. The results of this comparison infers that it is safer to use column buckling curve "*c*" to calculate the resistance to axial compression for concrete filled steel columns under fire conditions.

Analysis of the results in Table 1 reveals that the accuracy of the predictions seems to depend on the failure mode of the column. For slender columns which fail by buckling, using column buckling curve "*a*" over predicted the column resistance to axial compression by about 4% on average, using column buckling curve "*c*" gave safer predictions of column resistance to axial compression, being lower than the test results by 16% on average. For stocky columns which fail by compression, there were gross over predictions in the column resistance to axial compression using either column buckling curve "*a*" or "*c*" (the over prediction being 32% and 23% respectively), suggesting that the concrete strength retention factors at high temperatures used in this paper (proposed by Lie and Chabot 1990) may be too high.

To make predictions on the safe side, concrete models giving lower strength retention factors at elevated temperatures such as the one in Eurocode 4 Part 1.2 (CEN 1994) should be used. This would reduce the predicted strengths for columns failing in compression, therefore improving the accuracy of the predictions. For columns failing in buckling, predictions will only be slightly affected, due to column failure being mainly affected by the Young's modulus and this value is not changed. Thus the good accuracy observed for columns failing in buckling in this paper would be preserved. Consequently, using concrete models giving lower strength retention factors would give safer and more accurate predictions for column resistance to axial compression. Nevertheless, since concrete properties at high temperatures are difficult to measure, the issue of which concrete model is more accurate is not pursued further in this study. The main conclusion from the comparison between predictions and test results is that the accuracy of the method in Eurocode 4 Part 1.2 (CEN 1994) is reasonable and it is safer to use column buckling curve "*c*" in

Table 2 Temperatures in an infinitely large concrete slab exposed to fire on one side

Fire exposure time (minutes)	Distance of centre of sub-area from fire side (mm)					
	Fire	10	30	50	70	>70
30	840	470	250	140	100	70
60	945	642	421	250	150	130
90	1005	738	519	345	245	190
120	1049	850	591	415	310	240

this method.

3. Simplified approach for evaluation of column resistance to axial compression

The two other steps associated with calculating the column resistance to axial compression are the evaluation of temperature distribution in the composite cross-section and the column squash load and rigidity. In the following section a simplified approach is proposed for calculating these parameters.

3.1. Evaluation of temperature distribution

To calculate the exact temperature distribution in an unprotected composite cross-section, complicated numerical analysis should be used. However, for design applications, an approximate temperature distribution can be calculated based on the approach discussed by Lawson and Newman (Lawson and Newman 1996). This approach is applicable to both circular and square columns. For square columns, this method gives the average temperature in the sub-area which has an equal distance to the external surface.

This approximate method is based on the modification of the temperature distribution results of a one-dimensional heat transfer analysis. This temperature distribution is given in Table 2 and is obtained for an infinitely large concrete slab, exposed to fire on one side. To use this table for concrete filled steel sections, two multiplication factors, C_1 and C_2, are employed. The multiplication factor C_1, for each sub-area in the concrete core, accounts for the fact that the temperatures in a concrete filled steel section are greater than those given by the one-dimensional heat flow analysis. This is because the internal concrete sub-areas become progressively smaller, giving an increased heat flow into each sub-area. The multiplication factor C_1 is a function of the

Table 3 Values of multiplication factor C_1 for computing temperature in concrete core of a concrete filled steel column

Outside size (mm) of concrete	Distance of centre of sub-area from outside surface (mm)				
	10	30	50	70	>70
200	1.08	1.22	1.41	1.60	1.80
300	1.05	1.14	1.22	1.36	1.50
400	1.03	1.09	1.18	1.25	1.35
500	1.02	1.07	1.12	1.18	1.25

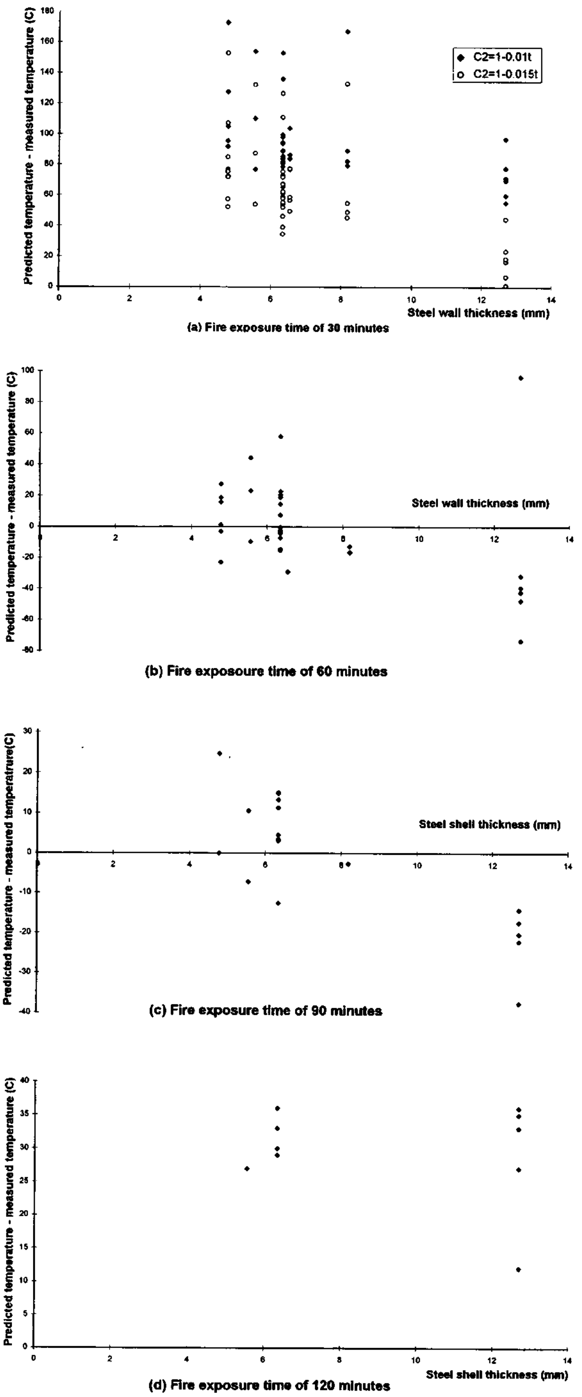

Fig. 4 Variation of temperature difference as a function of steel wall thickness

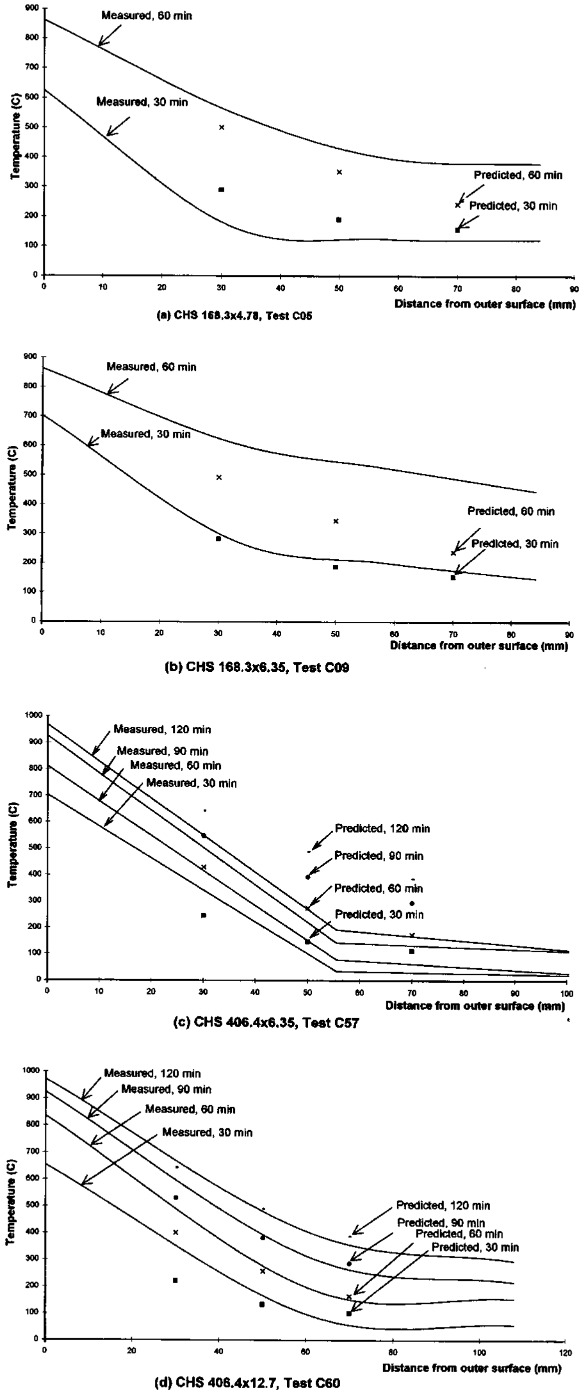

Fig. 5 Variation of temperatures across the section of a concrete filled steel column

cross-section dimension and its values are given in Table 3.

The second multiplication factor C_2 is used to account for the effect of the steel wall. The steel wall acts partly as a heat sink and partly as a thermal shield to the concrete. The effect of the steel wall in reducing the concrete temperature depends on its thickness and the fire exposure time. This multiplication factor is expressed as (Lawson and Newman 1996):

$$\begin{aligned}&\text{For fire resistance} \leq 60 \text{ minutes: } C_2=(1-0.01t)\\&\text{For fire resistance of 90 minutes: } C_2=(1-0.005t)\\&\text{for fire resistance} \geq 120 \text{ minutes: } C_2=1.0\end{aligned} \tag{6}$$

where t (mm) is the thickness of the steel wall.

The average temperature in the steel wall is obtained by modifying the fire temperature by the multiplication factor C_2.

The temperature in the reinforcement is assumed to be the same as the concrete temperature at the same location.

3.1.1. Comparison with test results

The accuracy of constants C_1 and C_2 is evaluated by comparing predicted temperatures with test data reported by Lie and Chabot (Lie and Chabot 1992). To determine the validity of values of C_2, the predicted temperatures, using Eq. (6), are compared with the measured temperatures in Fig. 4(a)-4(d). In these figures the differences between the predicted and measured steel temperatures at the external surface are plotted as a function of steel wall thickness at 30, 60, 90, and 120 minutes of the standard fire exposure, respectively. In these calculations, the measured fire temperatures were used. Bearing in mind the variation in the thermal properties of steel and the complexity in modelling radiation, these figures show that the proposed C_2 factor gives steel temperatures in quite good agreement with test results for the range of steel wall thickness and fire exposure time studied.

Nevertheless, Fig. 4(a) shows that for fire resistance of 30 minutes, the predicted steel temperatures are much higher than the test results. By changing the value of C_2 from $C_2=1-0.01t$ to $C_2=1-0.015t$, better agreement can be obtained between predicted and measured steel temperatures. In addition, the predicted steel temperatures are still on the safe side. The proposed modification to the C_2 factor can be justified based on the pretext that in many practical applications of unprotected concrete filled steel columns, fire resistance of 30 minutes is required. At this fire resistance, the steel wall still retains a high level of strength and rigidity.

In order to further assess the validity of the multiplication factors C_1 and C_2, temperatures predicted using the Table 3 values are compared with measured column temperatures for four concrete filled steel columns. These comparisons are shown in Figs. 5(a)-5(d), where the concrete temperatures are plotted as a function of the distance from the outer surface of the concrete core. In the calculations for fire exposure time of 30 minutes, the new values of $C_2=1.0-0.015t$ were used. Results in Figs. 5(a)-5(d) are given for four representative composite cross-sections: a small and a large section each with two steel wall thicknesses.

Considering the variations in the thermal properties of concrete and the high non-linearity in temperature distributions in the concrete core, results in Figs. 5(a)-5(d) indicate that the proposed temperature calculation method is acceptable for design use. In particular, the agreement between the predicted temperatures and the measured temperatures is quite good for concrete close to the outer surface. Temperatures in the inner concrete core are less well predicted. However, these temperatures are generally low and the concrete strength and stiffness reductions are not very sensitive to large variations in temperature. In addition, the overall contributions to the column strength and stiffness from the inner concrete core is low.

To summarise, the simplified temperature calculation method described by Lawson and Newman (1996), and reproduced in Tables 2 and 3 and Eq. (6), is acceptable for design use. For fire resistance of 30 minutes, the values for the multiplication factor C_2 may be reduced to:

$$\text{for fire resistance of 30 minutes: } C_2=1-0.015t \tag{6a}$$

3.2. Evaluation of column squash load and rigidity

For column squash load, Eq. (4) can be rearranged as:

$$N_{u,T}=N_{u,T,R=0}+N_{u,T,R} \tag{7}$$

where $N_{u,T,R}=0$ is the unreinforced column squash load and $N_{u,T,R}$ the reinforcement contribution.

Since reinforcing bars are usually laid at equal distance to the outer surface and therefore have the same temperature, the reinforcement contribution to the squash load is easily determined in one single calculation. However, for the unreinforced column, the cross-section has to be divided into many sub-areas to give an accurate evaluation of the squash load. This is because the temperature distribution in the concrete core is highly non-uniform. A simplified calculation method is sought in the following section.

3.2.1. Squash load

Since the number of steel hollow sections used in practice is limited, it is not difficult to use Eq. (4) to produce a design aid which gives the exact column squash loads for all the available concrete filled steel sections for one specific combination of steel yield stress and concrete cylinder strength at different standard fire resistance times. Obviously, for each different combination of steel yield stress and concrete cylinder strength, column squash loads will be different.

This paper seeks to establish the relationship between the column squash loads with different combinations of steel yield stress and concrete cylinder strength. Therefore, if the exact squash load for a set of "standard" strengths of steel and concrete is calculated using Eq. (4), the squash load for any other set of design strengths of steel and concrete can be obtained using the proposed relationship. This relationship may be expressed as:

$$N^1_{u,T,R=0}=\alpha_{N,T} \quad N^0_{u,T,R=0} \tag{8}$$

where superscripts "1" and "0" refer to composite columns with design strengths and "standard" strengths of steel and concrete respectively.

At ambient temperature (T=0), the value of $\alpha_{N,T}$ is easily calculated as:

$$\alpha_{N,T=0}=\frac{N^1_{u,T=0,R=0}}{N^0_{u,T=0,R=0}} \tag{9}$$

Define $\beta_{N,T}$ as the ratio of the squash load at temperature T to that at ambient temperature for the "standard" strengths of steel and concrete:

$$\beta_{N,T} = \frac{N^0_{u,T,R=0}}{N^0_{u,T=0,R=0}} \tag{10}$$

At ambient temperature (T=0), $\beta_{N,T}$=1.0.

After a long fire exposure time, the steel wall may be considered to have lost its strength and the unreinforced column squash load approaches that of the concrete core. Therefore, the value of $\alpha_{N,T\to\infty}$ is simply the ratio of the two concrete cylinder strengths, i.e.,

$$\alpha_{N,T\to\infty} = \frac{\sigma^1_c}{\sigma^0_c} \tag{11}$$

Under this circumstance, the ratio of the squash load at T=∞ to that at ambient temperature approaches zero, i.e.,

$$\beta_{N,T\to\infty} = \frac{N^0_{u,T\to\infty,R=0}}{N^0_{u,T=0,R=0}} = 0 \tag{12}$$

Eqs. (9)-(12) define two end points in the relationship between $\alpha_{N,T}$ and $\beta_{N,T}$. Assuming the simplest form of relationship between these two variables, being linear, the following equation may be developed:

$$\alpha_{N,T} = \frac{\sigma^1_c}{\sigma^0_c} + \beta_{N,T} \times \left(\frac{N^1_{u,T=0,R=0}}{N^0_{u,T=0,R=0}} - \frac{\sigma^1_c}{\sigma^0_c} \right) \tag{13}$$

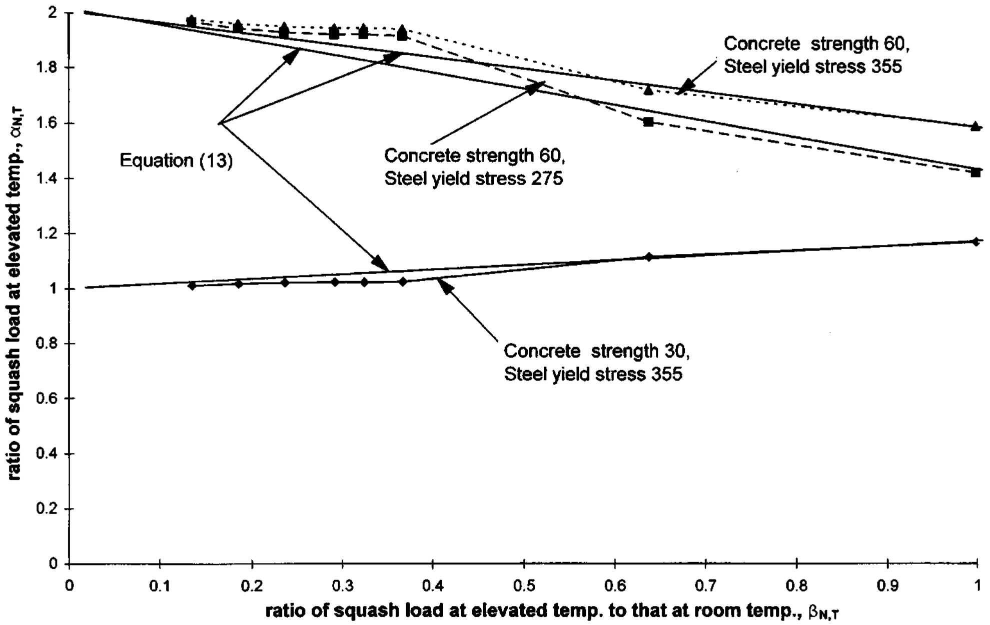

Fig. 6 Determination of column squash load for different grades of steel and concrete, CHS 406.4×10

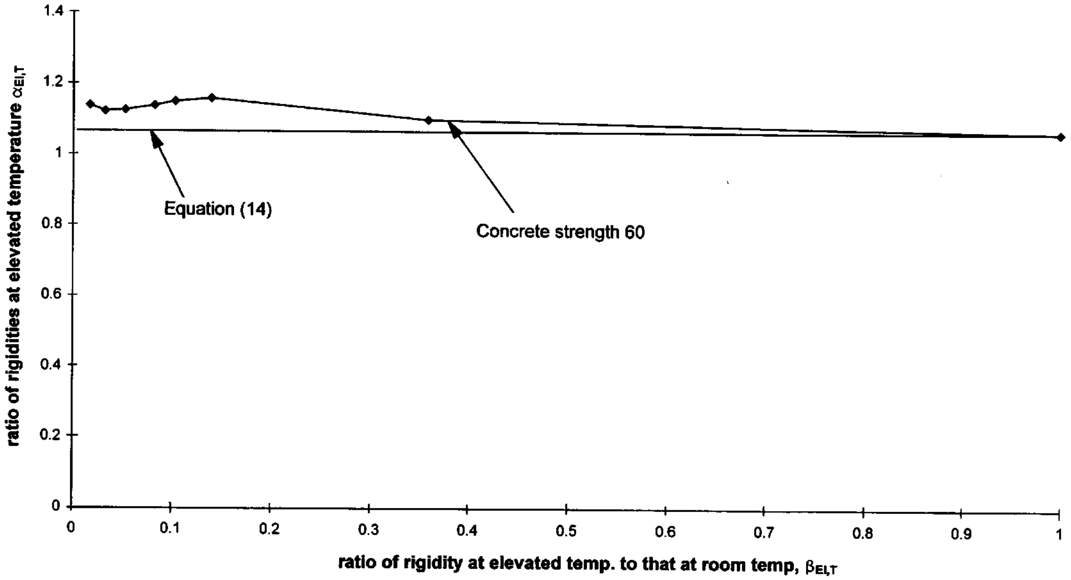

Fig. 7 Determination of column rigidity for different grades of steel and concrete, CHS 406.4×10

The relationship in Eq. (13) has been checked against exact squash loads calculated using Eq. (4) for a number of unreinforced concrete filled steel columns of varying steel and concrete strengths and steel hollow section sizes. Fig. 6 gives a typical example of the results obtained. The "standard" steel yield stress and concrete cylinder strength are 275 MPa and 30 MPa respectively. In this figure, the ratio $\alpha_{N,T}$ of the squash load calculated using different design strengths of steel and concrete to that using the "standard" strengths of steel concrete, is plotted against the ratio $\beta_{N,T}$ of the squash load at elevated temperatures to that at ambient temperature for the composite column with the "standard" strengths of steel and concrete. It can be seen that the values of $\alpha_{N,T}$ predicted using the linear function in Eq. (13) compare well with those obtained from the exact calculation method of Eq. (4).

3.2.2. Rigidity

Similar to calculations for the column squash load, the column rigidity may also be calculated using an equation having the same form as Eq. (13). However, for a large number of composite columns at different standard fire resistance times, Eq. (5) gives similar results for different combinations of steel yield stress and concrete cylinder strength for the following reasons:

1. The reduction in concrete Young's modulus with temperature is rapid.
2. The rigidity is proportional to the fourth power of the distance from the section centre,

Table 4 Design aid for section SHS 254.0×254.0×6.35

	0 min	30 min	60 min	90 min	120 min
Squash load (kN)	3214.6	1985.3	1292.0	1062.0	855.7
Rigidity (kN·m²)	18518	4643.3	1689.9	1104.0	821.0

therefore, the contribution from the steel wall cannot be ignored even at very high temperatures.

3. There is no change in the Young's modulus for various grades of steel. For different strengths of concrete, the variation in its Young's modulus is small.

Therefore, values of $\alpha_{EI,T}$ may be given by:

$$\alpha_{EI,T} = \frac{(EI)^1_{T,R=0}}{(EI)^0_{T,R=0}} = \text{Const} = \frac{(EI)^1_{T=0,R=0}}{(EI)^0_{T=0,R=0}} \tag{14}$$

The predicted values of $\alpha_{EI,T}$, using Eq. (14), have been checked against the exact solutions obtained from Eq. (5). Fig. 7 gives a typical example of the results obtained. The "standard" concrete cylinder strength was 30 MPa.

From the results in Figs. 6 and 7, it can be seen that for an unreinforced concrete filled steel column with any design steel yield stress and concrete cylinder strength, its squash load and rigidity at high temperatures may be easily calculated by multiplying those with the "standard" strengths of steel and concrete by a factor. For the squash load, the value of this multiplication factor is expressed in Eq. (13). For the rigidity, the value of this multiplication factor is a constant and is given in Eq. (14).

Since the number of design steel hollow sections is limited, the values of column squash load and rigidity at different fire resistance times for these composite sections with the "standard" strengths of steel and concrete may be given in a table as a design aid. As an example, Table 4 gives the squash load and rigidity for a square hollow section 254.0×254.0×6.35 mm. For this example, the concrete cylinder strength and Young's modulus were assumed to be 30 MPa and 20000 MPa, the steel yield stress and Young's modulus were taken as 275 MPa and 200000 MPa. Eqs. (4) and (5) were used to obtain the values in Table 4.

4. Design applications

The proposed approach, described in the previous section, can be conveniently used to design concrete filled steel columns under fire conditions. In these calculations, a design aid is used to give the unreinforced column squash load and rigidity with "standard" strengths of steel and concrete. For this example, these values are given in Table 4.

The four steps associated with the calculations are:

- calculating column squash load and rigidity, at ambient temperatures, for the unreinforced column and the reinforcement.
- calculating column squash load and rigidity, at elevated temperatures, for the unreinforced column and the reinforcement.
- calculating the column resistance to axial compression at the required fire resistance time.
- checking the column resistance to axial compression against the applied load.

The applicability of the approach to a design situation is illustrated through a numerical example. In this example, the following values are assumed (Test No. 2, Lie and Irwin 1992):

Section size:	Square Hollow Section 254.0×254.0×6.35 mm.
Reinforcement:	4 bars of 19.5 mm diameter with 23 mm concrete cover.
Material properties:	Steel yield stress 350 MPa, steel Young's modulus 200000 MPa, concrete cylinder 48.1 MPa, concrete Young's modulus 30000 MPa,

	yield stress of reinforcement 400 MPa.
Column:	effective length=1905 mm.

In this example, the column resistance to axial compression at a standard fire resistance of 120 minutes is calculated.

4.1. Step 1. Column squash load and rigidity at ambient temperature:

4.1.1. Unreinforced section

Steel area:	6290 mm^2
Concrete area:	58226 mm^2
Steel second moment of inertia:	6434 cm^4
Concrete second moment of inertia:	28252 cm^4
Squash load for "standard" strength (also Table 4):	$\underline{3214.6\ kN}$
Rigidity for "standard" strength (also Table 4):	$\underline{18518\ kN \cdot m^2}$
Squash load for "design" strength:	$\underline{4582.2\ kN}$
Rigidity for "design" strength:	$\underline{21485\ kN \cdot m^2}$

4.1.2. Reinforcement

The reinforcement contributions to column squash load and rigidity are as follows:

Area:	1194.6 mm^2
Distance to outer surface:	23+19.5/2+6.35=39.1 mm
Distance to section centre:	254/2 − 39.1=87.9 mm
Second moment of inertia:	923 cm^4
Contribution to squash load (Eq. (4)):	$\underline{477.84\ kN}$
Contribution to rigidity (Eq. (5)):	$\underline{1846\ kN \cdot m^2}$

4.2. Step 2. Column squash load and rigidity at 120 minutes

4.2.1. Unreinforced section

Squash load for "standard" strength (Table 4):	855.7 kN
Rigidity for "standard" strength (Table 4):	821.0 $kN \cdot m^2$

Eq. (13) gives: $\alpha_{N,T=120}=48.1/30+855.7/3214.6\times(4582.2/3214.6-48.1/30)=1.556$
Eq. (14) gives: $\alpha_{EI,T=120}=21485/18518=1.16$

Squash load (Eq. (8)):	$1.556\times 855.7=\underline{1331.5\ kN}$
Rigidity (Eq. (14)):	$1.16\times 821.0=\underline{952.4\ kN \cdot m^2}$

4.2.2. Reinforcement

Basic temperature (Table 2):	503°C
Multiplication factor C_1 (Table 3):	1.25
Multiplication factor C_2 (Eq. 6):	1.0
Reinforcement temperature:	$503\times 1.25\times 1.0=629$°C
Strength retention factor (Fig. 3):	0.4 (at 629°C)

Stiffness retention factor (Fig. 3):	0.256 (at 629°C)
Contribution to squash load:	$0.4\times477.84=\underline{191.1\ \text{kN}}$
Contribution to rigidity:	$0.256\times1846=\underline{484.4\ \text{kN}\cdot\text{m}^2}$

4.2.3. Reinforced composite column

Squash load:	$N_{u,T}=1331.5+191.1=\underline{1522.6\ \text{kN}}$
Rigidity:	$(EI)_T=952.4+472.6=\underline{1425\ \text{kN}\cdot\text{m}^2}$

4.3. Step 3. Column resistance to axial compression at 120 minutes

Euler load (Eq. (3)):	$N_{cr,T}=\pi^2\times1425/(1.905^2)=\underline{3875.5\ \text{kN}}$
Relative slenderness (Eq. (2)):	$\bar{\lambda}_T=\sqrt{1522.6/3928.2}=\underline{0.62}$
Multiplication factor (buckling curve "*c*" in Fig. 1):	$\chi_T=\underline{0.774}$
Column resistance to axial compression (Eq. (1)):	$N_T=0.774\times1444.4=\underline{1118\ \text{kN}}$

5. Conclusions

In this paper, a simple design procedure for evaluating the failure loads of concrete filled steel columns is described. This method is based on the model proposed in Eurocode 4 Part 1.2 (CEN 1994). In particular the following three aspects were investigated:

1. The column buckling curve adopted.
2. The determination of the temperature distribution in the composite cross-section.
3. The calculations for the column squash load and rigidity at high temperatures.

Based on the information presented in this paper the following conclusions may be drawn:

1. It is suitable to use column buckling curve "*c*" to calculate the resistance to axial compression for concrete filled steel columns under fire conditions.
2. The proposed simplified procedure for calculating the temperature distribution in the composite cross-section gives reasonable results for design purpose.
3. For a concrete filled steel column under the standard fire exposure with any combination of design strengths of steel and concrete, its squash load and rigidity can be related to those of the column with a set of "standard" strengths of steel and concrete.
4. The example described in the paper illustrates how to use the simplified design method to calculate the column resistance to axial compression under the standard fire exposure, and thus the column fire resistance.

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Notations

A	area
C_1	temperature multiplication factor, to account for section size
C_2	temperature multiplication factor, to account for steel wall thickness
E	Young's modulus

$(EI)_T$	column rigidity at high temperature
$(EI)_{T,R}$	reinforcement contribution to column rigidity
$(EI)_{T,R=0}$	unreinforced column rigidity at high temperature
I	second moment of inertia
L	column buckling length
$N_{cr,T}$	column Euler load at high temperature
N_T	column resistance to axial compression at high temperature
$N_{u,T}$	column squash load at high temperature
$N_{u,T,R}$	reinforcement contribution to column squash load at high temperature
$N_{u,T,R=0}$	unreinforced column squash load at high temperature
t	steel wall thickness
$\alpha_{EI,T}$	ratio of unreinforced column rigidity of design strengths (of steel and concrete) to that of "standard" strengths (of steel and concrete)
$\alpha_{N,T}$	ratio of unreinforced column squash load of design strengths (of steel and concrete) to that of "standard" strengths (of steel and concrete)
$\beta_{EI,T}$	ratio of unreinforced column rigidity of "standard" strengths (of steel and concrete) at high temperature to that at normal temperature
$\beta_{N,T}$	ratio of unreinforced column squash load of "standard" strengths (of steel and concrete) at high temperature to that at normal temperature
$\overline{\alpha}_T$	column relative slenderness at high temperature
σ	design strength of a material
χ_T	column strength reduction coefficient at high temperature

Subscripts

s	steel
c	concrete
r	reinforcement
T	high temperature

Superscripts

0	for "standard" strength (of steel and concrete)
1	for design strength (of steel and concrete)