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Behaviour of Axially Loaded Concrete Members During and Following Fire Events

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Abstract

Effect of Fire on concrete structures is one of the research topics that have received noticeable attention from researchers in the last four decades. Concrete compressive strength, modulus of elasticity, and tensile strength experience significant deterioration during fire events. Following being exposed to fire, these properties were found to regain their original values with time. The same phenomenon was observed for reinforcing steel rebars. This recovery in material properties is expected to affect the overall behaviour of fire-damaged reinforced concrete elements. This paper reviews the available models proposed by different researchers to predict concrete mechanical properties during and after fire exposure. The investigated properties are those affecting the axial capacity of a compression member, concrete compressive strength, initial modulus of elasticity, and thermal induced strains. Steel tensile strength was also studied to account for its contribution to the capacity of axially loaded reinforced concrete sections. Based on the proposed models, an analytical study was conducted to assess the overall axial behaviour of a specific concrete section during and after experiencing an ASTM-E119 fire scenario for up to one hour.

Keywords: Concrete; fire; mechanical properties; strength regain; stress-strain relationship; axial capacity.

Introduction

Concrete structures are generally considered as an effective design solution for fire hazards. This is supported by the fact that concrete material does not experience a large reduction in its strength for temperatures less than 300 °C[1]. Fire affects concrete structures by generating a heat flow that initiates at the exposed surfaces and produces high temperatures and pore pressure gradients within the concrete mass. As a result, concrete properties, including compressive strength, tensile strength, elastic modulus, and effectiveness of the confinement provided by the transverse reinforcement deteriorate[2-4]. The level of deterioration is influenced by aggregate type, temperature level, heating rate, applied loading, and external sealing[5, 6]. After extinguishing a fire, concrete properties improve with time toward their original values[7-9]. Fire temperatures also affect concrete strains. It increases the strains required for equilibrium by shifting the value of the strain corresponding to the peak stress[3]. It also introduces new strains; transient and thermal[3, 4].

Fire-damaged buildings are currently being assessed by following a common process. It starts with site investigation to judge on the severity of damage by visual inspection and non-destructive tests such as rebound and ultrasonic tests. Based on results of this investigation, the number and locations of needed cylindrical cores are defined. The results of compression tests on these cores are used to judge on the safety of the reinforced concrete elements. However, there are no design guidelines to help designers predict the change in the capacity with elapsed time after fire exposure. This paper reviews the available work on concrete properties during and after exposure to elevated temperatures. The reported models address mechanical properties of un-sealed normal-weight concrete with siliceous, carbonate, or lightweight aggregates. Natural cooling by air is assumed.

Concrete compressive strength

Concrete compressive strength at elevated temperatures, $(f'_{cT})_{t=0}$, can be evaluated experimentally by heating unloaded or loaded concrete specimens and testing the hot concrete in compression[10]. Hertz model, given by Eq. (1), was recommended to predict $(f'_{cT})_{t=0}$ for different aggregate types[1]. The time t is measured from the day concrete was exposed to fire. The values of $(f'_{cT})_{t=0}$ calculated using Eq. (1) should be increased by 25% to account for a preloading stress level ($\lambda_L = f_c / f'_c$) of 0.25 to 0.30 [1].

$$(f'_{cT})_{t=0} = \frac{f'_c}{1 + \frac{T}{T_1} + \left(\frac{T}{T_2}\right)^2 + \left(\frac{T}{T_8}\right)^8 + \left(\frac{T}{T_{64}}\right)^{64}} \qquad (1)$$

Where T is the elevated temperature in degree Celsius, f'_c is the concrete compressive strength at ambient temperature, and

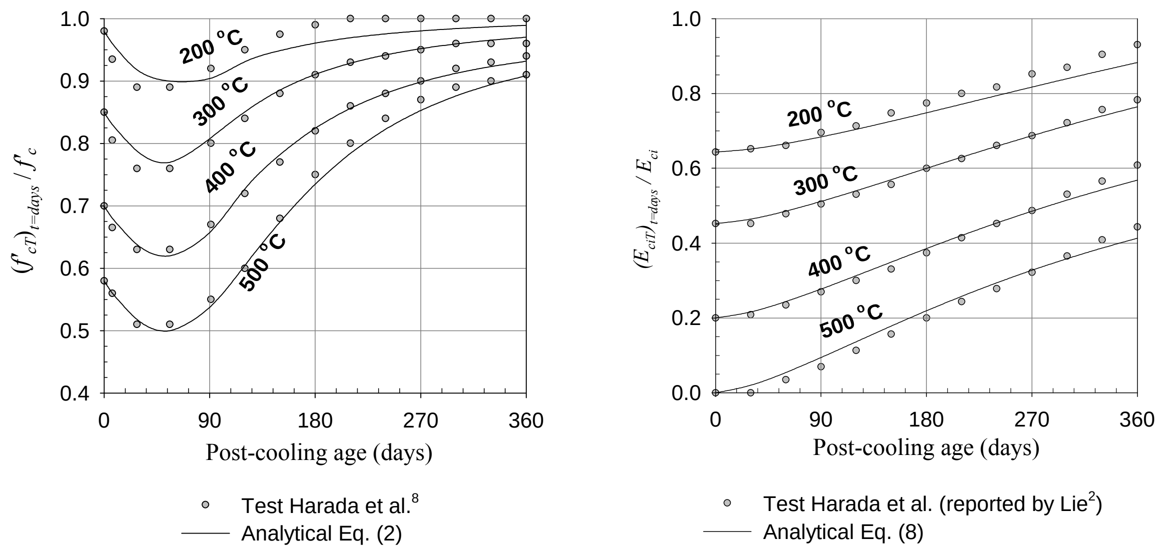
$T_1 = 15{,}000$, $T_2 = 800$, $T_8 = 570$, $T_{64} = 100{,}000$ for Siliceous aggregates;
$T_1 = 100{,}000$, $T_2 = 1100$, $T_8 = 800$, $T_{64} = 940$ for lightweight aggregates;
$T_1 = 100{,}000$, $T_2 = 1080$, $T_8 = 690$, $T_{64} = 1000$ for carbonate aggregates.

The residual compressive strength $(f'_{cT})_t$ is greatly affected by the type of the cooling process. Rapid cooling, such as quenching in water, result in filling the micro-cracks formed during the heating stage by new hydration products and thus $(f'_{cT})_t$ increases upon cooling[1]. On the other hand, slow cooling results in widening the micro-cracks[1] due to the relative hydration expansion between the moist outer layer and the dry inner core[6] and thus $(f'_{cT})_t$ decreases upon cooling. With time, the induced cracks during the heating and cooling stages start to be refilled with the new hydration products[7]. Harada et al.[8] experimental work is considered the most appropriate to simulate the behaviour of fire damaged concrete in reality. Harada et al.[8] reported $(f'_{cT})_t$ values at temperatures up to 500 °C and for one year, as shown in **Fig. 1a**. Based on these test results, Eq. (2) was developed to provide estimates for $(f'_{cT})_t$ following a fire event. Values of $(f'_{cT})_t$ estimated using Eq. (2) are shown in Fig. 1a.

$$(f'_{cT})_{t=days} = (f'_{cT})_{t=0} + \left(28\times10^{-6}\, t^2 - 3\times10^{-3}\, t\right)(f'_c) \qquad \le f'_c \qquad t \le 90 \text{ days} \tag{2.a}$$

$$(f'_{cT})_{t=days} = 0.74\,(f'_{cT})_{t=0} + 0.01\times\left(\frac{a.t^b}{c^b + t^b}\right)(f'_c) \qquad \le f'_c \qquad t > 90 \text{ days} \tag{2.b}$$

Where $a = 0.09\ T + 10.1$, $b = 0.0028\ T + 1.025$, $c = 0.336\ T - 3.5$, and T is in °C.



(a) Compressive strength (b) Modulus of Elasticity

Fig. 1 - Recovery of fire-damaged concrete properties with time.

Concrete strain

A review of the numerical models to predict concrete strains components during the heating stage is given by Youssef and Moftah[3]. Total concrete strain at elevated temperatures (ε_{tot}) is expressed as the summation of three terms *Instantaneous stress related strain* (ε_{fT}), *Unrestrained thermal strain* (ε_{th}), and *Transient creep strain* (ε_{tr}).

1. *Instantaneous stress related strain* (ε_{fT}): Its value at the peak stress $(\varepsilon_{oT})_{t=0}$ defines the stress-strain relationship during the heating stage. While elevated temperatures result in an increase in the value of $(\varepsilon_{oT})_{t=0}$, preloading concrete during heating reduces the amount of this increase. The model proposed by Terro[4], Eq. (3), was found to have good accuracy for estimating $(\varepsilon_{oT})_{t=0}$

$$(\varepsilon_{oT})_{t=0} = (50\lambda_L^2 - 15\lambda_L + 1)\,\varepsilon_{o1} + 20(\lambda_L - 5\lambda_L^2)\,\varepsilon_{o2} + 5(10\lambda_L^2 - \lambda_L)\,\varepsilon_{o3} \quad (3)$$

where
$$\varepsilon_{o1} = 2.05\times10^{-3} + 3.08\times10^{-6}T + 6.17\times10^{-9}T^2 + 6.58\times10^{-12}T^3$$
$$\varepsilon_{o2} = 2.03\times10^{-3} + 1.27\times10^{-6}T + 2.17\times10^{-9}T^2 + 1.64\times10^{-12}T^3$$
$$\varepsilon_{o3} = 0.002$$

2. *Unrestrained thermal strain* (ε_{th}): it is the free thermal strain resulting from fire temperature. It develops mainly due to the inter-particle cracking and thus its value is sensitive to the type of aggregates[11]. ε_{th} can be well predicted using the Eurocode model[2].

for concrete with siliceous aggregates:
$$\varepsilon_{th} = -1.8\times10^{-4} + 9\times10^{-6}\,(T-20) + 1.4\times10^{-11}\,(T-20)^3 \leq 12\times10^{-3} \quad (4.a)$$
for concrete with carbonate aggregates:
$$\varepsilon_{th} = -1.2\times10^{-4} + 6\times10^{-6}\,(T-20) + 2.3\times10^{-11}\,(T-20)^3 \leq 14\times10^{-3} \quad (4.b)$$
for concrete with lightweight aggregates:
$$\varepsilon_{th} = -8\times10^{-6}\,T \quad (4.c)$$

3. *Transient creep strain* (ε_{tr}) *or Load Induced Thermal Strain* (LITS): This strain is induced during the first heating cycle of loaded concrete and is considered the largest component of the total strain. It expresses the ability of the concrete to relax due to external compressive stresses[1, 11]. ε_{tr} can be estimated using Terro's model[4]

$$\varepsilon_{tr} = \varepsilon_{0.3}\times\left(0.032 + 3.226\frac{f_c}{f'_c}\right)\frac{V_a}{0.65} \quad (5)$$

Where $\mathrm{V_a}$ is the volume fraction of aggregates and f_c / f'_c (stress level) ≤ 0.7

$$\varepsilon_{0.3} = 43.87\times10^{-6} - 2.73\times10^{-8}\,T - 6.35\times10^{-8}\,T^2 + 2.19\times10^{-10}\,T^3 - 2.77\times10^{-13}\,T^4$$

Thermal strains developed during the heating stage are irrecoverable and become residual strains during the cooling stage. In addition, it was found experimentally that the effect of external loads becomes more pronounced during the cooling process as they cause significant contraction[12, 13]. A parametric study on residual strains during the cooling process was conducted by Khoury et al.[14].

Initial modulus of elasticity

Concrete initial modulus of elasticity at ambient temperature, E_{ci}, is defined as the slope of the compressive stress-strain relationship at zero strain. Its value during heating $(E_{ciT})_{t=0}$ for a preloaded concrete can be calculated using the model proposed by Anderberg and Thelandersson and reported by Youssef and Moftah[3], Eq. (6).

$$(E_{ciT})_{t=0} = \frac{2\times(f'_{cT})_{t=0}}{(\varepsilon_{oT})_{t=0}} \tag{6}$$

As the concrete starts regaining its original compressive strength after fire exposure, the initial modulus of elasticity was found to start recovering its original value. **Fig. 1b** shows the natural recovery of the initial modulus of elasticity $(E_{ciT})_t$ with time for siliceous concrete as experimentally evaluated by Harada et al.[8] and reported by Lie[1]. Due to the lack of analytical models, Eq. (7) was developed to estimate the $(E_{ciT})_t$. The predictions of this equation for Harada et al.[8] tests are shown in Fig. 1b.

$$(E_{ciT})_t = (E_{ciT})_{t=0} + E_{ci} \times \frac{0.8 \times t^{1.5}}{\left(\frac{3.2\times10^6}{T}\right)+t^{1.5}} \qquad \leq E_{ci} \tag{7}$$

Evaluating the value of $(E_{ciT})_{t=days}$ allows estimating the strain value at the maximum stress $(\varepsilon_{oT})_{t=days}$.

$$(\varepsilon_{oT})_t = \frac{2\times(f'_{cT})_t}{(E_{ciT})_t} \tag{8}$$

Yield strength of reinforcing bars

Fire temperature reduces the yield strength of reinforcing bars and eliminates the yielding plateau observed in tensile tests of mild steel specimens. Due to large strains exhibited at elevated temperatures, yield stress (f_{yT}) is usually evaluated using the 1% or 2% proof stress rather than the conventional ambient value of 0.2%. Lie's model[2] can be used to predict f_{yT}.

$$f_{yT} = \left[1+\frac{T}{900\times\ln(T/1750)}\right]\times f_y \qquad 0 < T \leq 600\,°\text{C} \tag{9.a}$$

$$= \left[\frac{340 - 0.34\times T}{T-240}\right]\times f_y \qquad 600 < T \leq 1000\,°\text{C} \tag{9.b}$$

Original yield and tensile strengths were found to be recovered on cooling after fire exposure. This phenomenon was related to the formation of some compounds during the cooling process[15]. For typical hot-rolled steel bars, yield strength is completely recovered up to temperatures of 600 ^{o}C [15, 16]. For higher temperatures, a linear decay can be assumed until reaching 70% of the original yield strength value at 900 ^{o}C [15]. Typical cold-worked steel bars were found to have some variation in yield strength recovery[16]. However, a linear variation can be assumed starting from 100% at the ambient temperature and ending at 60% after cooling from 800 ^{o}C [16].

Compressive stress-strain relationship

Reinforced concrete sections are usually analysed using stress-induced strain relationships. It was found that the general form the stress-strain curve of concrete at ambient temperature is maintained at elevated temperatures[4]. Scott et al.'s model[17] can be modified at elevated temperatures[2] by replacing f'_c and ε_o (strain at maximum stress at ambient temperature) with the temperature dependent terms $(f'_{cT})_{t=0}$ and $(\varepsilon_{oT})_{t=0}$ (strain at maximum stress at elevated temperature). Youssef and Moftah[2] proposed to account for the transient strain ε_{tr} by shifting the peak strength point by its value. Consequently, the total strain will be the summation of the load induced and thermal strains components.

$$(f_{cT})_{t=0} = (f'_{cT})_{t=0}\left[2\times\left(\frac{\varepsilon_{cT}}{(\varepsilon_{oT})_{t=0}}\right)-\left(\frac{\varepsilon_{cT}}{(\varepsilon_{oT})_{t=0}}\right)^2\right] \qquad \varepsilon_{cT} \le \varepsilon_{oT} \qquad (10)$$

$$(f_{cT})_{t=0} = (f'_{cT})_{t=0}\left[1 - Z\left(\varepsilon_{cT} - (\varepsilon_{oT})_{t=0}\right)\right] \qquad \ge 0.2\times f'_{cT} \qquad \varepsilon_{oT} \ge \varepsilon_{oT}$$

$$Where, Z = \frac{0.5}{\varepsilon_{50uT} - (\varepsilon_{oT})_{t=0}} \text{ and } \varepsilon_{50uT} = \frac{3+0.29f'_c}{145f'_c - 1000}$$

The same concrete model can be extended to address the damaged concrete in the same way of the heating stage. The authors suggest replacing f'_c and ε_o with the temperature and time dependent terms $(f'_{cT})_t$ and $(\varepsilon_{oT})_t$ (strain at maximum stress at time t from the end of fire exposure).

Case study

The axial compressive capacity of reinforced concrete members during and after fire events is predicted in the following case study. A 305 mm by 305 mm concrete column with 4 #25 steel bars, **Fig. 2a**, is analyzed during and after exposure to a specific fire scenario. The column material is assumed to be siliceous concrete with moisture content of 3% and aggregate volume fraction (V_a) of 0.65. In order to simplify the analytical calculations, the column top and bottom ends are assumed to be prevented from expansion during fire exposure. An initial axial load of 1067 kN, about 26% of the ambient axial capacity, was applied prior to fire to represent the dead and portion of live loads that are existing during fire events.

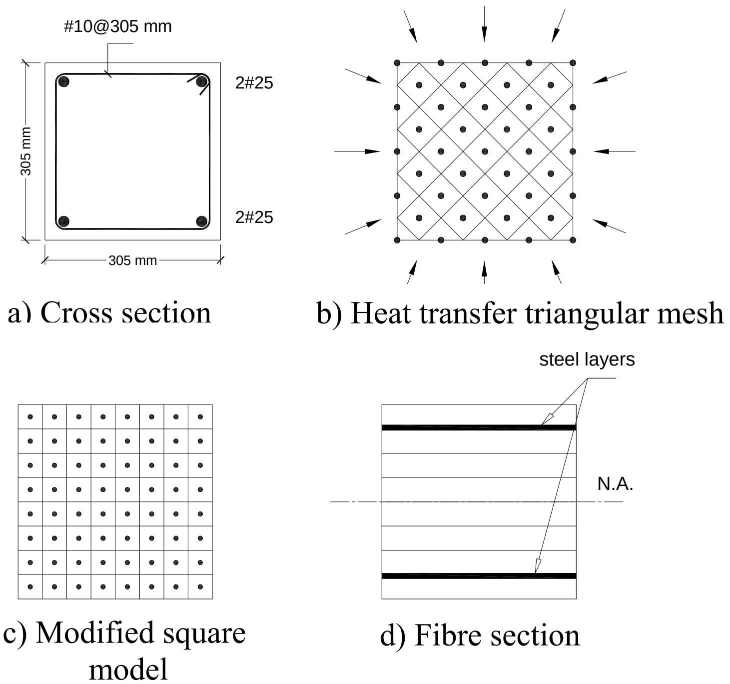
Heat transfer model

The concrete section is divided into a number of triangular elements, **Fig. 2b**, where the temperature at the centre of each element represents the temperature of the entire element. The fire is assumed to follow the standard fire described in ASTM-E119 and approximately described by the following expression:

$$T_f^j = 20 + 750\,(1 - exp(-3.79553\sqrt{t_f})) + 170.41\sqrt{t_f} \qquad (11)$$

Where T_f^j is the fire temperature at time step j, t_f is the fire duration. t_f is divided into a number j time steps Δt (i.e. $t_f = j \,.\, \Delta t$)

The temperature gradient through the concrete cross section can be estimated at each time step using the Finite Difference Method (FDM) [2]. The effect of steel rebars in the heat transfer model was neglected and the temperature distribution was predicted using only the concrete thermal properties.



a) Cross section

b) Heat transfer triangular mesh

c) Modified square model

d) Fibre section

Fig. 2 – Analysis of reinforced concrete section

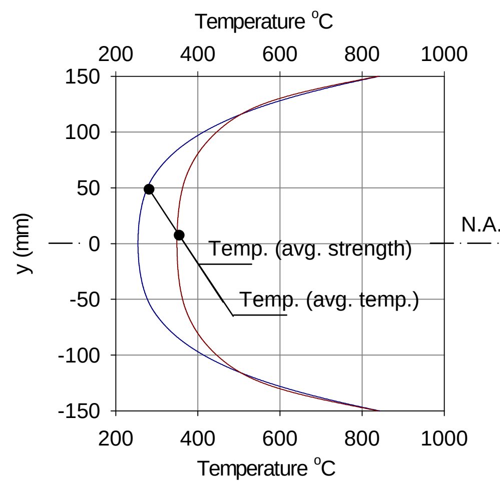


Fig. 3 – Temperature distribution

Sectional analysis

The concrete sectional behaviour can be studied using a fibre model. The triangular mesh is modified to a square mesh as shown in **Fig. 2c**. Square elements were grouped into a number of layers as shown in **Fig. 2d**. As each of the square elements is having a different temperature, an equivalent temperature for each layer was needed. **Fig. 3** shows two temperature distributions for the layers. While the first distribution assumes that the layer equivalent temperature is equal to the average temperature of the square elements composing it, the second assumes that the layer equivalent temperature results in a concrete strength equivalent to the average concrete strength of the square elements. It is clear that the two methods can produce significantly different results. To correctly evaluate the column axial capacity, the average concrete strength method is adopted in this paper.

Knowing the elevated temperature experienced by each layer, the concrete and steel mechanical properties can be estimated using the models proposed earlier in this paper. Utilizing the uniaxial stress-strain relationship for each fibre and taking into account equilibrium and kinematics, the mechanical behaviour of the section is analyzed. The effect of concrete confinement provided by transverse reinforcement was neglected.

Results and discussion

Fig. 4 shows the load-axial compressive strain relationship for the studied column at different elapsed time during the heating stage. A good agreement of the predicted axial compressive capacity and the work done by Lie et al. [18], **Fig. 5,** is found.

The proposed strength recovery models for concrete and steel are implemented in the analysis to investigate the residual capacity of the column at different ages after fire exposure. As mentioned before, the concrete induced thermal strains during the heating period are considered to be residual strains and irrecoverable after cooling down.

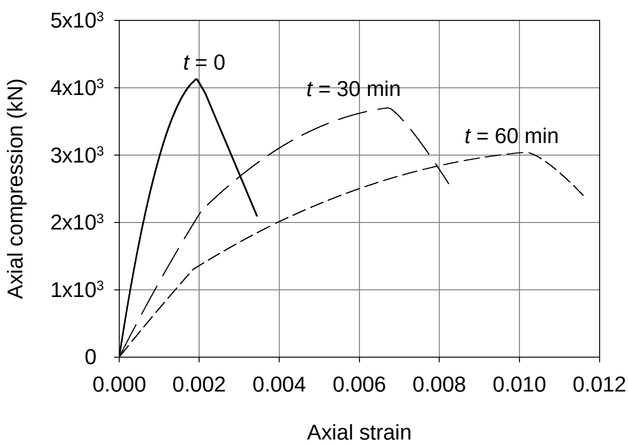


Fig. 4 – Effect of fire exposure on axial behaviour

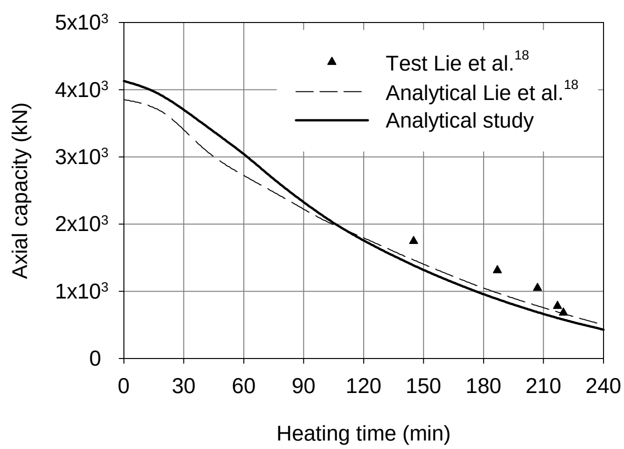


Fig. 5 – Validation of analytical program

Fig. 6a presents the predicted post-cooling load-axial strain relationships for the studied column after being heated for 30 minutes. During the heating stage, the axial capacity becomes 90% of the ambient temperature capacity. However, the minimum capacity, 86% of the original capacity, is found to occur after 2 months from the end of fire exposure. The axial compressive capacity reaches 97% of its original overall strength after 1 year. **Fig. 6b** presents the predicted post-cooling load-axial strain relationships for the studied column after being heated for 60 minutes. The axial capacity reduced by 26%. 94% of the ambient capacity is expected to be recovered after 1 year of fire exposure.

Conclusion

Assessment of fire-damaged buildings after experiencing fire events is a mandatory process to evaluate the safety of the buildings. Some buildings may be reoccupied after a certain period of time while others may be demolished and replaced with new structures. This paper focuses on the axial behaviour of reinforced concrete members during and after fire events. Available models to predict concrete and steel mechanical properties during and following fire exposure are presented. These models are utilized in an analytical study where a column cross-section is analyzed during and after an ASTM-E119 fire scenario. The studied column is found to regain 97% and 94% after 1 year of being exposed to a 30 and 60 minutes fire. Further work is needed to validate the material properties recovery models considering the aggregate type and environmental factors affecting the regain process.

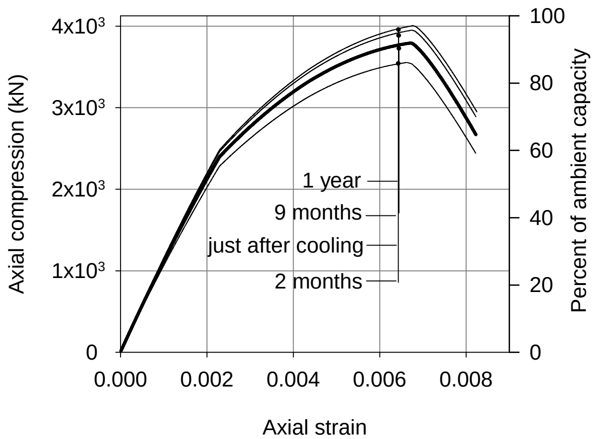


Fig. 6 – Axial behaviour after 30 min fire exposure

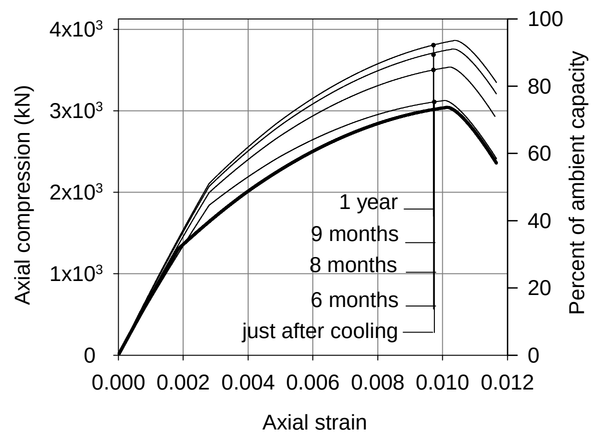


Fig. 7 – Axial behaviour after 60 min fire exposure

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