



Mechanical behaviour of full unit masonry panels under fire action

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ABSTRACT

An analytical model is presented to predict the mechanical resistance of full unit masonry panels under combined compressive and bending stress, in the presence of a nominal fire acting on one side. Transitory thermal analysis of the panels is conducted, while accounting for the convection and radiation effects produced by the fire, to determine the collapse conditions for combined compression and bending; five strain field collapse configurations are considered. The numerical analyses, based on data drawn from European regulations EN 1996-1-2, permit the determination of the *M–N* crushing domains for brick and lightweight aggregate concrete masonry walls, with various thicknesses and exposure times to nominal fire. A numerical example demonstrates the applicability of the model, and a comparison with a set of experimental data on masonry panels demonstrates the efficiency of the model.

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Introduction

The fire behaviour of masonry elements has prompted a great deal of interest in building construction because of the masonry's considerable capacity to isolate parts of a building's interior from both the fire itself and the effects of smoke (so-called compartment walls). In addition, masonry is quite resistant to weakening with increasing temperatures. In fact, the structural blocks, generally made up of clay or lightweight aggregate concrete (pumice), maintain their bearing and deformational capacities for a wide range of temperatures, and even when they do degrade, they do so gradually. In addition, the joint mortar generally exhibits favourable properties in terms of resistance and response to fire.

Although the response of masonry walls to fire has been the subject of a good deal of study in the past, only recently have experimental results been compared with those from numerical models. Nadjai et al. [1,2] report on numerical studies of the behaviour of walls conducted in the presence of fire on one side. They assumed two preset curves of the compression strength–temperature relation proposed by Abrams [3] and Thelandersson [4] and two possible curves for the crushing strain–temperature relation proposed by Terro [5] and Anderberg and Thelandersson [6]. The stress–strain curve shown in Fig. 1 depicts a classical relation, with two descending segments whose shapes differ in the two regions of different strain signs, while the isothermal

lines have been calculated under the hypothesis of a linear distribution of temperatures throughout the thickness.

Al Nahhas et al. [7] address the case of walls made of double-cavity cored bricks, which were subjected to experimental tests and subsequent analytical modelling. The distribution of the isotherms was determined through an energy-balance approach by measuring the proportions of convection, conduction and radiation: the temperature–time curves were determined at various points throughout the wall thickness. The influence of changes in the material mechanical strength with varying temperature was, however, not modelled. In order to check the experimental results reported by Shields et al. [9], Dhanasekar et al. carried out numerical analysis in a thermal field to determine the bowing of panels exposed to fire [8].

With regard to regulations, the recent adoption of the Eurocodes on structures, in particular the adjustments to EN 1996-1-2 [24], has led to the application of new methods for experimental or analytical tabular design of bearing masonry walls, in part based on the results of a series of experimental trials conducted by Hahn et al. [10]. The analytical methods prescribed in the Eurocodes depend on knowing the ultimate strain and compression resistance as functions of temperature, as well as the main thermal parameters (conductivity, specific heat and density). It should, however, be noted that such parameters are significantly influenced by brick humidity, so caution should be exercised when applying standard diagrams in the design process.

As is already the case for reinforced concrete, steel and wood constructions, in planning masonry structures, it would be useful for designers to know the fracture domains, *N–e* (axial force–eccentricity) or *M–N* (bending moment–axial force), which enable rapid calculation of the bearing capacity of a wall with preset

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exposure times to fire. The method proposed herein aims to define such M – N diagrams for walls subjected to the eccentric normal force acting on various types of blocks exposed to fire on one side. To this end, the temperature distributions across the wall thickness are first determined. Then, as the laws governing the decay of the material resistance and axial stiffness as a function of the temperature are known, the wall crushing strain fields are calculated as a function of the curvature. Lastly, based on the isotherms already calculated and the stress–strain–temperature constitutive relation, we determine the crushing surfaces on the plane M – N (in which M is the out-of-plane bending moment and N is the axial force) for increasing exposure time to nominal fire. Application of the procedure is illustrated through a simple example.

1. Thermal analysis

We shall refer to a wall of height h , thickness t and unit depth, subjected to a thermal transient due to a heat source acting on one side.

The convective thermal action in terms of net unitary heat flow is

$$\dot{h}_{net,c} = \alpha_c (\Theta_g - \Theta_m) (W/m^2), \quad (1)$$

where Θ_m is the element's surface temperature ($^{\circ}C$), while the temperature Θ_g of the gases produced by fire is determined via the expression for the nominal temperature–time curve

$$\Theta_g = 20 + 345 \log_{10}(8t + 1) (^{\circ}C), \quad (2)$$

The heat transmission coefficient for convection α_c has been taken as $25 W/m^2 K$, as indicated in [10] EN 1991-1-2.

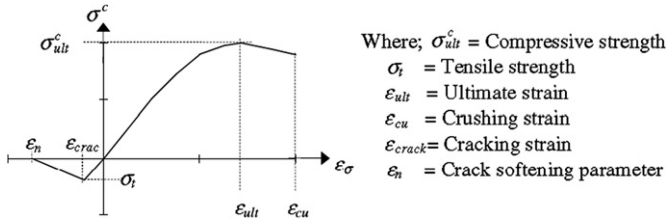


Fig. 1. Stress–strain relation of masonry materials under compression and tension, drawn from [1,2].

The net unitary heat flow due to irradiation is

$$\dot{h}_{net,r} = \Phi \varepsilon_m \varepsilon_f \sigma [(\Theta_r + 273)^4 - (\Theta_m + 273)^4] (W/m^2), \quad (3)$$

with the emissivity $\varepsilon_f = 1.0$ and temperature Θ_r tied to the normalised curve (2). The Stephan–Boltzmann constant σ is $5.67 \times 10^{-8} W/m^2 K^4$, while the emissivity value for the receiving surface, ε_m , has been set to 0.8, as recommended in [22] EN 1991-1-2. The configuration coefficient Φ has been assumed equal to 1.

The nonlinear thermal transitory was analysed via the code *ANSYS Multiphysics rel. 11.0* using triangular (or quadrangular) shaped plane elements (*PLANE55 “2-D Thermal Solid”*) of dimensions varying from 1 to 2 cm.

The initial temperature was $20^{\circ}C$, while a unit convective thermal flow was applied to the surface not exposed to the fire, using a transmission coefficient α_c of $4 W/m^2 K$. The functions for specific heat $c_a(T)$ and transmissibility $\lambda_a(T)$ have been drawn from Appendix D of EN 1996-1-2 [24], as shown in Fig. 2 (brick with density $\rho = 1000 kg/m^3$) and Fig. 3 (lightweight aggregate concrete with density $\rho = 800 kg/m^3$).

The results of such analyses of the nonlinear thermal transitory are presented in the plots of temperature T versus the thickness x for various exposure times to nominal fire (Figs. 4 and 5).

The same procedure has also been performed on a panel of aerated autoclaved concrete (AAC) blocks to obtain numerical results for a comparison with a set of experimental data.

2. Mechanical analysis

2.1. Description of the mathematical model and basic hypotheses

In the model, failure of the masonry is associated with the attainment of crushing strain conditions in some point of a generic cross section. The basic underlying hypotheses are the following:

- cross sections remain planar (*Bernoulli–Navier* hypothesis);
- the material does not react to tension;
- the material is elastic–perfectly plastic (i.e., *Prandtl* constitutive relation);
- the material responds to the “Maximum strain” yield criterion.

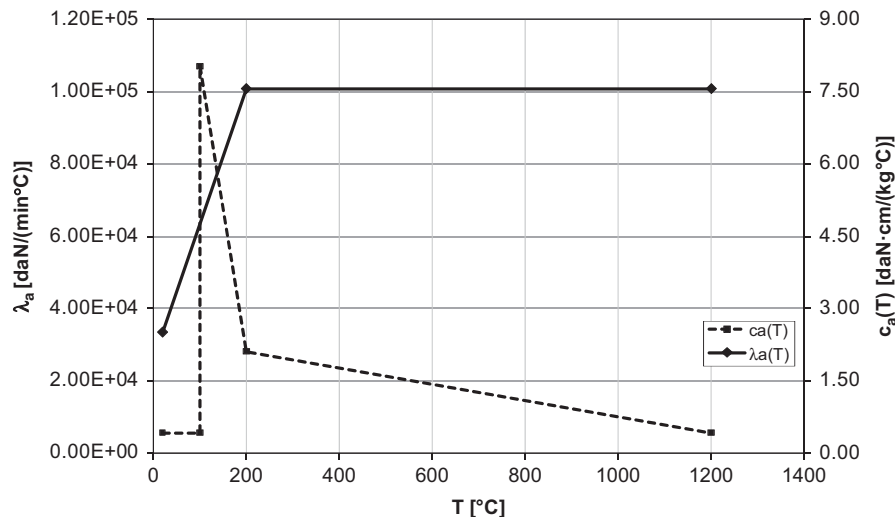


Fig. 2. Thermal conductivity $\lambda_a(T)$ [da N/min $^{\circ}C$] and specific heat $c_a(T)$ [da N cm/kg $^{\circ}C$] functions adopted for the brick masonry.

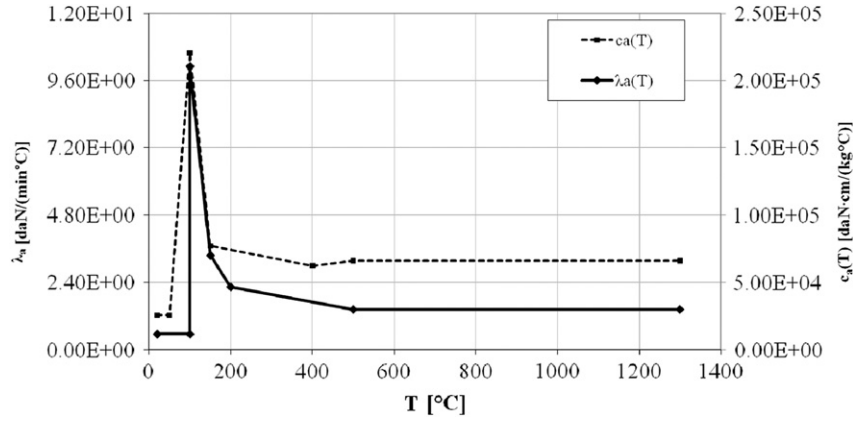


Fig. 3. Thermal conductivity $\lambda_a(T)$ [da N/min °C] and specific heat $c_a(T)$ [da N cm/kg °C] functions adopted for the lightweight concrete blocks.

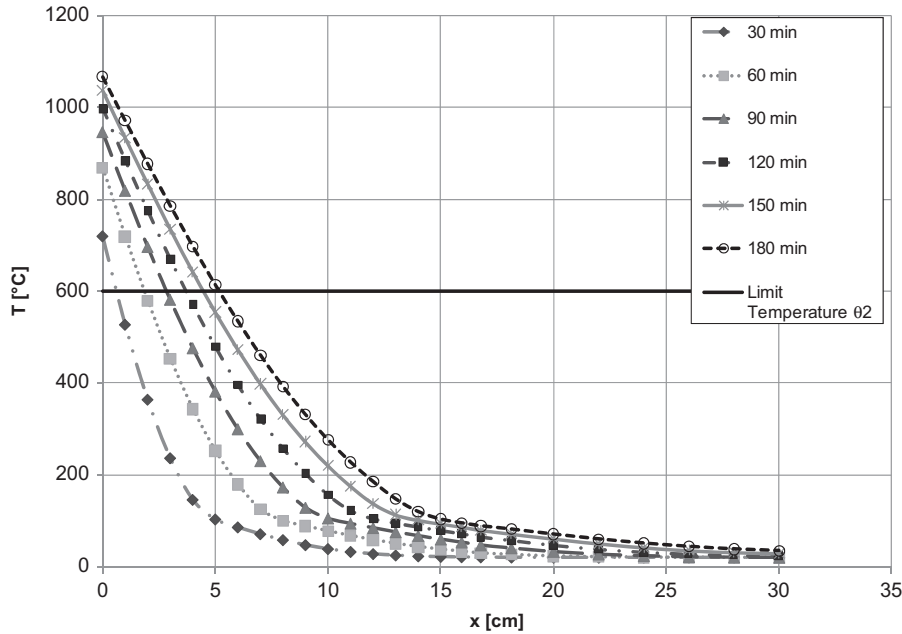


Fig. 4. Brick wall: temperature profile for various exposure times. The value of θ_2 (the temperature beyond which the material is to be considered ineffective) is indicated at 600 °C, as per Appendix C of EN 1996-1-2 [24].

In order to simplify the analysis, it has also been considered that thermal strain ε_{th} , transient strain ε_{tr} and slenderness effects are formally computed increasing the applied stress.

Referring to the quantities indicated in Fig. 6, with a fixed exposure time to the nominal fire, the results of the thermal analyses lead to the following function for the temperature profile:

$$T = T(X). \quad (4)$$

The term X is the abscissa with its origin on the face exposed to fire, x is the abscissa whose origin lies on the border of the effectively resistant section, t_{ineff} is the depth of the portion of wall deemed ineffective, t_{Fr} is the thickness of the still reacting part, and t is the total thickness of the wall itself. By placing

$$\begin{cases} X = x + t_{ineff} \\ x = X - t_{ineff} \end{cases}, \quad (5)$$

the value of the temperature T as a function of x can be obtained

$$T = T(x). \quad (6)$$

By observing the behaviour of the resistance $f_{cu}(T)$, the elasticity modulus $E(T)$ and the crushing strain $\varepsilon_u(T)$, from Fig. 6, it follows immediately that:

$$f_{cu} = f_{cu}(x), \quad (7)$$

$$E = E(x), \quad (8)$$

$$\varepsilon_u = \varepsilon_u(x), \quad (9)$$

For this last equation, we assume, where definable, the condition

$$\frac{\partial^2 \varepsilon_u}{\partial x^2} \geq 0. \quad (10)$$

The distributions $\varepsilon(x)$ vary within five limit fields, as illustrated in Fig. 7, where t_{θ_1} indicates the thickness of the part between the unexposed face and isotherm θ_1 (limit temperature for material integrity) and t_{θ_2} the portion whose temperature varies between θ_1 itself and θ_2 (limit temperature for material ineffectiveness), both furnished by EN 1996-1-2 [24].

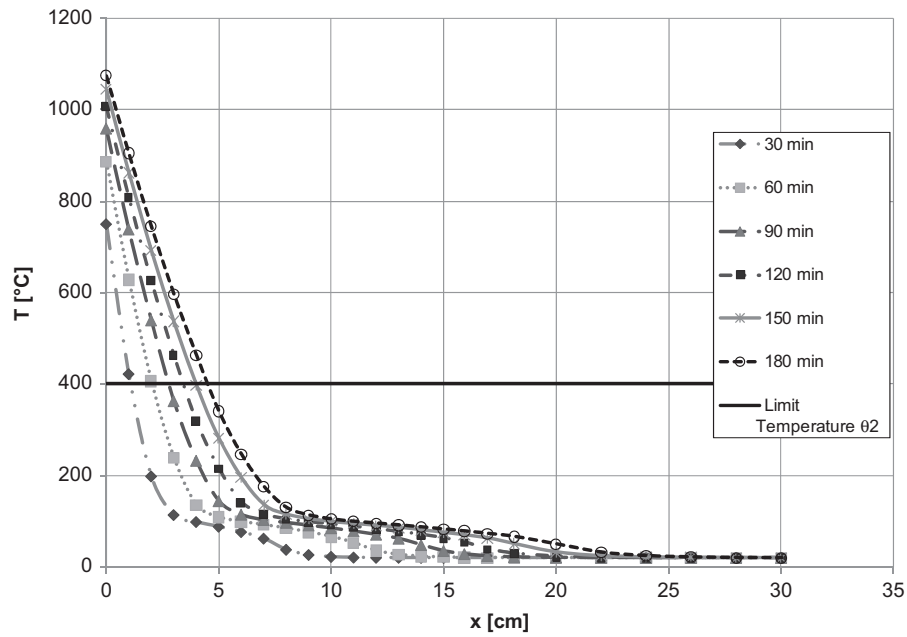


Fig. 5. Lightweight aggregate concrete masonry wall: temperature profile for various exposure times. The value of θ_2 (the temperature beyond which the material is to be considered ineffective) is indicated at 400 $^{\circ}\text{C}$, as per Appendix C of EN 1996-1-2 [24].

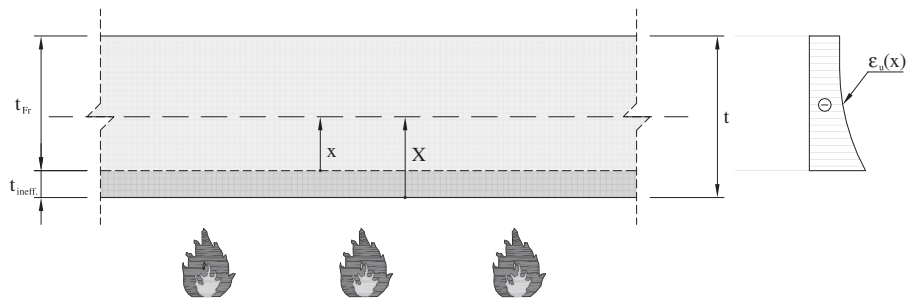


Fig. 6. Horizontal cross section of the wall; the geometric parameters are indicated, together with a diagram of the crushing strain diagram $\epsilon_u(x)$.

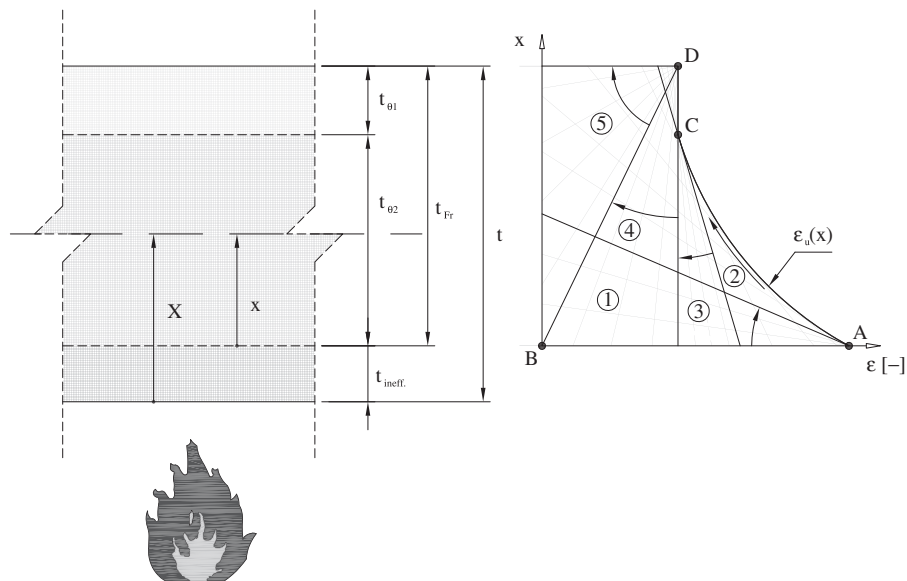


Fig. 7. Limit ϵ fields.

2.1.1. Field 1

Field 1 (Fig. 8) represents the subset of the sheaf of straight lines at point

$$A \equiv (0; \varepsilon_u(0)), \quad (11)$$

whose directive coefficient χ , or section curvature, belongs to the following domain:

$$\chi \in]-\infty; \frac{\partial \varepsilon_u(x)}{\partial x} \Big|_{x=0}], \quad (12)$$

and which is defined by

$$\varepsilon(x) = \chi x + \varepsilon_u(0). \quad (13)$$

By means of the constitutive relation, the function of the normal stress $\sigma(x)$ through the thickness, which in the *compressed zone*, equals

$$\sigma(x) = \begin{cases} E(x)\varepsilon(x) & \text{if } E(x)\varepsilon(x) < f_u(x) \\ f_u(x) & \text{if } E(x)\varepsilon(x) \geq f_u(x) \end{cases}, \quad \forall x \in [0; x_1(\chi)], \quad (14)$$

while in the *tensile zone*,

$$\sigma(x) = 0, \quad \forall x \in [x_1(\chi); t_{Fr}]. \quad (15)$$

In previous expressions, x_1 denotes the following function:

$$x_1(\chi) = \begin{cases} -\frac{\varepsilon_u(0)}{\chi} & \text{if } \chi < -\frac{\varepsilon_u(0)}{t_{Fr}} \Rightarrow \text{large eccentricity} \\ t_{Fr} & \text{if } \chi \geq -\frac{\varepsilon_u(0)}{t_{Fr}} \Rightarrow \text{small eccentricity} \end{cases}. \quad (16)$$

2.1.2. Field 2

Field 2 represents the subset of the straight lines tangential to the curve in Fig. 9, in which χ varies within the following existence field:

$$\chi \in \left[\frac{\partial \varepsilon_u(x)}{\partial x} \Big|_{x=0}; \lim_{\eta \rightarrow 0^+} \frac{\varepsilon_u(t_{\theta 2} + \eta) - \varepsilon_u(t_{\theta 2})}{\eta} \right], \quad (17)$$

where $\bar{x}$ is the generic abscissa of wall thickness belonging to the domain

$$\bar{x} \in [0; t_{\theta 2}]. \quad (18)$$

The equation of the straight lines in this field is

$$\varepsilon(x) = \frac{\partial \varepsilon_u(x)}{\partial x} \Big|_{x=\bar{x}} (x - \bar{x}) + \varepsilon_u(\bar{x}). \quad (19)$$

From the constitutive relation, the normal stresses $\sigma(x)$ in the *compressed zone* are

$$\sigma(x) = \begin{cases} E(x)\varepsilon(x) & \text{if } E(x)\varepsilon(x) < f_u(x) \\ f_u(x) & \text{if } E(x)\varepsilon(x) \geq f_u(x) \end{cases}, \quad \forall x \in [0; x_2(\bar{x})], \quad (20)$$

while in the *tensile zone*,

$$\sigma(x) = 0, \quad \forall x \in [x_2(\bar{x}); t_{Fr}]. \quad (21)$$

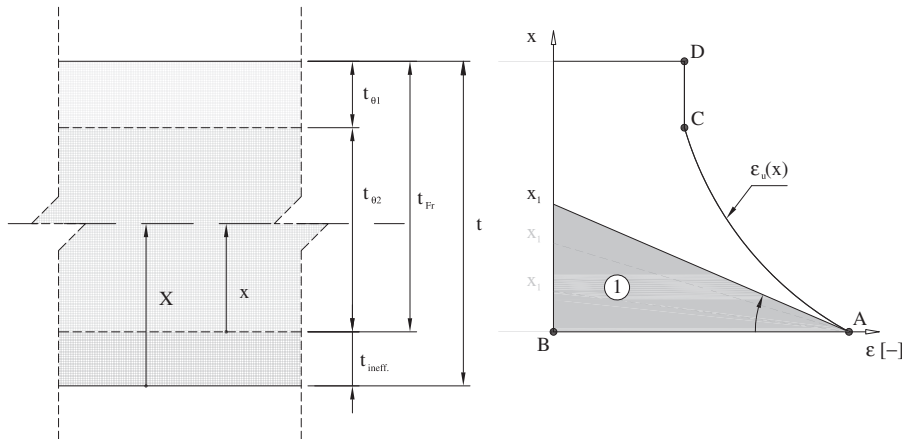


Fig. 8. Field 1.

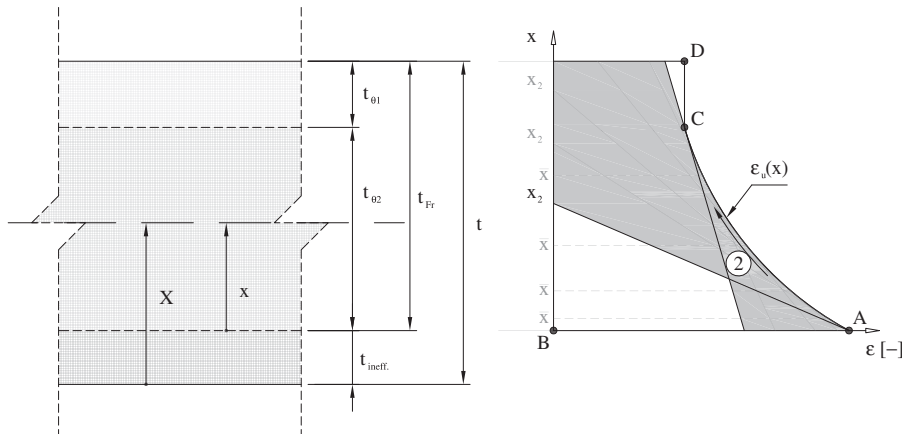


Fig. 9. Field 2.

In the previous expressions, x_2 denotes

$$x_2(\bar{x}) = \begin{cases} \frac{-\varepsilon_u(\bar{x})}{(\partial \varepsilon_u(x)/\partial x)|_{x=\bar{x}}} + \bar{x} & \text{if } \varepsilon_u(\bar{x}) < \frac{\partial \varepsilon_u(x)}{\partial x} \Big|_{x=\bar{x}}(\bar{x}-t_{Fr}) \Rightarrow \text{large eccentricity} \\ t_{Fr} & \text{if } \varepsilon_u(\bar{x}) \geq \frac{\partial \varepsilon_u(x)}{\partial x} \Big|_{x=\bar{x}}(\bar{x}-t_{Fr}) \Rightarrow \text{small eccentricity} \end{cases} \quad (22)$$

2.1.3. Field 3

Field 3 (Fig. 10) accounts for the possibility that the function $\varepsilon_u(x)$ not be derivable at the point C whose coordinates are

$$C \equiv (t_{\theta 2}; \varepsilon_u(t_{\theta 2})). \quad (23)$$

Such a field only exists under the aforementioned condition or in the event that the entire section is at a temperature above θ_1 , which would imply that

$$t_{\theta 1} = 0; \quad (24)$$

otherwise, we proceed directly to examination of Field 4.

Bearing in mind the foregoing premises, Field 3 is the subset of the sheaf of straight lines at point C where χ falls within the interval

$$\chi \in \left[\lim_{\eta \rightarrow 0} \frac{\varepsilon_u(t_{\theta 2} + \eta) - \varepsilon_u(t_{\theta 2})}{\eta}; 0 \right], \quad (25)$$

and whose equation is

$$\varepsilon(x) = \chi(x - t_{\theta 2}) + \varepsilon_u(t_{\theta 2}). \quad (26)$$

The normal stresses $\sigma(x)$ in the compressed zone are

$$\sigma(x) = \begin{cases} E(x)\varepsilon(x) & \text{if } E(x)\varepsilon(x) < f_u(x) \\ f_u(x) & \text{if } E(x)\varepsilon(x) \geq f_u(x) \end{cases}, \quad \forall x \in [0; x_3(\chi)], \quad (27)$$

while in the tensile zone,

$$\sigma(x) = 0, \quad \forall x \in]x_3(\chi); t_{Fr}]. \quad (28)$$

In the preceding expressions, x_3 indicates the following function:

$$x_3(\chi) = \begin{cases} t_{\theta 2} - \frac{\varepsilon_u(t_{\theta 2})}{\chi} & \text{if } \chi < -\frac{\varepsilon_u(t_{\theta 2})}{t_{\theta 1}} \Rightarrow \text{large eccentricity} \\ t_{Fr} & \text{if } \chi \geq -\frac{\varepsilon_u(t_{\theta 2})}{t_{\theta 1}} \Rightarrow \text{small eccentricity} \end{cases} \quad (29)$$

2.1.4. Field 4

Field 4 (Fig. 11) represents the subset of the sheaf of straight lines of the point

$$D \equiv (t_{Fr}; \varepsilon_u(t_{Fr})), \quad (30)$$

whose directive coefficient χ falls within the following interval:

$$\chi \in \left[0; \frac{\varepsilon_u(t_{Fr})}{t_{Fr}} \right], \quad (31)$$

and whose equation is

$$\varepsilon(x) = \chi(x - t_{Fr}) + \varepsilon_u(t_{Fr}). \quad (32)$$

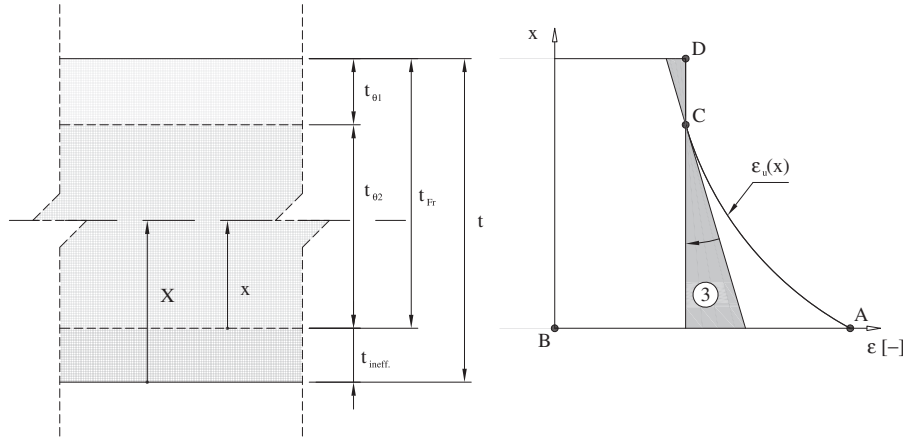


Fig. 10. Field 3.

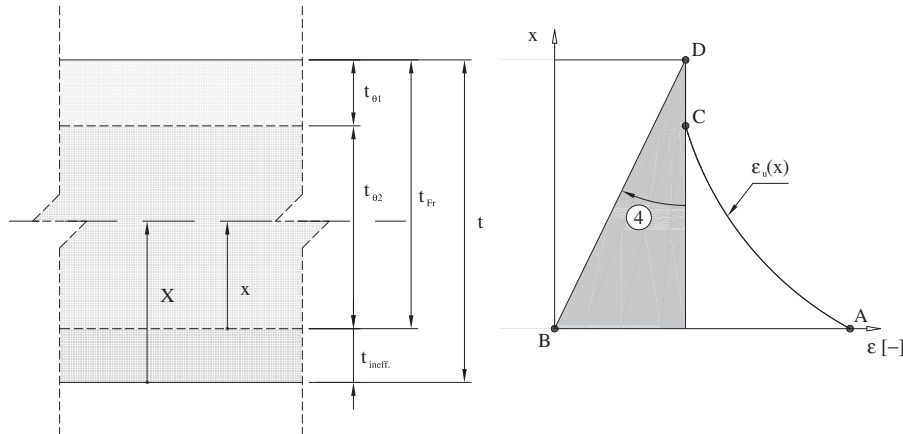


Fig. 11. Field 4.

The normal stresses $\sigma(x)$ within the thickness are

$$\sigma(x) = \begin{cases} E(x)\varepsilon(x) & \text{if } E(x)\varepsilon(x) < f_u(x) \\ f_u(x) & \text{if } E(x)\varepsilon(x) \geq f_u(x) \end{cases}, \quad \forall x \in [0; t_{Fr}]. \quad (33)$$

2.1.5. Field 5

Field 5 (Fig. 12) represents the subset of the sheaf of straight lines of point D whose coordinates are given by Eq. (30), whose directive coefficient χ falls within the following interval:

$$\chi \in \left[\frac{\varepsilon_u(t_{Fr})}{t_{Fr}}; +\infty \right], \quad (34)$$

and whose equation is still Eq. (32).

The normal stresses $\sigma(x)$ in the *compressed zone* are

$$\sigma(x) = \begin{cases} E(x)\varepsilon(x) & \text{if } E(x)\varepsilon(x) < f_u(x) \\ f_u(x) & \text{if } E(x)\varepsilon(x) \geq f_u(x) \end{cases}, \quad \forall x \in [x_5(\chi); t_{Fr}], \quad (35)$$

while in the *tensile zone*,

$$\sigma(x) = 0, \quad \forall x \in [0; x_5(\chi)]. \quad (36)$$

In the preceding expressions, x_5 indicates the following function:

$$x_5(\chi) = t_{Fr} - \frac{\varepsilon_u(t_{Fr})}{\chi}. \quad (37)$$

2.1.6. Definition of the M–N crushing domain

By integration, we obtain the curvature functions $N_u(\chi)$ and $M_u(\chi)$ for fields 1, 3, 4 and 5 via Eqs. (38) and (39), and the functions for the generic abscissa through the thickness, $N_u(\bar{x})$ and $M_u(\bar{x})$ for field 2 via Eqs. (40) and (41). Therefore, the boundary points of the resistance domain M–N can be determined for any given value of χ or $\bar{x}$

$$N_u(\chi) = \int_{a(\chi)}^b \sigma(x) dx, \quad (38)$$

$$M_u(\chi) = \int_{a(\chi)}^b \sigma(x) \left(t_{Fr} - x - \frac{t}{2} \right) dx, \quad (39)$$

$$N_u(\bar{x}) = \int_{a(\bar{x})}^b \sigma(x) dx, \quad (40)$$

$$M_u(\bar{x}) = \int_{a(\bar{x})}^b \sigma(x) \left(t_{Fr} - x - \frac{t}{2} \right) dx, \quad (41)$$

The integration extremes a and b in the preceding expressions are indicated in Table 1.

2.2. Application of the model and presentation of results

The mathematical model already presented has been applied to materials subjected to preliminary thermal analyses, and the results obtained are described here.

2.2.1. Lightweight aggregate concrete (pumice)

With reference to the stress–strain curves reported in Appendix D of EN 1996-1-2 [24] (Fig. 13), a substantial simplification is applied to determine a *Prandtl*-type constitutive relation that

Table 1
Integration extremes in relations (38)–(41).

Field	a	b
1	0	$x_1(\chi)$
2	0	$x_2(\bar{x})$
3	0	$x_3(\chi)$
4	0	t_{Fr}
5	$x_5(\chi)$	t_{Fr}

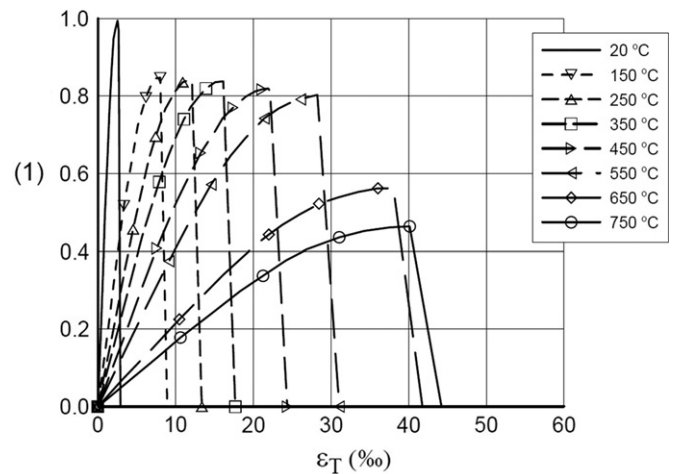


Fig. 13. Constitutive relation of lightweight aggregate concrete (pumice) as a function of temperature, as per Appendix D of EN 1996-1-2 [24]; (1) denotes the ratio between the normal stress value at any given temperature T and the compression strength at 20 °C.

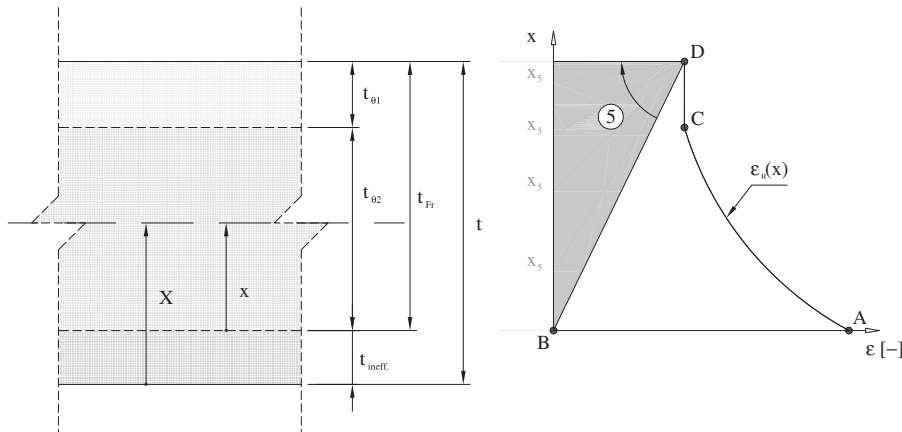


Fig. 12. Field 5.

similarly accounts for temperature dependence. Hence, the two-sided curves presented in Fig. 14 have been considered.

Such a constitutive relation has been used to derive the behaviour of the resistance, ultimate strain and elastic modulus as a function of temperature; the values are reported in the graphs shown in the Figs. 15 and 16.

The temperature profiles across the thickness were determined by preliminary thermal analyses (Fig. 5). Therefore, by then changing the reference system, through Eqs. (5) and (6), we were able to define relations (7)–(9). Lastly, considering a masonry material exhibiting, at 20 °C, a resistance of $f_{cu} = 5 \text{ N/mm}^2$, a crushing strain ε_u of 2.5‰, and an elasticity modulus equal to 2800 N/mm^2 , and applying the procedure formulated above, we have determined the M – N crushing domains (per unit length) for different values of total wall thickness and exposure time to the nominal fire (see Figs. 17–19).

2.2.2. Brick

By referring to the stress–strain curves reported in Appendix D of EN 1996-1-2 [24] (Fig. 20), it can be seen that the constitutive relation prescribed for brick is of the limited linear-elastic type.

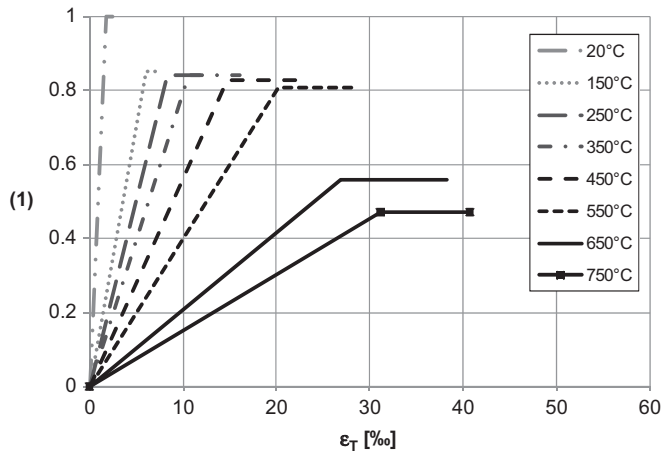


Fig. 14. Approximate constitutive relation for lightweight aggregate concrete (pumice) as a function of temperature; (1) denotes the ratio between the normal stress value at any given temperature T and the compression strength at 20 °C.

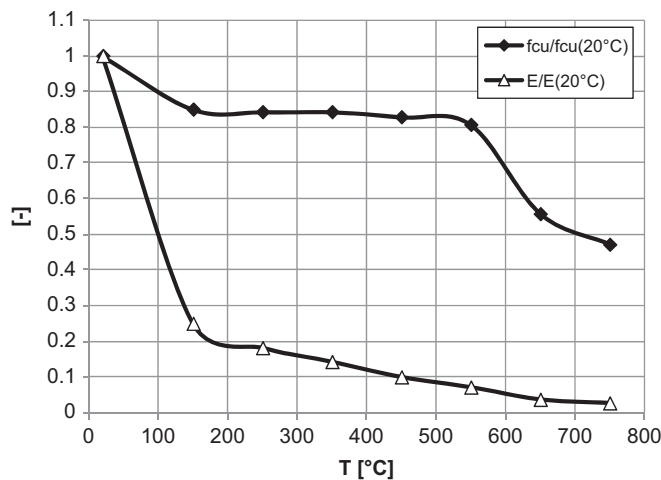


Fig. 15. Compression strength and elasticity modulus values, adimensionalized to ambient temperature values, as a function of temperature.

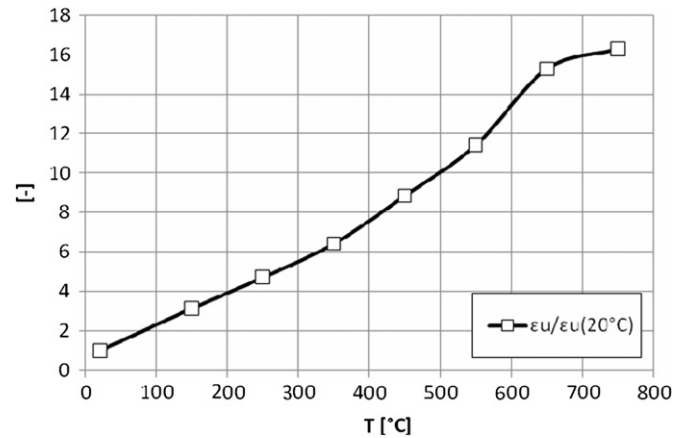


Fig. 16. Crushing strain values, adimensionalized to ambient temperature values, as a function of temperature.

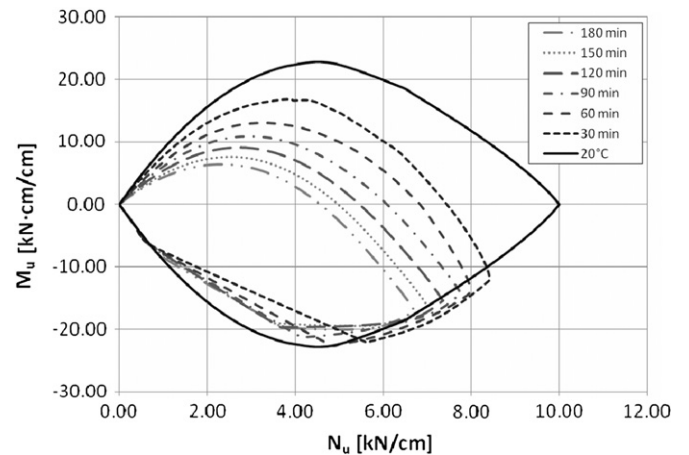


Fig. 17. Crushing domains of a 20 cm thick wall for various exposure times to nominal fire.

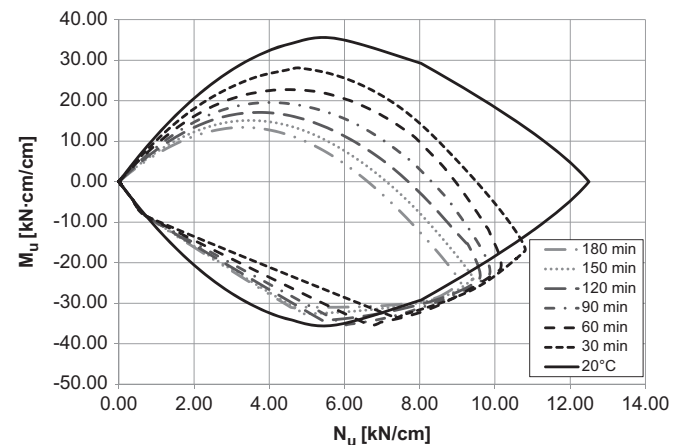


Fig. 18. Crushing domains of a 25 cm thick wall for various exposure times to nominal fire.

This constitutive law is used to deduce the resistance, ultimate strain and elastic modulus as a function of temperature; the values are reported in the graphs shown in Fig. 21.

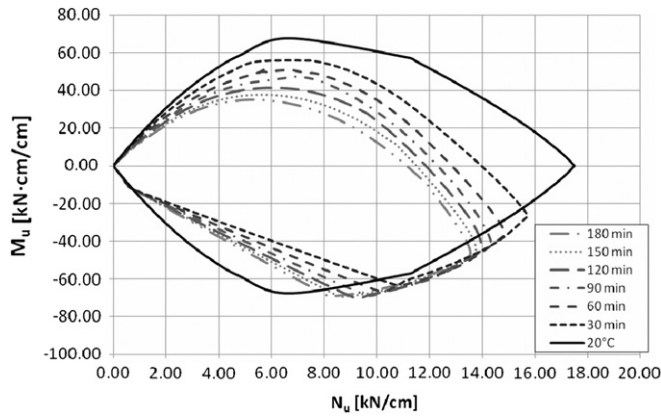


Fig. 19. Crushing domains of a 35 cm thick wall for various exposure times to nominal fire.

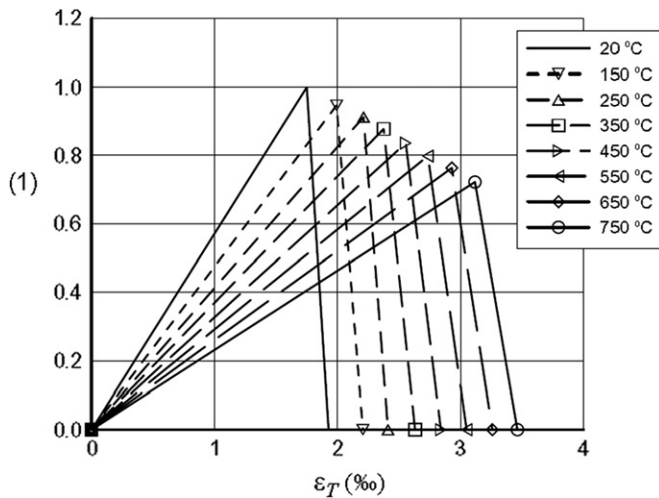


Fig. 20. Constitutive relation of brick as a function of temperature, as per Appendix D of EN 1996-1-2 [24]; (1) denotes the ratio between the normal stress value at any given temperature T and the compression strength at 20 °C.

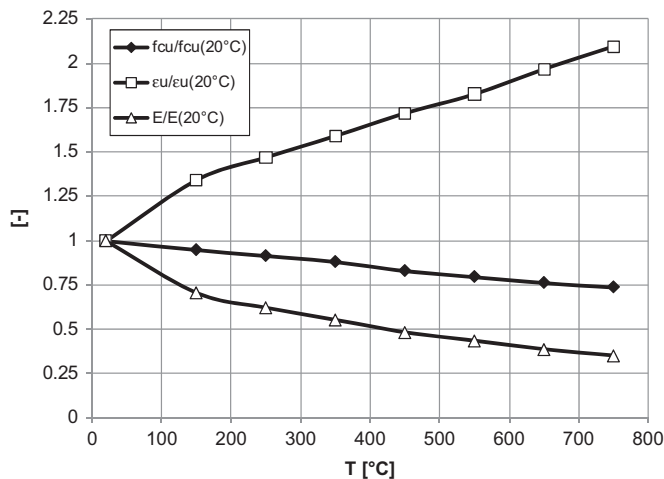


Fig. 21. Elasticity modulus, compression strength and crushing strain values, adimensionalized to the ambient temperature values, as a function of temperature.

Considering a masonry material having, at 20 °C, a resistance of $f_{cu} = 20 \text{ N/mm}^2$, a crushing strain ϵ_u of 1.49‰, and an elasticity modulus equal to $13,423 \text{ N/mm}^2$, and applying the procedure

presented above, we determine the $M-N$ crushing domains (per unit length) for different values of total wall thickness and exposure time to the nominal fire (Figs. 22–24).

2.2.3. Aerated autoclaved concrete (AAC)

A tested masonry panel of dimensions $11.5 \times 100 \times 300 \text{ cm}^3$ has been examined. A comparison of the previous calculus

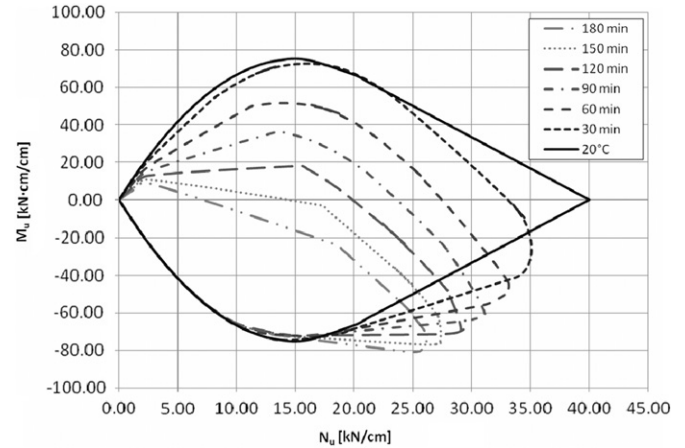


Fig. 22. Crushing domains of a 20 cm thick wall for various exposure times to nominal fire.

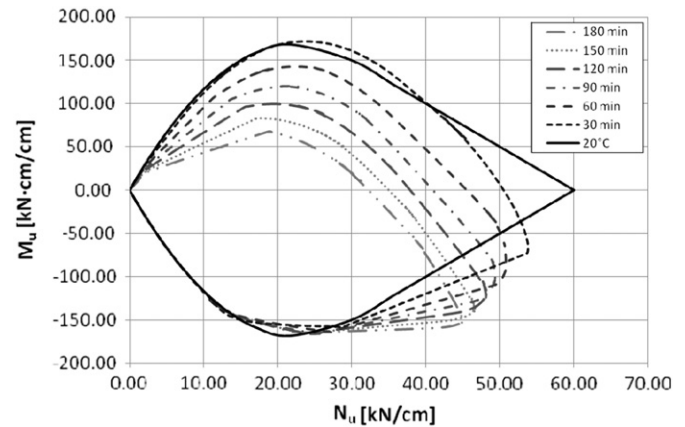


Fig. 23. Crushing domains of a 30 cm thick wall for various exposure times to nominal fire.

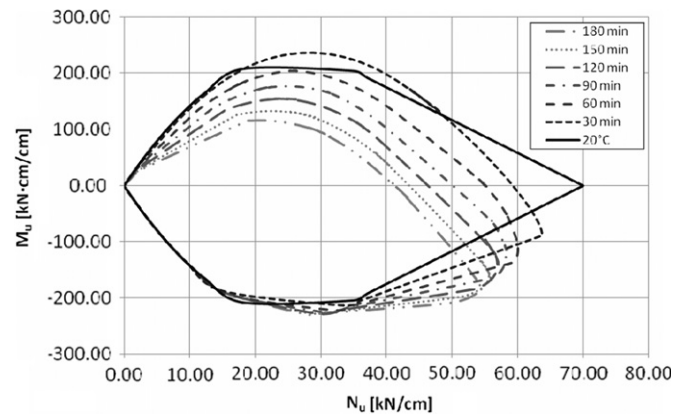
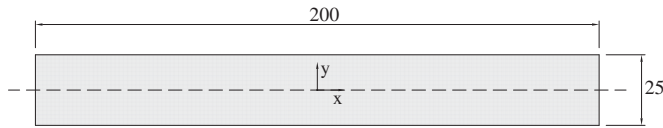


Fig. 24. Crushing domains of a 35 cm thick wall for various exposure times to nominal fire.

Table 2

Comparison of the proposed calculus procedure against experimental results by Hahn et al. [10].

Test	Utilisation factor α	Exposure time (min)	Axial force by Hahn et al. [10](kN/m)	Axial force calculated (kN/m)	Estimated error (%)
(1) $f_{td}(20\text{ }^{\circ}\text{C})=4.01\text{ MPa}$	1.0	57	230	265	−15.2
(2) $f_{td}(20\text{ }^{\circ}\text{C})=4.01\text{ MPa}$	0.6	64	138	160	−15.9
(3) $f_{td}(20\text{ }^{\circ}\text{C})=4.61\text{ MPa}$	1.0	48	414	317	23.4
(4) $f_{td}(20\text{ }^{\circ}\text{C})=4.61\text{ MPa}$	0.6	57	253	183	27.7

**Fig. 25.** Cross section of the considered building wall (heights in cm).**Table 3**

Cold mechanical and thermal properties.

Mechanical and thermal properties	Value
Density ρ (kg/m ³)	800
Conductivity λ_a (W/m $^{\circ}\text{C}$)	0.21
Specific heat c_a (J/kg $^{\circ}\text{C}$)	1170
Ultimate stress f_u (da N/cm ²)	50
Ultimate deformation ϵ_u (%)	2.5
Elasticity modulus E (da N/cm ²)	28,000

procedure against simple compression tests by Hahn et al. [10] are shown in Table 2.

3. Example application of the model to a load-bearing wall of a hospital

Hospitals are subject to special regulations, amongst which is the Italian Ministerial Decree of September 18, 2002, which prescribes an **R120** resistance class for such structures.

Let us consider a masonry wall made of lightweight aggregate concrete blocks (Fig. 25), with the cold mechanical and thermal properties reported in Table 3.

The per-unit-length values considered for axial force and out-of-plane bending moment acting on any given transverse section have been drawn from the analysis results and are reported in Table 4.

From the combination for exceptional actions, according to EN 1990 [21], we obtain

$$N_{Ed} = 3 + 1 + (0.6 \times 2) = 5.20 \text{ kN/cm} \quad (42)$$

$$M_{x,Ed} = 4 + 2 + (0.6 \times 4) = 8.40 \text{ kN} \quad (43)$$

At this point, referring to Fig. 18 (crushing domains of a 25 cm thick wall for various exposure times to nominal fire), we select the curve for 120 min and check that it contains the point with coordinates (5.20 kN/cm; 8.40 kN), as shown in Fig. 26.

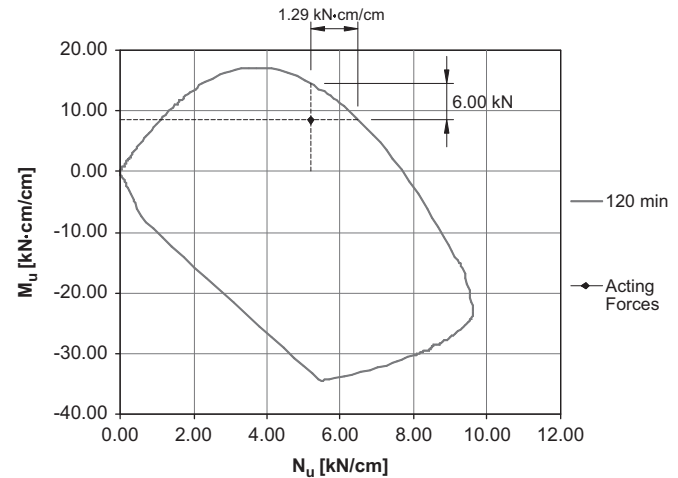
It can be seen that the assessment is satisfied and that the residual available moment, $M_{x,Res}$, equals 6.00 kN cm/cm. Hence, under the hypothesis of a 400 cm high wall, free to rotate at its extremities, the maximum horizontal pressure q_x that may be exerted by the gases produced by the fire is given by

$$q_x = \frac{8M_{x,Res}l}{lh^2} = \frac{8 \times 600 \times 200}{200 \times 400^2} = 0.03 \text{ da N/cm}^2. \quad (44)$$

Table 4

Characteristic values of axial force and out-of-plane bending moment.

Description	Characteristic axial force N_k (kN/cm)	Characteristic bending moment M_{xk} (kN cm/cm)
Structural element self-weight G_1	3.00	4.00
Non-structural element self-weight G_2	1.00	2.00
Variable overload (Cat. C) Q_k	2.00	4.00

**Fig. 26.** Assessment of fire resistance by means of domain $M-N$.

4. Conclusions

Observing the different domains for the lightweight aggregate concrete units and the brick masonry walls, it can be seen that increasing the exposure time causes the curves delimiting the borders to intersect each other, signifying that the walls in question do indeed improve their performance at higher temperatures. However, such improvement is in part fictitious, in that the resisting moment associated to each limit field has been calculated via the rotational equilibrium of the original section around the barycentric axis, and the stress characteristics at ambient temperature are also determined with respect to this same original section. In fact, given equal axial force, as the exposure time progressively increases, the resistant part of the section becomes ever smaller, with a consequent increase in the eccentricity.

Moreover, as we have taken the “maximum strain” as the yield criterion, any increase in material deformability with increasing temperature produces a real performance improvement of the generic section in terms of resistant axial force and bending moment.

The choice of referring to a barycentric axis of the original section for calculation of the bending moments was dictated by our aim of simplifying the verification procedures to be carried out by an eventual user, who would thereby be free to insert into the graphs the values of the stress characteristics calculated via the usual methods of cold analysis.

The computational method presented in this paper represents not only an extension to the procedures introduced in Eurocode 6 part 1–2, but also an alternative to them, in that it may be adapted to any type of masonry whose stress–strain curves and thermal properties are known as a function of temperature.

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