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Reliability-based evaluation of the inherent safety presumptions in common fire safety design

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ABSTRACT

The structural safety of a simply supported one-way concrete slab exposed to the ISO 834 standard fire curve is investigated. The proposed methodology allows for an objective comparison of the structural performance of different concrete slab design alternatives and to derive indirectly a lower bound of the fire ignition frequencies that can be associated to the Eurocode guidelines. A comparison is made with empirical fire ignition frequencies found in literature and a detailed study is presented of the significance of the concrete cover and its uncertainty on the safety level and fire resistance time. Results indicate that the safety level during fire and the fire resistance time are both highly sensitive to the ratio of the variable to the permanent load and that the accepted design solutions given in EN 1992-1-2 are rather conservative.

1. Introduction

Requirements with respect to fire safety are changing from prescriptive toward performance based design codes and regulations. Amongst others the United Kingdom [1], New Zealand [2,3], Japan [4], and the German state of Nordrhein-Westfalen [5] have all adopted legal requirements allowing for the use of equivalent solutions with respect to fire safety measures. However, little progress has been made with respect to evaluation methods to compare design alternatives. In order to compare structural alternatives with respect to fire, the safety level inherent to the design format of EN 1992-1-2 [6] for concrete elements during fire should be determined as a function of the geometry and load characteristics.

In this research the safety level of simply supported concrete slabs subjected to bending and exposed to the ISO 834 standard fire curve from the bottom side only is assessed using a probabilistic calculation tool. This probabilistic assessment allows to examine in detail the reliability level implicitly associated to the design format of EN 1992-1-2 and to study the influence of basic variables as e.g. the concrete cover.

Many researchers have made valuable contributions to the probabilistic assessment of structures during fire exposure. Sidibé et al. investigated the reliability of concrete columns exposed to the ISO 834 standard fire curve [7], while Wang et al. [8] and Eamon and Jensen [9,10] used an analytical formulation to assess the reliability of concrete beams. Recently, Guo et al. [11] presented a framework for assessing the reliability of structural elements, taking into account an uncertain fire load, and Van Coile et al. proposed an easy-to-use reliability-based framework for assessing the residual load bearing capacity after fire exposure [12]. Despite the increasing interest in this research topic, a clear evaluation of the reliability of the design format of EN 1992-1-2 with respect to the load bearing capacity has not yet been developed. Therefore, the current research aims to evaluate the implicit safety level which can be associated with the Eurocode design format for concrete elements exposed to fire.

2. Probabilistic characterization of the bending moment capacity during fire

During fire, a non-linear temperature distribution develops over the cross-section of the concrete element. As both concrete and reinforcing steel are affected by elevated temperatures, the mechanical properties during fire of a finite element of material depend on the local temperature, i.e. the location of the element within the cross-section and the time of fire-exposure. Consequently, calculating the bending moment capacity of a concrete slab requires a numerical tool which takes into account the non-linear distribution of the mechanical properties across the height and width of the cross-section.

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A simplified numerical model is developed to calculate the bending moment capacity $M_{R,fi,t}$ of concrete beams and slabs during fire, as described in [13,14]. The cross-section is divided in square elements with an area of 25 mm^2 and the temperature and mechanical properties are assumed to be uniform within every element. This temperature distribution within the concrete cross-section is calculated by the finite element program DIANA, taking into account the thermal conductivity, thermal capacity, emissivity and convection coefficients as prescribed by EN 1992-1-2 [6]. The basic concepts of the discretization of the cross-section are illustrated in Fig. 1.

The moment curvature diagram associated with the heated cross-section is calculated by imposing a curvature on the cross-section and iteratively calculating the strain of the central fiber for which the longitudinal restraining forces are in equilibrium with the sum of the compressive and tensile forces resulting from the square elements. In this paper, it is assumed that the slab is not longitudinally restrained and the tensile strength of the concrete is conservatively neglected. Consequently, for a given curvature, the moment is defined by the strain distribution for which the tensile forces of the reinforcement are in equilibrium with the compressive forces of the concrete.

For calculating the equilibrium of the longitudinal forces, the same assumptions as made by Kodur and Dwaikat [15] are considered:

(1) Plane sections remain plane (Navier–Bernouilli hypothesis).
(2) Bond slip between concrete and the reinforcement is neglected.
(3) Spalling is neglected.

Consequently, the local mechanical strain $\varepsilon_{\sigma,\underline{\theta}}$ is calculated by (1),

$$\varepsilon_{\sigma,\underline{\theta}} = \varepsilon_{tot,\underline{\theta}} - \varepsilon_{th,\theta} \tag{1}$$

with $\varepsilon_{th,\theta}$ the free thermal elongation dependent on the local temperature θ and $\varepsilon_{tot,\underline{\theta}}$ the linear deformation of the cross-section, resulting from the Navier–Bernouilli hypothesis, the curvature of the cross-section and the strain of the central fiber.

The local mechanical strain can be associated with a local stress by taking into account the mechanical properties associated with the local temperature. As the stress is assumed uniform in every square element of 25 mm^2, the local contribution of the concrete to the overall compressive force is determined by multiplying the local stress with 25 mm^2. For every reinforcement bar, the temperature, strain and stress of the centroid are applied to the entire reinforcement area. The model concepts with respect to the calculation of these local forces are illustrated in Fig. 2.

The mechanical properties and the stress–strain diagrams are implemented in accordance with EN 1992-1-2 [6]. This implies that the concrete material model includes a linear softening branch from the strain corresponding to peak stress up to the ultimate strain tabulated in [6]. The reinforcement stress–strain model follows a parabolic curve between the proportionality limit f_{sp} and the yield stress f_y, a yield plateau and a decreasing branch until the ultimate strain.

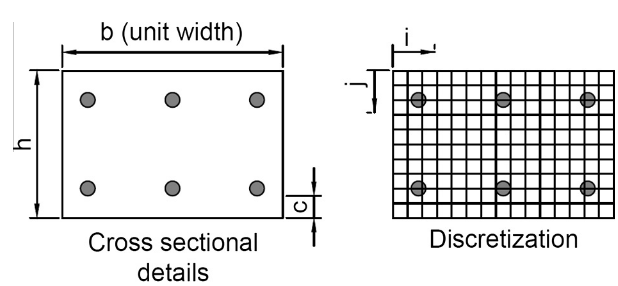

Fig. 1. Model concepts: cross-sectional details and discretization of the cross-section.

The reduction of the concrete compressive strength f_c and the steel yield stress f_y at elevated temperatures are evaluated in accordance with EN 1992-1-2 [6] by multiplying the strength parameters at 20 °C with a reduction factor $k_{fc(\theta)}$ and $k_{fy(\theta)}$ respectively, as defined by (2) and (3).

$$k_{f_c(\theta)} = \frac{f_c(\theta)}{f_c(20\,^\circ\mathrm{C})} \tag{2}$$

$$k_{f_y(\theta)} = \frac{f_y(\theta)}{f_y(20\,^\circ\mathrm{C})} \tag{3}$$

In literature many alternatives to this strict implementation of the Eurocode material models are proposed. In [16] the softening branch of the concrete material model is neglected and the material model for the reinforcement is simplified. In [17] the concrete stress–strain diagram at elevated temperatures was idealized by a bilinear model for both ordinary concrete and high-strength concrete, while the reinforcement stress–strain curves were implemented in accordance with EN 1992-1-2 [6]. These alternative material models can be very valuable for many practical purposes, but in this research the basic material models that are implemented are in full agreement with EN 1992-1-2 [6].

In order to assess the uncertainty of the bending moment capacity during fire, the uncertainties with respect to the mechanical and geometric basic variables are incorporated. The probabilistic models used to describe these uncertainties are chosen considering the literature study performed in [18]. For the concrete cover a Beta distribution is used, since this bounded distribution avoids unrealistically high or low values and can easily be modified to take into account any possible skewness in the distributions of values for the concrete cover. Furthermore, the Beta distribution has been recommended by the Joint Committee on Structural Safety as a suitable distribution for the concrete cover [19].

The uncertainty with respect to the reduction factors $k_{fc(\theta)}$ and $k_{fy(\theta)}$ is modeled by a Beta-distribution, with a mean value equal to the nominal value given in EN 1992-1-2 [6] and a coefficient of variation based on laboratory tests [20], i.e. for the compressive concrete strength the coefficient of variation of $k_{fc(\theta)}$ is 0 at 20 °C and is considered to be 0.045 at 700 °C. For the steel yield stress similar considerations yield a coefficient of variation of 0.052 for $k_{fy(\theta)}$ at 500 °C. Linear interpolation is used for intermediate values and the coefficient of variation is assumed to be constant for temperatures above 700 °C and 500 °C respectively, based on observations from previous test results executed at Ghent University. Alternative models for the uncertainty of $k_{fc(\theta)}$ and $k_{fy(\theta)}$ can be implemented easily. For example, the coefficient of variation can be assumed to continue increasing linearly beyond 700 °C and 500 °C. However, this alternative assumption does not influences the results strongly. In [7] a similar procedure was used to incorporate the uncertainty with respect to the reduction factors $k_{fc(\theta)}$ and $k_{fy(\theta)}$ for assessing the reliability of concrete columns during fire.

The probabilistic calculation of the bending moment capacity is carried out using crude Monte Carlo simulations [21] for a concrete slab with stochastic properties according to Table 1.

As legal requirements and the Eurocodes define the structural fire resistance R in relation to the ISO 834 standard fire curve, this standard fire curve should be considered when assessing structural fire safety in relation to normative design. For specific structures, the hydrocarbon standard fire curve is prescribed. If a probabilistic fire load were to be used, it would not be possible to relate the design with the legally required structural fire resistance. Furthermore, legal requirements do not allow to take into account