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Strengthening of Fire-damaged Columns by Cross-sectional Enlargement: A Computational Evaluation

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ABSTRACT

An analytical model is presented to calculate the load carrying capacity of concrete columns strengthened by enlarging the cross-sections after high temperature exposure. The calculated column temperatures and load carrying capacities are compared with those measured. The results indicate the capability of the analytical model to predict the temperatures, and the load carrying capacities of the strengthened column with accuracy adequate for structural practice. The model enables the determination of the load carrying capacity of a strengthened column after high temperature exposure for different values of the significant parameters, such as load, eccentricity, original column dimension, the depth of the strengthening concrete, concrete strength, the time-temperature curve in the furnace, and the percentage of reinforcing steel, without the need for testing.

Keywords: Reinforced concrete columns; strengthening; load carrying capacity; high temperature; finite difference.

1. INTRODUCTION

Concrete material properties deteriorate (decrease of strength and stiffness) during and after fire exposure(Abrams, 1996; Malhotra, 1956; Khour, 1992; RILEM-committee, 1995; Takeuchi et al., 1993). Enlargement of the cross-section is commonly used to strengthen reinforced concrete columns after fire damage, which increases the strength and stiffness significantly.The method for calculating the load carrying capacity and deflection of strengthened columns, without high temperature exposure, is well known. Many analytical simulations have been developed to calculate the performance of concrete members during fire (Anderberg, 1993; Mohamad J. Terro, 1998; Lie, T. T. 1984; Lie, T. T., 1993). These numerical analysis cannot be generalized for strengthened columns after fire damage. It is complicated to evaluate the mechanical properties for the strengthened column due to several factors. First, the temperature distribution during the fire exposure is not uniform throughout the section, causing non-uniform reduction in the strengths of the concrete and the reinforcing steel. Secondly, the strengths of the original concrete and the strengthening concrete are generally different, resulting in a sudden change at the interface. In practice, the load carrying capacity is the most important mechanical property of RC columns. In this paper, an analytical method is proposed to calculate the load-carrying capacities of strengthened RC columns, taking many significant parameters into account, such as temperature curve, fire duration, the depth of strengthening concrete, original concrete section size, concrete strength and percentage of reinforcing steel, without the necessity of testing.

The author (Liu, 2002) tested i) nine strengthened reinforced concrete columns, for compression and bending after high temperature exposure, and ii) two strengthened control specimens without fire damage. These tests provided the basic data that enable the development and validation of analytical models enabling determination of the load carrying capacity for specified values of the important input variables.

2. TEST SPEIMENS

The eleven specimens tested, were described in detail by Liu, 2002, and illustrated in Fig. 1. All columns were divided into two groups (A and B, Table 1) in terms of the grade of the strengthening concrete. The original columns were 600mm long with a rectangular cross section of 100mm×200mm. Four longitudinal reinforcing bars of 12mm dimeter and web reinforcement of 4mm diameter were used. The strengthened columns were 900mm long, and had an enlarged cross section of 100mm×300mm, and two additional longitudinal reinforcing bars of the same diameter, i.e., 12mm, and extended web reinforcement of the same size, i.e., 4mm. The locations of all the main reinforcing bars, and all the web reinforcement, are shown in Fig.1. The concrete cover to main reinforcing bars was 20mm.

All main reinforcing bars had the same yield strength 322 MPa. The mix proportions and strength of original and strengthening concrete are shown in Table 1.

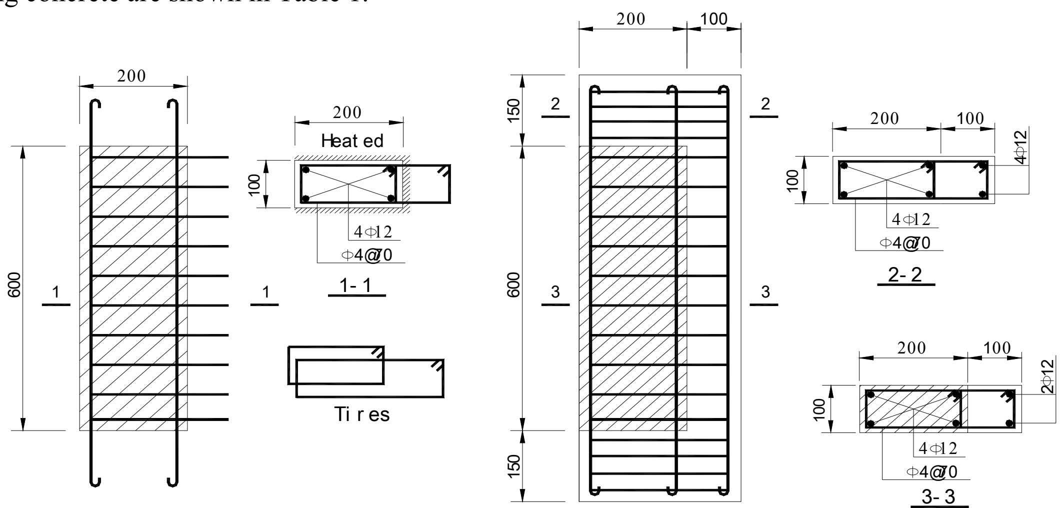


Fig.1 Dimensions and reinforcing bars of specimens

Table 1 Concrete mix –design and strength

Concrete		**Mix proportion(weight)**				**Cube Compression strength MPa**	**Prism compression strength MPa**
		Cement	Water	Sand	Coarse aggregate*		
Original		1	0.617	2.257	4.127	32	25
Strengthening	A	1	0.521	2.181	3.997	40	35
	B	1	0.483	2.256	4.021	49	41

(* Carbonate aggregate)

3. TEST CONDITIONS AND PROCEDURES

The procedure of the test:

(1) Three sides of the original columns, Fig. 2, were heated up to 800◦C, and the temperature kept constant for 10 minutes;
(2) After natural cooling, the columns were taken out from the high temperature furnace, and kept at ambient temperature for two weeks;
(3) The columns were strengthened on one side (the interior side of an exterior column) exposed to fire (Fig.1);
(4) After 30 days of curing, they were loaded to failure.

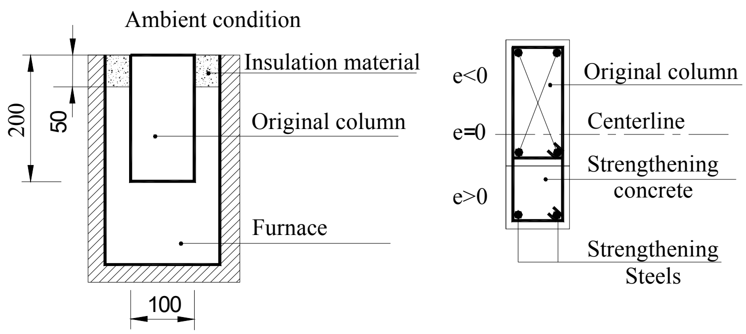
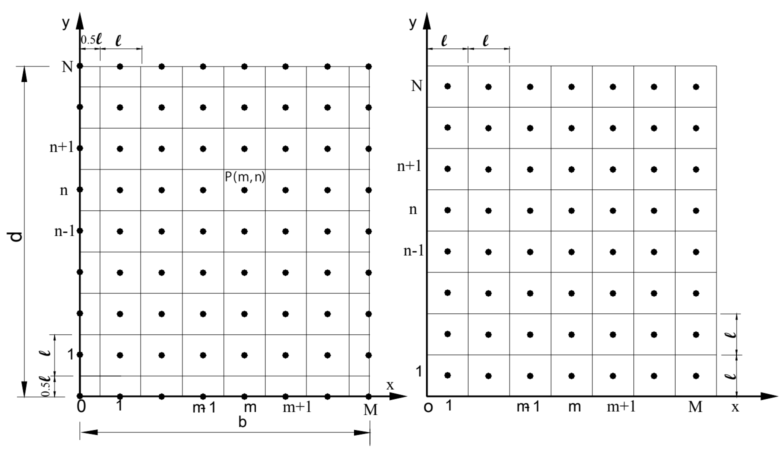


Fig. 2 Sketch of the high temperature furnace and the sign specification for eccentricity

4. TEMPERATURES OF THE COLUMNS DURING FIRE EXPOSURE

The calculation of the load carrying capacity of strengthened columns, after high temperature exposure, as carried out in various steps. It involved the calculation of the temperatures inside the column, and the load carrying capacity of strengthened columns after fire exposure.

The column temperatures were calculated by the finite difference method (Yunus, 2003). A similar method has been applied to the calculation of temperatures of various building components exposed to fire(Lie, T. T. 1993, Reddy et al, 2008). Because of the method of deriving the heat transfer equations and the similarities to those described in detail by Reddy et al.(2008), it will not be discussed in detail here, only the main equations for the calculation of the column temperatures and the calculated results will be presented.



(a) Thermal Network (b) Stress-strain Network

Fig 3 Thermal and stress-strain networks in cross Section

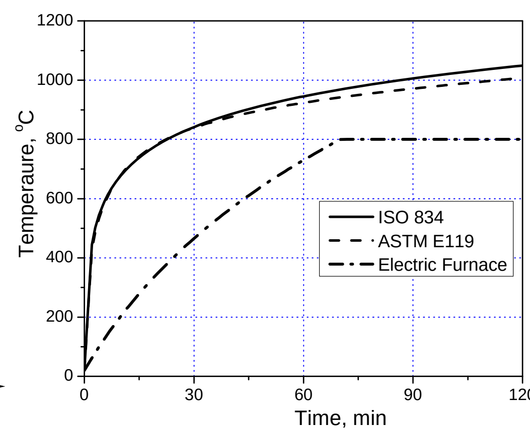


Fig. 4 Temperature rise curves

4.1 DIVISION OF CROSS- SECTION INTO ELEMENTS

The cross-sectional area of the original column was subdivided into a number of elements, arranged in a rectangular network (Fig. 3 (a)). The elements are square for elements inside the column, and rectangular at the surface. As illustrated in Fig.3 (a), in an x-y coordinate system, the point p (m, n) has the coordinates $x = (m + 0.5)l$ and $y = (n + 0.5)l$.

4.2 EQUATIONS FOR THE FIRE-CONCRETE BOUNDARY

Three sides of the columns were exposed to heating controlled in such a way that the average temperature in the furnace closely followed the following expression:

$$T_f^i = \begin{cases} 668\ln(0.0316t + 1) + 20 & t \le 70\,\text{min} \\ 800 & t > 70\,\text{min} \end{cases} \tag{1}$$

where t is the time in minutes , and T_f^i is the fire temperature in °C at time t=iΔt. The furnace curve and the standard curves of ISO 834 and ASTM E 119 stardand are shown in Fig. 4.

The temperature increases in the column were derived by using the energy balance for each element. In the following section, all calculations are carried out for the surface of the column.

For the surface subjected to fire directly, taking node (m, 0) as an example, the temperature at time t= (i+1) Δt is given by the following expression:

$$T_{(m,0)}^{i+1} = \tau(T_{(m-1,0)}^i + 2T_{(m,1)}^i + T_{(m+1,0)}^i) + [1-\tau(4+\frac{2hl}{k_{(m,0)}^i})]T_{(m,0)}^i + \frac{4hl}{k_{(m,0)}^i}\tau T_f^i + \frac{2\sigma\varepsilon_c l}{k_{(m,0)}^i}\tau[(T_f^i)^4 - (T_{(m,0)}^i)^4] \quad (2)$$

where τ is the dimensionless Fourier number given by

$$\tau = \frac{k_{(m,n)}^i}{\rho C_{p(m,n)}^i l^2}\Delta t \quad (3)$$

in which $T_{(m,n)}^i$ is the temperature at node (m, n) for the time step i, σ is the Stephan-Boltzman constant (5.67×10^{-8} W/m^2•K^4), ε is the emissivity of the column surface, k is the conductivity coefficient, ρ is the mass density, C_p is the heat specific concrete, and h is the heat transfer coefficient

For nodes on the surface not subjected to fire, taking node (m, N) as an example, the temperature at time t= (i+1)Δt is given by the following expression:

$$T_{(m,N)}^{i+1} = \tau(T_{(m-1,N)}^i + 2T_{(m,N-1)}^i + T_{(m+1,N)}^i) + [1-\tau(4+\frac{2hl}{k_{(m,N)}^i})]T_{(m.N)}^i + \frac{2hl}{k_{(m,N)}^i}\tau \times T_a \quad (4)$$

where T_a is the ambient temperature.

For elements at the corner exposed to fire directly, the temperature at the time t= (i+1) Δt, taking node (0, 0) as an example, is given by

$$T_{(0,0)}^{i+1} = 2\tau(T_{(1,0)}^i + T_{(0,1)}^i) + [1-\tau(4+\frac{4hl}{k_{(0,0)}^i})]T_{(0,0)}^i + \frac{4hl}{k_{(0,0)}^i}\tau T_f^i + \frac{4\sigma\varepsilon_c l}{k_{(0,0)}^i}\tau[(T_f^i+275)^4 - (T_{(0,0)}^i+275)^4] \quad (5)$$

For elements at the corner not exposed to fire, the temperature at the time t= (i+1) Δt, taking node (0, N) as an example, is given by

$$T_{(0,N)}^{i+1} = 2\tau(T_{(1,N)}^i + T_{(0,N-1)}^i) + [1-\tau(4+\frac{2hl}{k_{(0,N)}^i})]T_{(0,N)}^i + \frac{2hl}{k_{(0,N)}^i}\tau \times T_a \quad (6)$$

4.3 EQUATIONS FOR THE INTERIOR OF THE COLUMN CROSS-SECTION

The temperature at time t= (i+1) Δt is given by

$$T_{(m,n)}^{i+1} = \tau(T_{(m-1,n)}^i + T_{(n+1,m)}^i + T_{(m+1,n)}^i + T_{(n\_1,m)}^i) + (1-4\tau)T_{(m,n)}^i \quad (7)$$

4.4 STABILITY CRITERIA

To avoid divergent oscillations in nodal temperatures, the value of Δt must be maintained below a certain upper limit. Based on the second law of thermodynamics, the stability criterion is satisfied, when the coefficients of all $T_{(m,n)}^i$ in the $T_{(m,n)}^{i+1}$ expressions are greater than or equal to zero for all nodes (m, n). Different equations for different nodes may result in different restrictions on the size of the time step Δt , and the criterion that is the most restrictive should be used in the solution of the problem. For this case, the smallest, and thus the most restrictive primary coefficient is the coefficient of $T_{(0,0)}^i$ or $T_{(0,N)}^i$ (the highest temperature in the cross section; hence the stability criterion can be expressed as follows:

$$1-\tau(4+\frac{4hl}{k^i_{(0,0)}})\geq 0 \Rightarrow \quad \Delta t \leq \frac{\rho C^i_{p(0,0)} l^2}{4(k^i_{(0,0)}+hl)} \leq \frac{\rho C_{p,\min} l^2}{4(k_{\max}+h_{\max} l)} \tag{8}$$

where $C_{p,\min}$ is the minimum value of the specific heat of the concrete, $k_{\max}$ is the maximum value of its thermal conductivity, $h_{\max}$ is the maximum value of the coefficient of the heat transfer to be expected during the exposure to fire.

With the aid of Eqs. (1) through (8), and the relevant material properties given by (Eurocode 4, 2005), the temperature distribution in the column and on its surface can be calculated at time t= (i+1) Δt, if the initial temperature is known.

5. Load Capacity of the Enhanced Cross-section after Fire Exposure

5.1. Assumptions in the Calculation of the Load capacity after Fire exposure

After exposure to fire, the residual strength of the original column decreases with the duration of the fire exposure. After strengthening, the load carrying capacity and stiffness of the strengthened column increase, and can be calculated by load-deflection analysis, with the columns idealized with effective length l_0 . Due to imperfections of the column and the loading device, a small eccentricity $e_a = 2-5mm$ is considered.

Most reinforced concrete columns are submitted to compression and bending. Fig.5 shows the curvature effect due to such combined loading. The increased moments cause an additional eccentricity,Δ, which can be calculated from the load eccentricity, e_0, plus the additional eccentricity, $e = e_0 + e_a$, by multiplying with an eccentricity magnification factor η (Liu, 2002).

Thus for any given deflection at mid-height, the load strain and curvature are varied, until the internal moment and axial force at the midsection are equilibrium with the applied load.

In this way, the load deflection curve can be calculated for the strengthened column. From these curves, the load carrying capacity of the strengthened columns can be determined for different fire durations. The calculation of the load carrying capacity of the strengthened columns was based on the following assumptions:

1. Plane sections remain plane.
2. Concrete has no tensile strength.
3. There is no slip between original concrete and strengthening concrete.
4. The change in column length before applied load, consisting of shrinkage of the concrete, creep, and thermal expansion, is negligible.
5. The temperatures, stresses and deformations at the center of an element are representative of those of the whole element. Thus for a concrete element (m, n), the representative temperature is

$$(T^i_{(m,n)})_{strain-straiss} = (\frac{T^i_{(m-1,n)} + T^i_{(m+1,n)} + T^i_{(m,n-1)} + T^i_{(m,n+1)}}{4})_{thermol} \qquad 1 \leq m \leq M, 1 \leq n \leq N \tag{9}$$

where the subscripts “strain-stress” and “thermal” refer to the network shown in Fig.3 (b) and (a), respectively.

6. The residual strength and elastic modular of reinforcing steel after high temperature exposure are the same as those without fire exposure (Lie, 1996; Qian and Cheng, 1990).
7. The compression strain-stress relation of concrete is given by Nan and Guo, 1997.

$$\sigma_c = \begin{cases} f_c^T \left[2\frac{\varepsilon_c}{\varepsilon_p^T} - (\frac{\varepsilon_c}{\varepsilon_p^T})^2 \right] & \varepsilon_c \le \varepsilon_p^T \\ f_c^T \dfrac{\frac{\varepsilon_c}{\varepsilon_p^T}}{5(\frac{\varepsilon_c}{\varepsilon_p^T} - 1)^2 + \frac{\varepsilon_c}{\varepsilon_p^T}} & \varepsilon_c > \varepsilon_p^T \end{cases} \tag{10}$$

where, σ_c =the stress in the concrete, ε_c= the strain in the concrete, and f_c^T= the residual strength of the concrete after fire exposure, are given by

$$f_c^T = (1.008 - \frac{T}{1+[0.17(T-2)/100)]^6}) f_c \ , \tag{11}$$

ε_p^T = the peak strain of concrete at after high temperature, given by

$$\varepsilon_P^T = 1 + 0.09\frac{T-2}{100} + 0.046(\frac{T-2}{100})^2 \ , \tag{12}$$

and

ε_p =the peak strain of concrete without fire damage, for normal strength concrete, ε_p =0.002.

5.2 DIVISION OF THE STRENGTHENING CONCRETE INTO STRIP ELEMENTS

The division of the original cross section is shown Fig. 3(b). The strengthening concrete area is subdivided into a number of strips the along depth (Fig. 6).

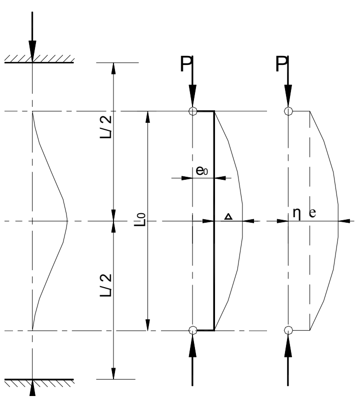


Fig.5 Eccentricity magnification factor

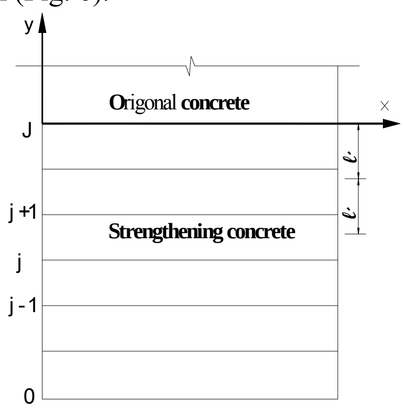


Fig.6 Arrangement of elements in the strengthened concrete area

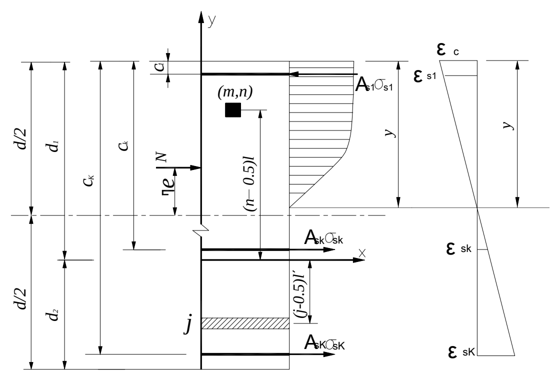


Fig. 7 Strain and stress distribution in the total strengthened cross-section

The compression zone changes from the original concrete zone to the strengthening concrete zone depending on the load eccentricity (with respect to the centerline of the total cross section, Fig. 2). In the following sections, all calculations are carried out for the load eccentricity e_0 <0 (Fig. 2).

5.3 CALCULATION OF THE LOAD CARRYING CAPACITY

The strain of the original concrete at the centriod of a square element is given by Fig. 7 as follows:

$$\varepsilon_{c,(m,n)} = \varepsilon_c (1 - \frac{d_1 - (n-0.5)l}{y}) \qquad 1 \le n \le N \tag{13}$$

The strain of the strengthening concrete at the centerline of a strip element is given by

$$\varepsilon_{c,j} = \varepsilon_c (1 - \frac{d_1 + (j-0.5)l'}{y}) \qquad 1 \le j \le J \tag{14}$$

The strain in the reinforcing steel is given by

$$\varepsilon_{s,k} = \varepsilon_c (1 - \frac{c_k}{y}) \qquad 1 \le k \le K \tag{15}$$

where y, the depth of compression zone; ε_c, strain at the outermost compression fiber; d_1, the depth of the original cross- section; d_2, the depth of the strengthening concrete, d, the total depth of the strengthened column cross-section; and c_k, the distance from the centroid of kth row steel to the top surface of original concrete.

Using the above procedure, the overall axial strain in the column is varied until the internal moment at mid-height, due to the contribution of each of the concrete elements and the reinforcing bars about the centerline of the column cross-section at mid-height, is equal to the external moment due to the applied load at mid-height. The internal moment at mid-height is calculated using

$$M_{\text{int}} = \sum_{m=1}^{M}\sum_{n=1}^{N}\sigma_{m,n}A_{m,n}d_{m,n} + \sum_{j=1}^{J}\sigma_j A_j d_j + \sum_{k=1}^{K}\sigma_{s,k}A_{s,k}d_{s,k} \tag{16}$$

where, $\sigma_{m,n}$ = the stress in any square element of the cross section corresponding to the fire exposure duration, $A_{m,n}$ = the area of the square element, $d_{m,n}$ = the distance from the centroid of the square element to the centerline of the strengthened column, σ_j = the stress of any strip concrete element, A_j = the area of the strip element, d_j =the distance from the centroid of the band element to centerline of the strengthened column, $\sigma_{s,k}$ = the stress of the kth row longitudinal reinforcing bar, $A_{s,k}$ = the area of the kth row longitudinal reinforcing bar ,and $d_{s,k}$ = the distance from the centroid of the kth row longitudinal reinforcing bar to the centerline of the strengthened column.

The external moment is calculated as the product of the total applied force at mid-height, P, times the horizontal deflection, ηe, of the column at that location as follows:

$$M_{ext} = P\eta e \tag{17}$$

where η is the eccentricity magnification factor, and **P** is calculated as a summation of the force contribution from the individual elements and longitudinal bars, that is

$$P = \sum_{m=1}^{M}\sum_{n=1}^{N}\sigma_{m,n}A_{m,n} + \sum_{k=1}^{K}\sigma_k A_k + \sum_{j=1}^{N_{ss}}\sigma_{s,j}A_{s,j} \tag{18}$$

With the aid of Eqs. (9) to (18), the load that the strengthened column carries, and the total internal moment at the midsection, can be calculated for any value of eccentricity.

6. RESULTS AND DISCUSSION

Using the analytical model described in the above section, the temperatures in the columns and the load carrying capacities change with the strain at the outermost compression fiber were calculated.

In Fig. 8, the calculated temperatures are compared with those measured at various locations in those columns. It can be seen that there is good agreement between the calculated and the measured concrete temperatures. The temperatures measured at the low temperature zone (points 2, 3) of the cross-section initially show a relatively rapid rise in temperature, followed by a period of nearly constant temperature in the early stages of the test. This temperature behavior may be the result of the column thermal-induced migration of moisture towards the low temperature area of the column (Lie, 1993), where, as shown in the Fig.7, the migration is most obvious.

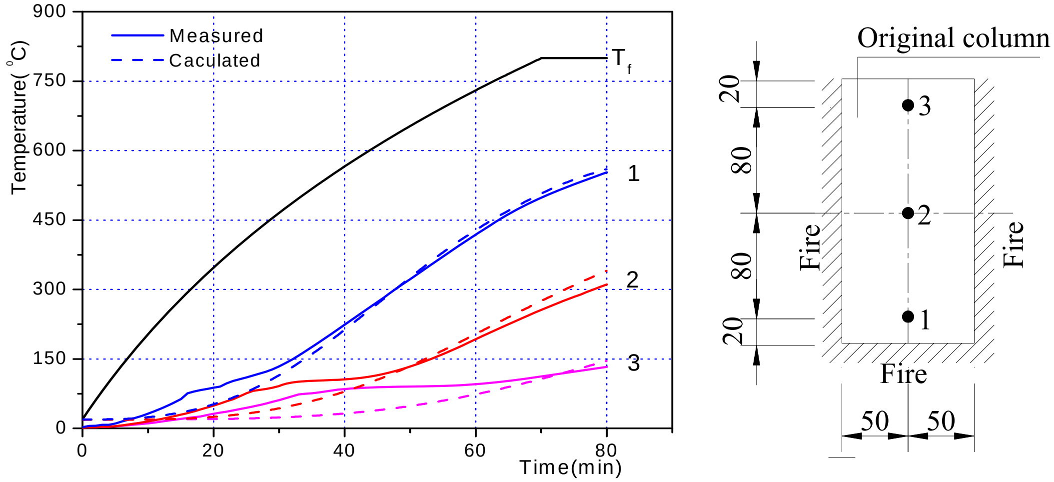


Fig. 8 Temperature of the concrete at different locations in the column, as a function of exposure

Table 2 Comparison Between the Calculated and Measured Strengths

Strength of strengthening concrete (Mpa)	No.	Initial Eccentricity (mm)	Measured Strength (kN)	Calculated Strength (kN)	$\frac{Calculated}{Measured}$
35	A0	0	991.5	889.3	0.90
	AT7	70	624.3	575.0	0.92
	AT4	40	765.9	734.3	0.96
	AT2	20	862.2	819.5	0.95
	AT0	0	744.6	706.3	0.95
	AT-4	-40	622.9	603.9	0.97
	AT-7	-70	487.9	441.6	0.91
41	B0	0	1028.7	979.8	0.95
	BT4	40	830.8	780.5	0.94
	BT0	0	827.9	791.3	0.96
	BT-4	-40	631.0	602.8	0.94

In Figs. 9, the calculated and measured loads are shown as functions of the strain of the outermost compression fiber. The load increases with the strain, until it reaches the load carrying capacity of the column. The calculated load carrying capacities are given in Table 2, together with the measured values. From Figs. 9 and Table 2, it can be seen that there is a good agreement between the calculated and measured load carrying capacities of the strengthened columns. Table 2 shows that the ratio of the calculated to the measured compressive strength is smaller than 1, ranging from 0.90 to 0.96, with an average 0.94, a coefficient of variation of 0.002, and a standard deviation of 0.022. It seems that analytical model is able to provide accurate and safer results.

The relative initial eccentricities, e_0/d_0 (d_0 is the effective depth of the column) of above specimens are small, and range from 0 to 0.233, with the failure governed by concrete compression. For large eccentricities, no calculation was done due to a lack of tested values. The load carrying capacity of columns with large eccentricities mainly depends on the strength of the tension steel, and the effect of the reduction of concrete compression strength is insignificant. Therefore, the above method can be used to calculate the strength of columns with large eccentricities.

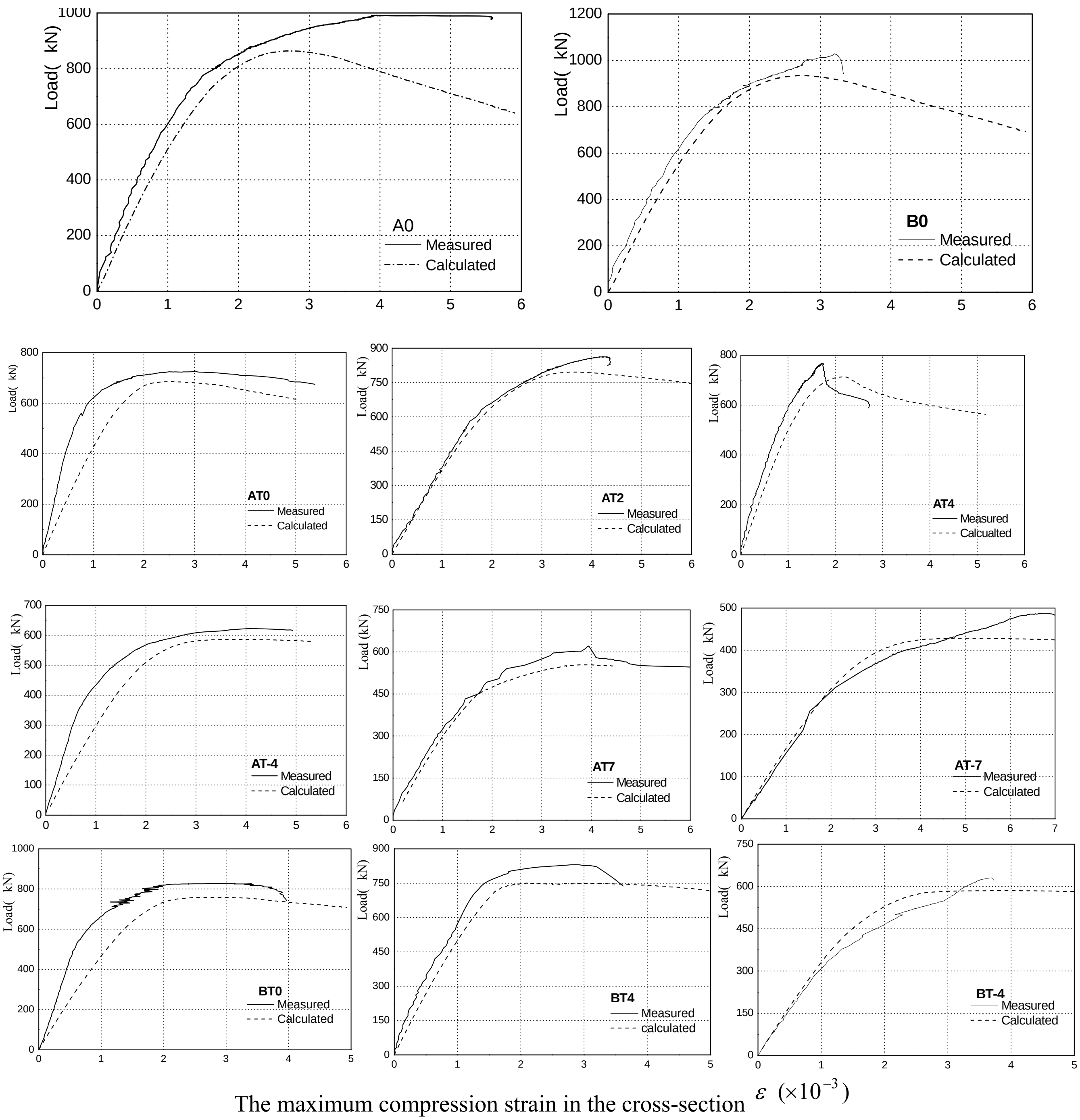


Fig. 9 Calculated and measured load of strengthened columns as a function of the maximum compression strain in the total strengthened cross-section

7. CONCLUSIONS

Based on the results of this study, the following conclusions can be drawn:

1. The analytical model employed in this study is capable of predicting the load carrying capacity of strengthened concrete columns after fire exposure, with an adequate accuracy for practical purposes.

2. The model will provide additional existing data on the load carrying capacities of strengthened reinforced concrete after fire exposure, which at present consists predominantly of data restricted to i) regular columns exposed to fire, and ii) strengthened columns not exposed to fire.
3. Using the model, the load carrying capacity of the strengthened reinforced concrete column can be evaluated for varying parameters, such as the time-temperature curve, fire duration, the original cross-section size, the increased cross-section depth of the strengthened column, concrete strength and percentage of reinforcing steel, without the necessity of testing.

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