

Interaction diagrams for reinforced concrete sections subjected to fire

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ABSTRACT

Strength interaction diagrams are common tools for reinforced concrete (RC) cross-section analysis and design. They usually relate the ultimate values of the bending moments and axial force, allowing quick assessment of a cross-section's strength. Under fire action, however, substantial changes occur on the behavior of the cross-section, and these diagrams become strongly dependent on the temperature field, which introduces thermal strains and modifies the stress–strain relationships. The purpose of this paper is to present an algorithm for the construction of strength interaction diagrams for arbitrary-shaped RC sections subjected to fire action. The diagrams are obtained by a stepwise variation of the deformed configuration, under assumptions of conventional ultimate strain values for concrete and steel. The procedure is illustrated by the construction of interaction diagrams for some RC sections subjected to different temperature distributions.

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1. Introduction

Concrete is a material with good behavior under fire conditions. It is noncombustible and has low thermal conductivity, protecting the reinforcement. The key role played by reinforced concrete (RC) columns on the stability of structures under fire conditions indicates that their fire resistance must be properly assessed for design.

The failure mechanism of RC columns is more difficult to predict than that of beams and slabs. Not only may crushing of the concrete or yielding of the tensile reinforcement occur, but also column overall buckling. Sometimes spalling is observed, dramatically reducing the overall resistance, especially with high-strength concrete columns [1]. Purkiss [2] and Fleischmann and Buchanan [3], among other authors, have presented methods for evaluation of the fire resistance of concrete structural members.

Three main design approaches are possible, according to Franssen and Dotreppe [4]: experimental tests, numerical modeling and simplified methods. Experimental (furnace) tests provide useful information, but are expensive, time consuming, restricted to small specimen sizes and may not take into account member continuity and restraint. Numerical modeling is the most powerful tool for predicting the behavior of structures under fire. Developments in numerical methods and computer hardware, associated to enhanced knowledge of the properties of concrete and steel at elevated temperatures, enabled the development of general-purpose as well as specialized programs capable of structural analysis under fire, such as VULCAN [5] and SAFIR [6]. However, these

sophisticated programs are not always available to the practising engineer, and they have therefore been employed mostly for research. Simplified methods, on the other hand, are readily available from design codes. European [7] and American [8] codes prescribe experimental, empirical-based correlations and minimum dimensions (tabulated data), as well as simple and advanced calculation methods for determining the fire resistance of concrete structures.

In general, analysis methods are carried out in two steps: (i) heat transfer analysis and (ii) structural analysis. Heat transfer analyses are used to evaluate the temperature distribution throughout the structure. Under fire conditions, the material resistance decreases, deformability increases, and thermal strains develop. Simplified calculation methods, applied to columns, generally perform the analyses at the cross-section level, taking into account buckling and second-order effects in a simplified way.

Several works have dealt with the fire resistance of RC columns. A brief review is listed in the following. Lie and Celikkol [9] presented a mathematical model to calculate the fire resistance of circular RC columns, and Lie and Irwin [10] applied this model for rectangular RC columns. The procedure is similar to the method prescribed by Eurocode 2 [7] for RC sections at ambient temperature.

Franssen and Dotreppe [4] describe experimental tests and the values obtained for the fire resistance of concrete columns of circular section. Theoretical methods are presented for quick design. One of them is based on previous work by Dotreppe et al. [11].

Tan and Yao [12] developed a simple design method to predict the fire resistance of four-face heated RC columns based on the American Concrete Institute Code [13] ambient temperature

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prescriptions. The method was later extended to one-, two- and three-face heated columns [14] with the prediction of the shift of the neutral axis due to non-symmetric heating.

Kang et al. [15] proposed a numerical method for the ultimate behavior of RC columns subjected to fire. It assumed that the capacity is determined by the axial force–bending moment interaction curves (N – M interaction curves).

Some simple calculation methods use a reduced cross-section to represent the deterioration of material strength and stiffness. Examples are the methods presented by Eurocode 2 [7]: the “500 °C isotherm method” and the “zone method”, based on the methods developed by Anderberg [16] and Hertz [17], respectively.

Whatever method is employed for column design, insight into the behavior of the cross-section under high temperature is highly desirable. Bending moment–axial force curves and surfaces, for uniaxial and biaxial bending, respectively, are useful tools for the design of RC sections and columns at ambient temperature. However, there seems to be no available general formulation to obtain these curves and surfaces for RC sections under temperature effects due to fire action. One of the few works to address this issue is the paper by Meda et al. [18].

The purpose of this work is to present a procedure to obtain interaction diagrams of RC sections of general shape subjected to fire action, similar to the ones commonly employed at ambient temperature design. Axial force–bending moment envelopes are obtained taking into account the material property degradation due to the temperature increase and the additional thermal strains, under the assumption that spalling does not occur. To illustrate the procedure, interaction diagrams are presented and compared with results from the literature and with the ambient temperature situation.

2. Heat transfer analysis

The purpose of the heat transfer analysis is to obtain the temperature distribution at the cross-section level, which is then used to obtain the thermal strains and evaluate the stiffness and strength degradation. Different numerical schemes may be employed to solve the problem. In the present work, an explicit time integration finite element method was employed, with the cross-section subdivided into a mesh of four-node elements [19,20].

Concrete, steel and fire protection material properties may be taken into account, allowing the thermal analysis of steel, concrete and composite cross-sections. In the thermal analysis reinforcement is neglected, with the temperature of the bars taken equal to the surrounding concrete temperature. Moisture effects, specific heat, thermal conductivity and density are considered according to the equations from Eurocode 2 [7]. After the thermal analysis, the values of the temperatures are available throughout the mesh, which is then employed to evaluate the axial force and bending moments by a fiber approach.

It should be noted, however, that since the procedure for diagram construction is independent of the way the temperature field is obtained, this field may be evaluated by any external software as well as obtained from experiments.

3. Material properties under high temperature

There has been much debate about the most reliable stress–strain constitutive relation for concrete at elevated temperatures. The book by Purkiss [2] provides a review of various models. It is clear that the stress–strain relationship should be able to include phenomena such as creep and transient strains, and different models have been proposed.

For design purposes and the development of the interaction diagrams, the present work adopts the temperature-dependent stress–strain relationship for concrete from the Eurocode 2 [7];

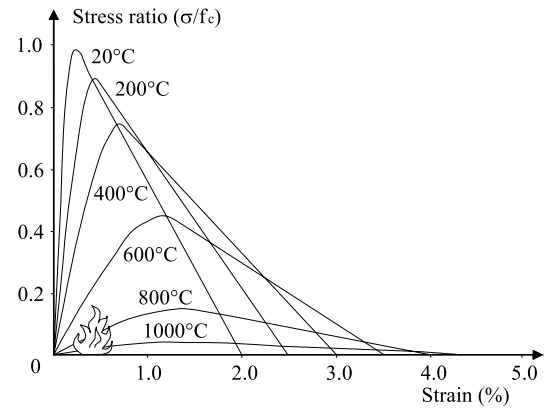


Fig. 1. Stress–strain relationships of concrete at elevated temperature [7].

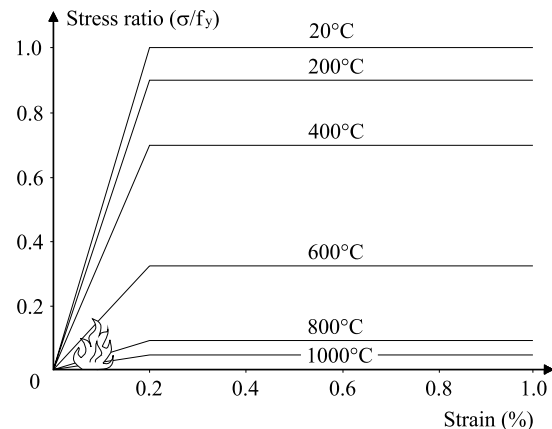


Fig. 2. Stress–strain relationships of steel at elevated temperature [7].

see Fig. 1. The ultimate compressive strain at temperature T is conservatively adopted as the value corresponding to the peak stress. It may be noted from Fig. 1 that, under high temperatures, larger strains may be reached on the descending branch. However, stress–strain diagrams obtained taking into account the transient strain suggest that this attainment of high strain values is not possible [21]. Tensile stresses in the concrete are not taken into account.

For steel, a bilinear stress–strain relation with maximum absolute strain of 0.1 is adopted, considering the strength reduction at 0.2% for class N reinforcement, indicated by Eurocode 2 [7] for use with simplified cross-section calculation methods (Fig. 2).

The stiffness reduction factors and thermal strains for concrete and steel are once again taken from Eurocode 2 [7].

4. Cross-section interaction diagrams

4.1. Ambient temperature

Assuming that the plane sections remain planar and that no slip occurs between the concrete and the steel, the deformed configuration of the section subjected to axial force and biaxial bending at ambient temperature may be described by different sets of generalized strain variables.

- (a) ε_o , k_x and k_y , which represent, respectively, the strain at the reference point (usually the centroid), and curvatures about the x -axis and the y -axis; see Fig. 3. The strain at a point (x, y) is

$$\varepsilon(x, y) = \varepsilon_o + k_x y - k_y x. \quad (1)$$

- (b) ε_o , k_o and α , where k_o is the curvature about the neutral axis (parallel to ξ) and α is the angle of the neutral axis with respect

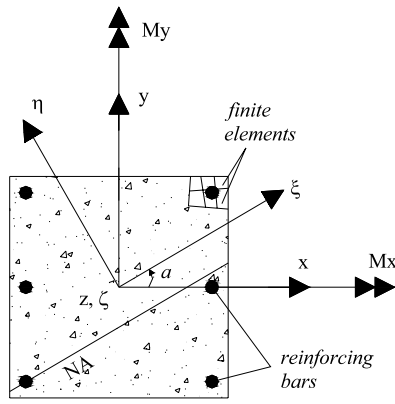


Fig. 3. Cross-section and coordinate systems. NA is the neutral axis.

to x (anticlockwise positive). The strain at a point (ξ, η) is then given by

$$\varepsilon(\xi, \eta) = \varepsilon_0 + k_0 \eta. \quad (2)$$

At ambient temperature design, a particular ultimate configuration of a cross-section is defined by either the steel or the concrete reaching a conventional limit strain value. The total strain is in this case purely mechanical. These limit strain values will occur on fibers or bars that are at the farthest distance, measured along the η -axis. For concrete and steel, these limits are usually taken as 0.003 and 0.01, respectively.

By keeping one of these values fixed for the respective material while varying the plane deformed configuration, and continuously evaluating the axial forces and bending moments, points of the interaction curve (axial force–bending moment) for that specific cross-section and angle of the neutral axis are obtained. A single variable, related to the depth of the neutral axis, is then sufficient to define the curve range for a fixed α .

4.2. Under fire action

In fire conditions (Fig. 4), the presence of thermal strains and the material property degradation introduce considerable difficulties for the cross-section limit state analysis. If the ultimate mechanical strain is taken as the value which corresponds to the peak stress, which also depends on the temperature of the point, it becomes impossible to know beforehand at which point of the section a limit state will be reached first. The mechanical (stress-inducing) strains are given by

$$\begin{aligned} \varepsilon_{mech}(x, y) &= \varepsilon_{total} - \varepsilon_{th} = \varepsilon_0 + k_x y - k_y x - \varepsilon_{th}, \\ \varepsilon(\xi, \eta) &= \varepsilon_{total} - \varepsilon_{th} = \varepsilon_0 + k_0 \eta - \varepsilon_{th}, \end{aligned} \quad (3)$$

where ε_{th} is the thermal strain, a function of temperature $T(x, y)$ or $T(\xi, \eta)$, and the mechanical strain is supposed to include the load-induced thermal strains effects.

A limit state is conventionally attained whenever $\varepsilon_{mech}(\xi, \eta)$ in any fiber of the cross-section or reinforcement reaches its temperature-dependent compressive strain limit value $\varepsilon_{ult}(\xi, \eta)$, taken from a stress–strain curve such as the ones depicted in Fig. 1. As both strains ε_{th} and ε_{ult} depend on the temperature field $T(x, y)$, it is possible to evaluate them for every fiber (finite element) and obtain an approximation of the surface $\varepsilon_{cu}(\xi, \eta) = \varepsilon_{th}(\xi, \eta) + \varepsilon_{ult}(\xi, \eta)$. If any point of the plane $\varepsilon_{total}(\xi, \eta)$ touches this limit surface, the condition for an ultimate state is reached; that is,

$$\varepsilon_{mech} = \varepsilon_{total} - \varepsilon_{th} = \varepsilon_{ult} \Rightarrow \varepsilon_{total} = \varepsilon_{th} + \varepsilon_{ult} = \varepsilon_{cu}. \quad (4)$$

For a fixed neutral axis orientation, it is possible to define an auxiliary variable, related to the neutral axis, which traces the

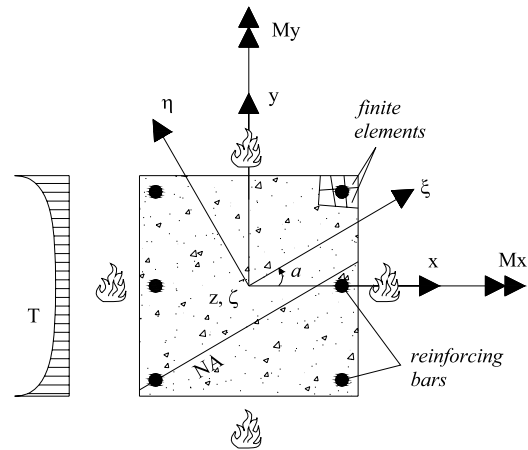


Fig. 4. Coordinate systems xy , $\xi\eta$ and temperature variation.

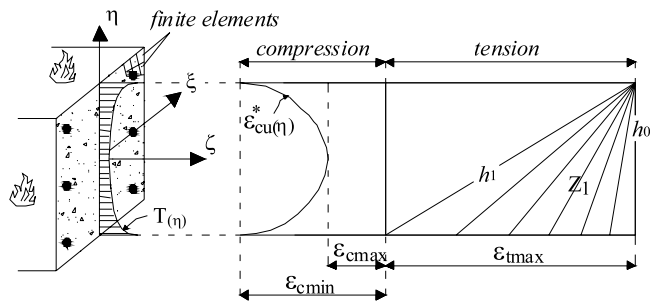


Fig. 5. Zone Z_1 .

evolution of the deformed configuration as the various possibilities of limit state design are swept. This choice is not unique, and one of the possibilities is adopted here as a continuous and dimensionless variable, henceforth named β , whose numerical values, though completely arbitrary, are chosen to give a simple description.

In a section subjected to a generic temperature field, and considering compressive strains negative, let $\varepsilon_{cu}^*(\eta)$ be the projected envelope of $\varepsilon_{cu}(\xi, \eta)$ over the plane which is parallel to η and normal to the cross-section. Moreover, let ε_{cmax} and ε_{cmin} be respectively the maximum and minimum for $\varepsilon_{cu}^*(\eta)$ (Fig. 5), and let ε_{tmax} be the maximum admissible tensile strain, conventionally adopted as 10% or 0.1. Without loss of generality, in the following Figs. 5–8, the case where α is zero is illustrated.

Starting from a limit state characterized by excessive plastic deformation on the steel bars, zone Z_1 is defined (Fig. 5), where the top fiber is ε_{tmax} and the bottom fiber takes values from ε_{tmax} to zero. Plane deformed sections h_0 e h_1 enclose this region, in which concrete has no resistance, and setting $0 \leq \beta \leq 1$, one gets

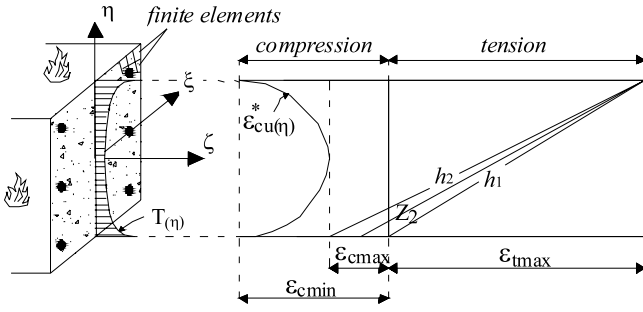
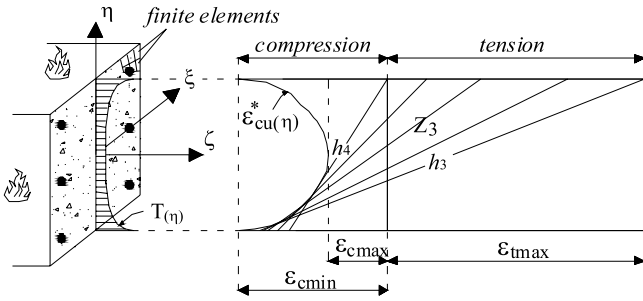
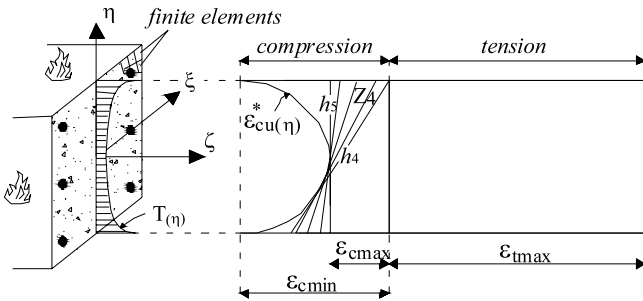
$$\varepsilon^{\eta max} = \varepsilon_{tmax} \quad \text{and} \quad \varepsilon^{\eta min} = (1 - \beta) \varepsilon_{tmax}, \quad (5)$$

where $\varepsilon^{\eta max}$ and $\varepsilon^{\eta min}$ are the strains on the fibers with maximum and minimum η coordinate respectively.

The next zone is defined by keeping the positive strain at the maximum value and entering the compression zone for the concrete, characterizing a flexural state. Deformed sections h_1 e h_2 enclose zone Z_2 (Fig. 6). This limit state is still controlled by the reinforcement. Taking $1 < \beta \leq 8$, one gets

$$\varepsilon^{\eta max} = \varepsilon_{tmax} \quad \text{and} \quad \varepsilon^{\eta min} = (\beta - 1) (\varepsilon_{cmax}/7). \quad (6)$$

The next zone considers deformed configurations with the concrete further into the compressive range, and with reduction on the maximum tensile strain of the top fiber. It starts on line h_3 , where the top fiber has maximum admissible tensile strain and

Fig. 6. Zone Z₂.Fig. 7. Zone Z₃.Fig. 8. Zone Z₄.

finishes on line h_4 , with the top fiber just entering the compression range. The bottom fiber strain $\varepsilon^{\eta \min}$ lies between $\varepsilon_{c \max}$ and $\varepsilon_{c \min}$, and has to be evaluated by a trial process for each value of β . The solution is found when the deformed configuration touches the limit surface. Setting $8 < \beta \leq 18$, zone Z₃ (Fig. 7) is defined by

$$\varepsilon^{\eta \max} = (18 - \beta) (\varepsilon_{t \max} / 10) \quad \text{and} \quad \varepsilon^{\eta \min} = f(\varepsilon^{\eta \max}(\beta)). \quad (7)$$

Proceeding further, zone Z₄ (Fig. 8) lies between lines h_4 and h_5 . The cross-section is fully compressed, and h_5 corresponds to a uniform compressive total strain distribution (but not to pure compression, unless in the case of symmetric geometries and temperature distributions). This zone may be represented, for $18 < \beta \leq 26$, by

$$\varepsilon^{\eta \max} = (\beta - 18) (\varepsilon_{c \max} / 8) \quad \text{and} \quad \varepsilon^{\eta \min} = f(\varepsilon^{\eta \max}(\beta)), \quad (8)$$

where once again $\varepsilon^{\eta \min}$ should be found iteratively for a fixed β .

For $\beta > 26$, there is a completely analogous set of zones, but now with compression mostly on the upper part of the section. The equations for these new zones are given by

$$\varepsilon^{\eta \min} = (34 - \beta) (\varepsilon_{c \max} / 8) \quad \text{and} \quad \varepsilon^{\eta \max} = f(\varepsilon^{\eta \min}(\beta)), \quad (9)$$

$$\varepsilon^{\eta \min} = (\beta - 34) (\varepsilon_{t \max} / 10) \quad \text{and} \quad \varepsilon^{\eta \max} = f(\varepsilon^{\eta \min}(\beta)), \quad (10)$$

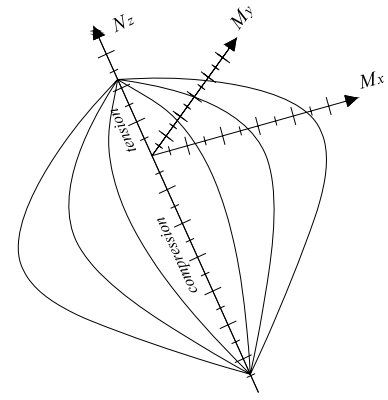


Fig. 9. Complete interaction surface for a cross-section.

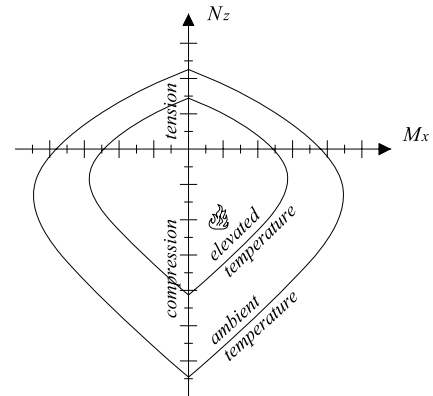


Fig. 10. Interaction diagram N–M, ambient and elevated temperatures.

$$\varepsilon^{\eta \min} = \varepsilon_{t \max} \quad \text{and} \quad \varepsilon^{\eta \max} = (51 - \beta) (\varepsilon_{c \max} / 7), \quad (11)$$

$$\varepsilon^{\eta \min} = \varepsilon_{t \max} \quad \text{and} \quad \varepsilon^{\eta \max} = (\beta - 51) \varepsilon_{t \max}, \quad (12)$$

Given β , and with $\varepsilon^{\eta \max}$ and $\varepsilon^{\eta \min}$ defined, evaluated at the center of the fibers with coordinates $\eta^{\max}$ and $\eta^{\min}$, the generalized strains may be obtained:

$$k_o = (\varepsilon^{\eta \max} - \varepsilon^{\eta \min}) / (\eta_{\max} - \eta_{\min}), \quad (13)$$

$$\varepsilon_o = \varepsilon^{\eta \max} - k_o \eta_{\max}, \quad k_x = k_o \cos(\alpha) \quad \text{and}$$

$$k_y = k_o \sin(\alpha).$$

With ε_o , k_x and k_y , the resistant forces of the concrete cross-section are evaluated by numerical integration through the elements:

$$N_z = \sum_{i=1}^n (\sigma(\varepsilon) A)_i, \quad M_x = \sum_{i=1}^n (\sigma(\varepsilon) y A)_i \quad \text{and} \quad (14)$$

$$M_y = - \sum_{i=1}^n (\sigma(\varepsilon) x A)_i,$$

with σ obtained from the concrete stress–strain relation for the temperature at the center of the fiber, and ε evaluated from Eq. (3) at the same point. The reinforcement contribution is added to Eq. (14) and its temperature is assumed to be equal to the temperature in the corresponding position of the mesh.

With the aid of the β parameter, with $0 \leq \beta \leq 52$, and for discrete values of α , one may evaluate as many (N_z, M_x, M_y) points as necessary to obtain a description of the interaction surface by its approximate meridian lines (Fig. 9). The intersection of this surface with one of the zero-moment planes gives N–M interaction diagrams for uniaxial bending (Fig. 10). Another useful

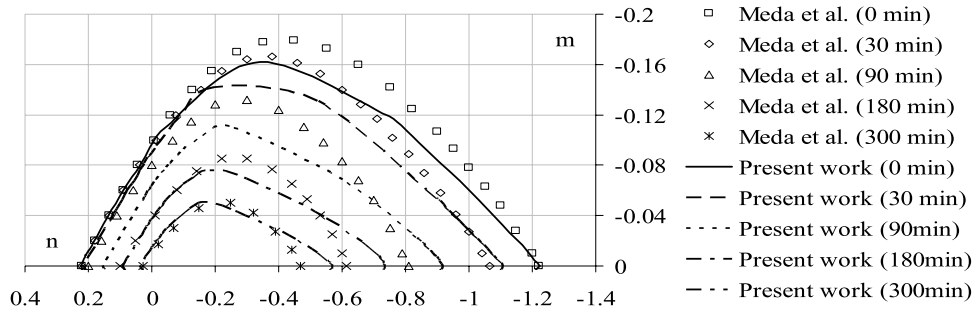


Fig. 11. Comparison of interaction curves with those of Meda et al. [18].

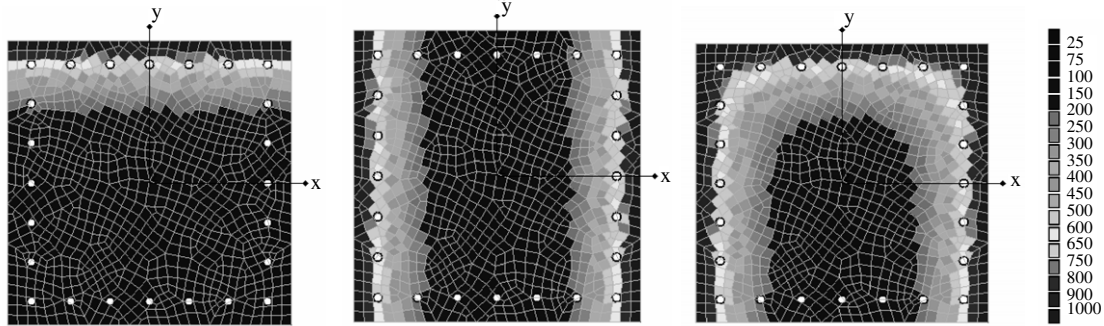


Fig. 12. Temperature for 300 min exposure: one-, two- and three-face heating.

representation considers a fixed value of axial force and presents the closed interaction curve in the M_x – M_y space.

5. Examples

In this section, the presented formulation is employed to obtain interaction diagrams for reinforced concrete sections submitted to fire action, and to investigate its influence on the shapes of these diagrams at ambient temperature.

5.1. Reinforced concrete section

A square normal-strength concrete section exposed to ISO 834 standard fire and studied by Meda et al. [18] was analyzed by the present formulation. The following properties are employed: area of $0.6 \times 0.6 \text{ m}^2$; 24 bars with 20 mm diameter and 50 mm axial distance from the closest boundary; 40 MPa concrete compressive strength; 430 MPa reinforcement yield stress and $2 \times 10^5 \text{ MPa}$ elastic modulus. The section was subdivided into 1022 four-node elements. The thermal properties are in accordance with Eurocode 2 [7] for concrete with siliceous aggregate and 0% moisture.

Fig. 11 shows the dimensionless axial force–bending moment interaction diagrams, and those obtained by Meda et al. [18], based on $n = N/(f_c b h)$ and $m = M/(f_c b h^2)$, where b is the width and h is the height of the section.

For a fixed compressive axial force, the moment resistance obtained with the present formulation is in general conservative. This is due to the consideration of the limit strain for concrete as the peak stress correspondent value, which underestimates the compression block resultant. On the other hand, for high temperatures, the compressive axial force is higher, due to the fact that the thermal strain is taken into account in the determination of the deformed configuration. In this way, points with higher temperatures, but less resistance, also contribute to the axial force. With positive axial force, the results compare well. In general terms, even with all the different assumptions, there is a good agreement with the results by Meda et al. [18].

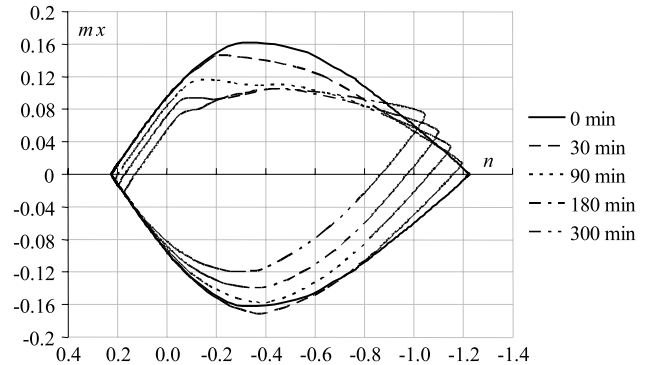


Fig. 13. One-face heating interaction diagrams, $\alpha = 0^\circ$.

In order to illustrate the possibility of applying the procedure to different fire exposure regimes, and to investigate the behavior of the diagrams, the same section is subjected to one-, two- and three-face exposures to the standard fire (see Fig. 12 for temperatures at 300 min). The diagrams are constructed for 0, 30, 90, 180 and 300 min of fire exposure. In all cases the non-exposed face is considered as protected by adiabatic material, with no heat flux. The procedure is carried out for two orientations of neutral axis, $\alpha = 0^\circ$ (flexure about the x-axis) and $\alpha = 90^\circ$ (flexure about the y-axis). For $\alpha = 0^\circ$, it is possible to build diagrams for uniaxial bending ($M_y = 0$) as the fire action on all three cases is symmetric with respect to y.

Figs. 13 and 14 show the interaction curves for one-face heating, for $\alpha = 0^\circ$ and $\alpha = 90^\circ$ respectively. The first situation is clearly a uniaxial bending diagram as no moments about y arise. The interaction diagram contracts and also becomes non-symmetric with respect to the axial force. Therefore, there is a translation of the mechanical axis so that pure compression is not obtained unless the load is applied eccentrically. It should be noted that, with $\alpha = 90^\circ$, the diagram in Fig. 14 is merely the projection of an interaction surface on that plane, since out-of-plane moments about x are present due to the lack of symmetry.

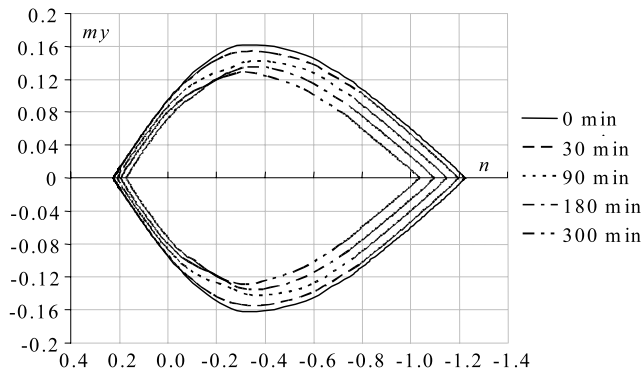
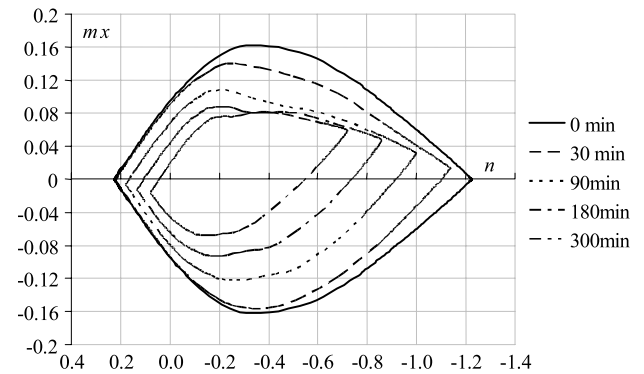
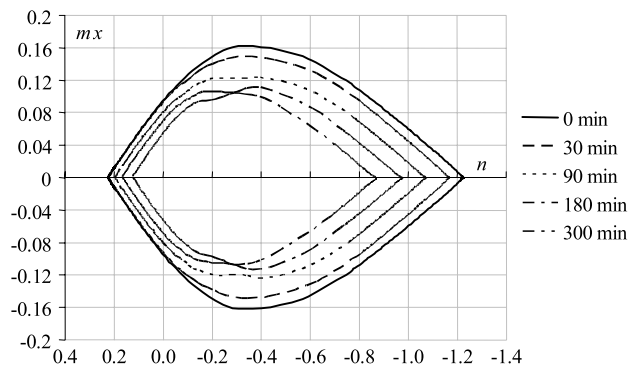
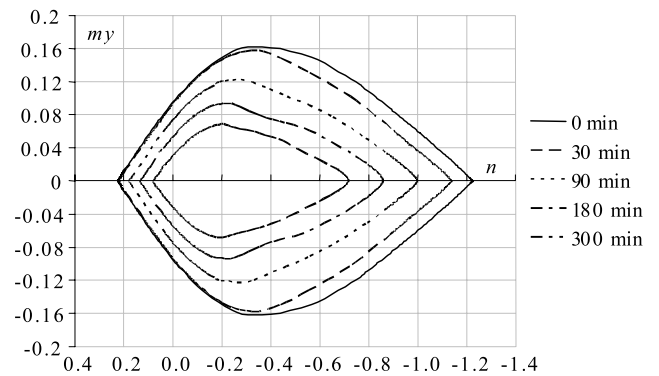
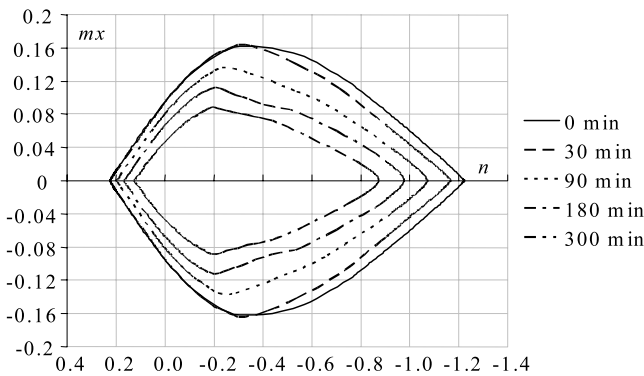
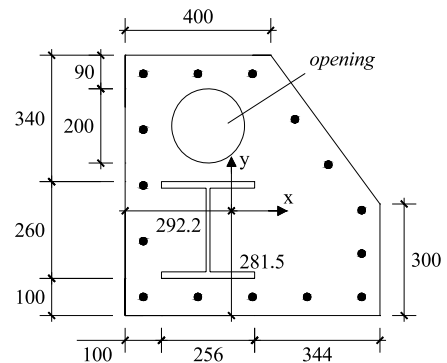
Fig. 14. One-face heating interaction diagrams, $\alpha = 90^\circ$.Fig. 17. Three-face heating interaction diagrams, $\alpha = 0^\circ$.Fig. 15. Two-face heating interaction diagrams, $\alpha = 0^\circ$.Fig. 18. Three-face heating interaction diagrams, $\alpha = 90^\circ$.Fig. 16. Two-face heating interaction diagrams, $\alpha = 90^\circ$.

Fig. 19. Generic composite cross-section (from [22], dimensions in mm).

Figs. 15 and 16 show the interaction diagrams for the two-face heating, with $\alpha = 0^\circ$ and $\alpha = 90^\circ$, respectively. In this case, the symmetry of the section and of the temperature field cause uniform contraction of the diagrams. Finally, Figs. 17 and 18 depict the diagrams for the three-face heating situation, and conclusions analogous to the one-face heating regime may be drawn.

5.2. Arbitrary composite section

The composite cross-section with 15 reinforcement bars, a circular opening and an embedded steel profile shown in Fig. 19 was analyzed at ambient temperature, and its interaction diagram obtained for a normal force of 4120 kN by Chen et al. [22]. In order to investigate the influence of fire action on the strength envelope, the section is subjected to the standard fire [23]. The following properties are used in the analysis of the section: reinforcement bars with 18 mm diameter and 50 mm axial distance from the nearest boundary; steel profile with 10.5 mm web thickness,

260 mm height, 17.3 mm flange thickness, 256 mm web width and 323 MPa yield stress; 20 MPa concrete compressive strength; 400 MPa reinforcement yield stress, and 2×10^5 MPa elastic modulus. The section was subdivided into 889 four-noded elements, and the thermal properties are taken from Eurocode 2 [7] for siliceous aggregates with 0% moisture.

The temperature field for a 300 min exposure is depicted in Fig. 20. Fig. 21 displays the interaction diagram of the biaxial bending moments for a value of the axial force equal to 4120 kN (compression, the same value as used in [22]), for ambient temperature and for several exposure times. The contraction of the diagram for higher exposure times illustrates the loss of load-carrying capacity for the cross-section. The ambient temperature results for the present formulation are conservative, due to the assumption of low limit strains for concrete. Chen et al. [22] employed the parabolic–rectangular stress–strain relationship, assuming higher concrete strain limits.

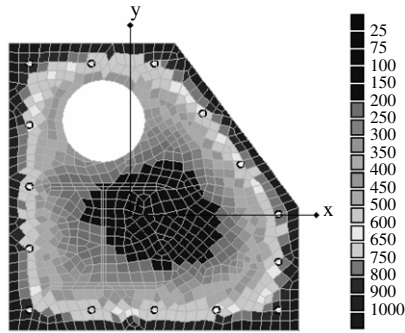


Fig. 20. Temperature distribution for 300 min.

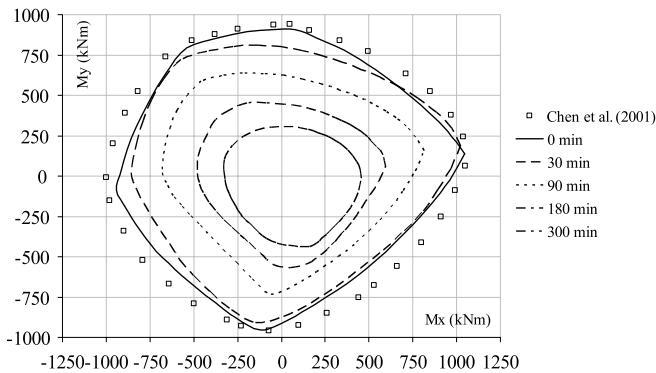


Fig. 21. Interaction diagrams, normal force 4120 kN.

6. Summary and conclusions

Interaction diagrams are common tools for the design of reinforced concrete sections and beam–columns at ambient temperature. However, few works on this topic have been published for the case of sections subjected to high temperatures. The purpose of this work was to develop and test a procedure for the evaluation of axial force–bending moment interaction curves and surfaces for reinforced concrete sections subjected to an arbitrary complex temperature field. Generic cross-sections and different fire exposure regimes may be considered, resulting in a flexible and powerful tool.

The interaction diagrams obtained from the formulation provide an improved understanding of the cross-section behavior when subjected to high temperature, especially for the cases when the temperature variation is complex, and the diagrams deviate considerably from their ambient temperature counterparts. This was shown by means of two examples, one for a common square symmetrical section and the other for an arbitrarily shaped section.

Some assumptions made for the construction of the diagrams, however, need to be addressed by further numerical and/or experimental research, such as the choice for the limit strain for the concrete, which was taken here in order to produce results on the safe side. The stress–strain relation for concrete under high temperatures adopted was that of the Eurocode 2, but other relations which take into account creep and transient strains may be more realistic [21]. Moreover, the possibility of spalling was not

considered, and accurate prediction of the extent and evolution of surface spalling is still a very difficult task. This should restrict the applicability of the diagrams to normal-strength concrete sections where spalling is less prone to happen.

The availability of such interaction diagrams under fire conditions may be helpful for the design of RC columns under fire conditions, either by numerical methods or simplified schemes. Ongoing research will focus on the development of procedures for the design of RC columns with the aid of the generated diagrams.

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