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J. Li, Pol D. Spanos, J.B. Chen & Y.B. Peng (Eds)

Probabilistic characterization of the performance of a composite slab panel during and after fire

R. K. Chaudhary[a], T. Gernay[b], A. Lucherini[a] and R. Van Coile[a]

[a]*Department of Structural Engineering and Building Materials, Ghent University, Belgium.*
[b]*Department of Civil and Systems Engineering, Johns Hopkins University, USA.*
Email: RanjitKumar.Chaudhary@UGent.be

ABSTRACT: Most of the structures exposed to a fire do not collapse and the decisions taken in the aftermath carry significant weight in terms of cost and resilience. Previous research has mostly looked at single members' response during fire exposure, at a deterministic level. While large uncertainties exist with respect to both the fire exposure and structural response, the probabilistic assessment of structural performance after fire has typically received little attention. This study focuses on the probabilistic evaluation of a structures' capacity during and after fire, and residual deflections. A composite slab panel is considered for the study. The probability of failure for the slab panel under natural fire exposure is estimated as 23.8%. After fire exposure, the residual capacity of the slab panel reduced to as low as 60% of the pre-fire capacity, and large residual displacements can be expected. These observations allow to determine the re-usability and repairability of the structure and ultimately estimate the life-cycle cost of structure.

KEYWORDS: Structural Fire Performance; Composite Slab Panel; Probabilistic Assessment; Residual Capacity.

1 INTRODUCTION

Structural systems are generally designed for a life span of several decades. During their design life, structures might be subjected to several hazards (accidental or man-made). Fire is one of the most severe hazards which structures might experience, and it can cause extensive loss of lives and resources. The current guidelines on structural fire design mainly deal with the structural integrity and stability, and post-fire performance and building reusability have often been overlooked. Yet, since most of the fires have a limited severity, fire-induced structural damages are often minor and may be repaired. It can also be expected that a small initial investment in the structural fire design may significantly improve the post-fire performance of the structure. Recently, there has been a shift in societal expectations towards a resilient structural design, to achieve a rapid and cost-effective recovery of the structure after an adverse event such as fire (Bocchini et al., 2013; DRDC, 2014). To enable a resilient structural fire design, it is important to determine the structural behavior during and after a fire event.

When exposed to fire, structural members typically experience a reduction in their strength and stiffness. In case of reinforced concrete (RC) structures, a severe fire exposure may lead to loss of concrete strength, spalling, buckling of reinforcing bars and large residual deformations (Schneider, 1990, The Concrete Society, 2008). However, moderate fires may not result in noticeable deformations. Even with limited visible deformations and damage, however, the permanent loss of structural capacity resulting from the thermal exposure may lead to an increased probability of structural failure if the structure continues to be used. Hence, in-depth evaluation of residual structural capacity has been recommended to give a more comprehensive idea of the damage state of a structure (Kodur and Agrawal, 2016).

The residual capacity of a fire-damaged structure can be numerically evaluated through thermo-mechanical analysis of the structure. The thermal analysis estimates the temperature distribution inside the cross-section of structural elements, while the mechanical analysis assesses the residual stresses, strains and deformations, as well as the residual capacity. Early studies (Hsu and Lin, 2008; Kodur et al. 2010) considered only the distribution of peak temperatures reached inside the cross-section to evaluate the residual capacity of structural members. However, the evaluation of residual capacity only based on this peak temperature and the corresponding residual material strength may be insufficient, since the residual stresses and strains are ignored. Recently, advanced numerical models such as finite element (FE) methods have been considered for the evaluation of residual capacity. For the FE estimation of residual capacity, thermo-mechanical analysis is carried out, where both the thermal and mechanical structural responses are taken into account. This numerical approach for the evaluation of residual capacity can be found for example in the studies carried out by Kodur and Agrawal (2016, 2021).

Based on the available literature, it can be observed that, to date, the residual capacity has mostly been investigated at member level, for instance isolated structural members such as beams, columns and slabs. However, the member response evaluated as a part of a structural system is different in comparison to the one evaluated for an isolated member (Gillie et al., 2001; Wald et al., 2006; Chaudhary et al., 2020). Studies at system level are needed to simulate the actual behavior of the overall structure.

Moreover, the existing literature mostly reports deterministic calculations for the residual capacity assessment. However, the behavior of structures under fire exposure is highly uncertain. The uncertainty in the assessment of structural behavior is notably related to the randomness of the fire event and the stochastic behavior of structural materials (e.g. steel and concrete). In order to incorporate the effect of these uncertainties in the structural fire design, probabilistic studies have been recommended (Qureshi et al., 2020; Van Coile et al., 2019; Gernay et al., 2019a). When these uncertainties are considered, probabilistic approaches can be applied to ensure a safe and reliable structural fire design, with a level of confidence which is beyond deterministic methods.

The current study focuses on the probabilistic evaluation of the performance of a structural system during and after fire. A composite slab panel is considered. The composite slab panel is supported by girders and beams and it is expected to undergo tensile membrane action when subjected to fire. The probabilistic evaluation is carried out by considering the uncertainties in both the fire load and thermo-mechanical model. The structural performance under fire is characterized by evaluating the probability density function for the structural capacity before fire, and for the residual structural capacity and residual deformations after fire. Considering uncertainties in the loads and damage thresholds for the feasibility of repair, fragility functions for structural failure during fire and reparability after fire are determined.

2 EVALUATION OF THE RESIDUAL CAPACITY OF FIRE-DAMAGED STRUCTURES

2.1 *Assessment framework*

The assessment of residual capacity is a necessary step to support the decision-making for re-usability of a structure after a fire event. A pre-assessment of the post-fire residual capacity can also be relevant in the design phase to design the structure with a view on post-fire performance. The current study involves the pre-assessment of residual capacity of structure by subjecting it to probable fire loads.

The methodology for the evaluation of residual capacity is presented in Figure 1. This approach is based on a study by Kodur and Agrawal (2016). According to this approach, the load-bearing capacity of the structural system at ambient temperature is initially evaluated by applying an incremental load until structural failure. This results in an assessment of the initial structural capacity (P_{amb}). P_{amb} is considered as a reference value to estimate the decrease in structural capacity after fire exposure.

The residual capacity after fire exposure is determined by first subjecting the FE structural model to the thermal effects caused by the fire. The structural response during fire is then evaluated. If the structure does not fail during fire (including the cooling phase), the residual deformation is determined after the structure has cooled back to ambient temperature. Subsequently, an incremental loading is applied until structural failure to determine the residual capacity after fire. This evaluation takes into account degraded material properties. The entire computation process (applying the mechanical loading relevant to the fire situation, applying the heating and cooling phases of the fire, cooling of the structure to ambient temperature, and finally increasing the mechanical loading until failure) is performed in a single transient FE analysis. The material models used in the analysis are temperature-dependent and they consider the irreversible effects during heating and cooling (i.e. loss of strength and stiffness, and permanent deformations). As a result of this transient analysis, the effects of fire in terms of residual stresses and strains in the structure and permanent degradation of the mechanical properties are automatically taken into account for the evaluation of the residual capacity.

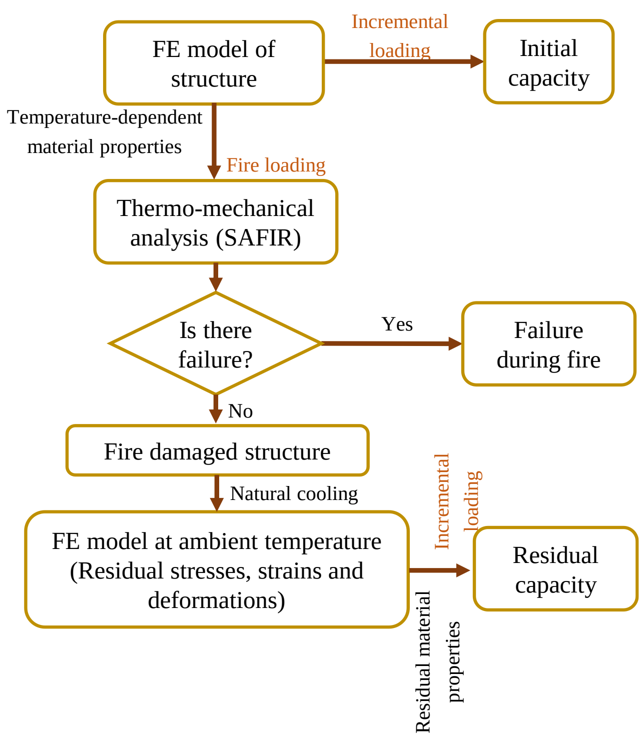

Figure 1: Methodology to evaluate the residual capacity of fire-damaged structures.

2.2 *Residual material properties*

The post-fire structural performance is determined taking into account the residual material properties of the structural materials. As regards to concrete in the heating phase, there is a marginal loss in strength for temperature up to 300°C. With an increase in temperature above this threshold, the concrete strength reduces gradually up to 500°C. Above this temperature, concrete loses its strength rapidly (Schneider, 1990; The Concrete Society 2008). In accordance with the EN 1994-1-2:2005, an additional decrease of 10% relative to the reduction at the peak temperature is considered for the concrete compressive strength while cooling down to ambient temperature. The residual tensile strength is considered equivalent to the tensile strength at the maximum temperature reached during the fire exposure.

The available studies related to the residual properties of steel after fire exposure are limited and results are quite scattered (Ni and Gernay, 2020). The steel yield strength is completely recovered for temperatures up to 450°C for cold-formed steel, and 600°C for hot-rolled steel (The Concrete Society, 2008). Even for higher temperatures, permanent loss in yield strength is marginal (Neves et al., 1996). In the current numerical study, the mechanical properties of steel are assumed to be fully reversible after fire exposure.

3 CASE STUDY: COMPOSITE SLAB PANEL AND NATURAL FIRE EXPOSURE

The composite slab panel considered here is the same as in Gernay et al. (2019b) in a probabilistic analysis of fire-exposed structures. The slab panel is a part of a larger structure designed according to ASCE 7-10 and AISC construction manual for steel. According to its configuration, the composite slab panel is expected to undergo tensile membrane action in fire conditions. The performance assessment of the composite slab panel during and after fire is carried out using the FE package SAFIR (Franssen and Gernay, 2017).

The composite slab panel consists of a reinforced concrete slab, two steel girders, two boundary beams and two central beams as shown in Figure 2. The composite slab panel is designed as a part of an office building. A uniformly distributed load of 5.4 kN/m^2 is considered as the characteristic load. The reinforced concrete slab is 93.5 mm thick and 9.15 m × 9.15 m in plan. The concrete slab is reinforced with a mesh of steel bars of area 503 mm^2/m at mid-height along both orthogonal directions. The cross-sections of the steel girders are W21×44, while W18×35 has been used for the boundary beams and central beams. The girders and boundary beams are protected to have a 2-hour fire resistance, while central beams remain unprotected. The fire resistance is achieved by applying a protective coating of Spray-Applied Fire Resistive Material (SFRM), 2.02 cm thick for the girders and 2.22 cm thick for the boundary beams. The thermal properties considered for the SFRM can be found in the study by Gernay et al. (2019b).

The concrete of the RC slab has a characteristic strength of 28 MPa, while the steel reinforcement has a characteristic yield strength of 416 MPa. The steel girders and beams have a yield strength of 345 MPa. The concrete is modeled using a concrete plasticity-damage model (Gernay et al., 2013), implemented within SAFIR as SILCOETC2DPR. The concrete strength retention factor at elevated temperature follows the probabilistic model as suggested by Qureshi et al. (2020). This model also explicitly considers the effect of transient creep strain, which is significant for modeling the concrete behavior during the cooling phase of fire exposure (Gernay, and Franssen, 2012). The reinforcing steel bars and steel members are modeled using the uniaxial material model 'STEC3PROBA'. As for the concrete, STEC3PROBA takes into account the probabilistic formulation for yield strength retention factor of steel at elevated temperature as specified in (Qureshi et al., 2020).

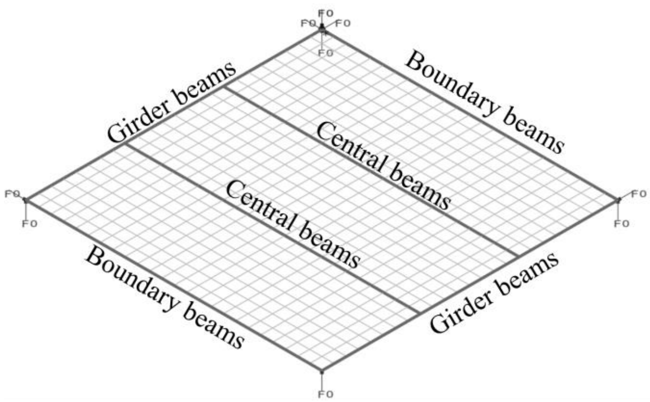

Figure 2: Composite slab panel (Gernay et al., 2019).

The other assumptions made in the current study can be found in Gernay et al. (2019b). As regards to the FE modeling of the slab panel, the concrete slab is modeled using shell elements, while the steel members are modeled using beam elements. The fire load on the slab panel is modeled based on the Eurocode parametric fire curve (EN 1991-1-2:2002). To estimate the parametric fire curve, a compartment of size 9.15 m × 9.15 m × 2.8 m with an opening factor of 0.0409 $m^{1/2}$ is considered. Considering the office building application, the characteristic fire load density for the compartment is 511 MJ/m^2 and the thermal inertia of the compartment linings is assessed as 938 $J/m^2s^{1/2}$ (walls and ceiling lined with gypsum boards).

4 STRUCTURAL FIRE PERFORMANCE

Considering the described structural parameters and fire load, Figure 3 shows the displacement time-history at the center of the composite slab panel during 4-hours of natural fire exposure. The slab panel experiences a maximum displacement of 500 mm and it remains stable for the entire simulation. However, after fire exposure, the structure exhibits permanent deflections and damage.

The slab panel is then cooled down for 24 hours (since the start of the fire exposure) at ambient conditions (20 °C). Successively, the post-fire assessment is carried out. Figure 4 displays the contour-plot of residual displacements after cooling. It can be observed that the slab panel has the highest displacements at the center. The maximum displacement is approximately 360 mm. The estimated residual displacements can be used to estimate the damage state and the repair cost for re-usability of the structure (Ni and Gernay, 2021).

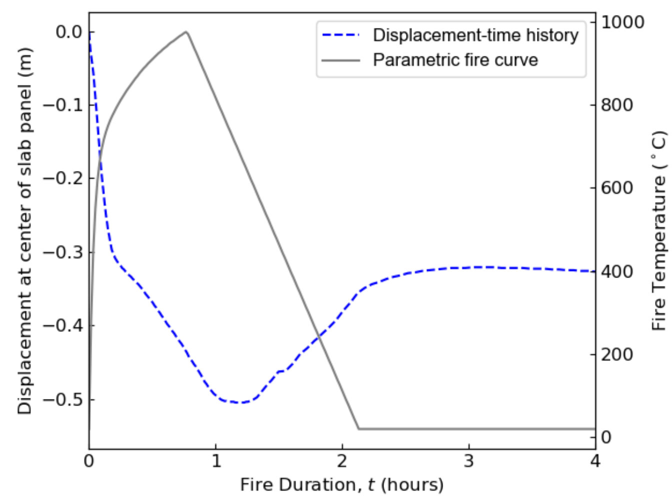

Figure 3: Displacement time-history at the center of slab panel during the natural fire exposure.

5 PROBABILISTIC EVALUATION OF THE FIRE PERFORMANCE

The probabilistic characterization of the fire performance for the composite slab panel is carried out by developing fragility curves. Such fragility curves can be developed based on a brute Monte-Carlo (MC) approach where a direct evaluation of the structural performance is carried out for a large number of realizations. Herein, 1000 realizations are developed through Latin hypercube sampling (LHS) scheme for the probabilistic investigations.

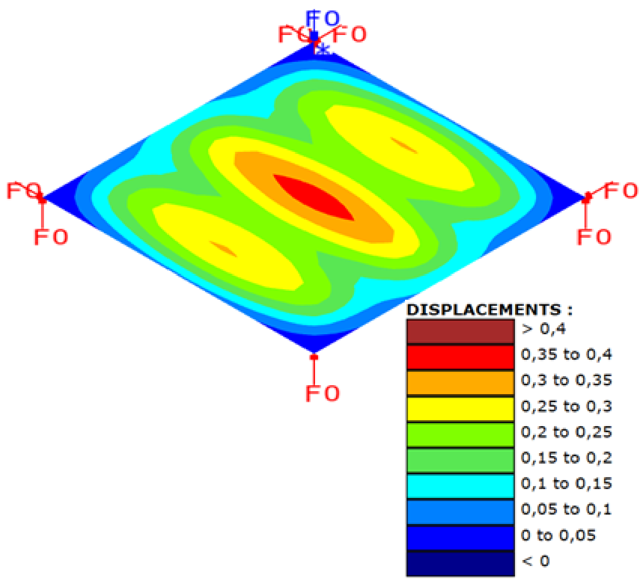

Figure 4: Residual displacements for the fire-exposed slab after cooling (24 hours since start of the fire exposure).

Table 1 lists the uncertain variables and their distribution considered for the probabilistic study, similarly to the assumptions made by Gernay et al. (2019b). As shown in Table 1, the probabilistic model for the strength retention factors for the concrete, reinforcements and steel at elevated temperature is as specified in Qureshi et al. (2020). This model incorporates uncertainty both at ambient and elevated temperatures. The residual capacity of the structure exposed to fire is assessed in the three stages visualized in Figure 1. The subsections below list the results for the three stages of the residual capacity evaluation of the slab panel.

5.1 *Initial structural capacity*

The initial structural capacity for the slab panel is evaluated considering the material properties at ambient temperature. For this probabilistic evaluation, only the retention factor parameters for concrete strength and steel strength as listed in Table 1 are considered, since only these parameters are stochastic at ambient temperature as discussed above.

Figure 5 shows the cumulative density function (CDF) for the load capacity of the slab panel at ambient temperature. The load capacity of the slab panel for the mean values of stochastic parameter is 21.13 kN/m^2, while the probabilistic evaluation estimates it as 21.74 kN/m^2. The coefficient of variation (COV) for the evaluated initial capacity of slab panel is 0.15. The load capacity of the slab panel with 99% exceedance probability (1% quantile) is evaluated as 16.91 kN/m^2. This result indicates a considerable effect of the uncertainties in the initial load capacity of the slab panel.

Table 1: Uncertain variables and their characteristics for the probabilistic study of composite slab panel.

Stochastic variables	**Distribution**	**Mean**	**Standard deviation**
Concrete strength retention factor parameter, ε_{kfc}	Logistic model (Qureshi et al., 2020)	Temperature-dependent	Temperature-dependent
Rebar yield strength retention factor parameter, ε_{kfy}			
Steel yield strength retention factor parameter, ε_{kfys}			
Thermal conductivity, *k*	Normal (Khorasani et al., 2015)	Temperature-dependent	Temperature-dependent
Fire load density, *q*	Gumbel	511	153.3
Opening factor, *O*	Deterministic	0.0409	-

5.2 *Structural performance during fire exposure*

Figure 6 visualizes, the displacement time-history for 5 selected realizations for the slab panel. As illustrated by these results, some of the LHS samples remain stable for the entire fire duration, including cooling, while others fail. Out of the 1000 LHS sample points, 238 cases show failure during the fire. Thus, the probability of structural failure is evaluated as 23.8% for the considered uncertainties and structural loading scenario. Note that variability in the imposed load is not considered, and a total characteristic load of 7.61 kN/m^2 is applied. The LHS samples which survive the fire are further evaluated for their residual capacity.

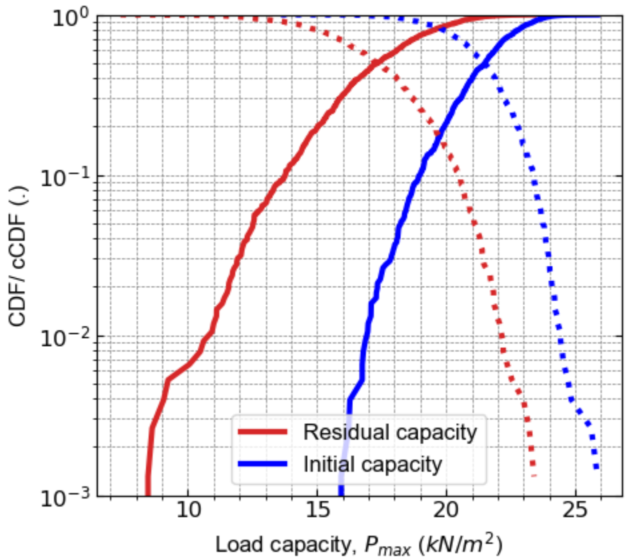

Figure 5 Cumulative distribution function for the initial capacity (1000 LHS) and residual capacity of the slab panel (768 LHS realizations that survived the fire burnout).

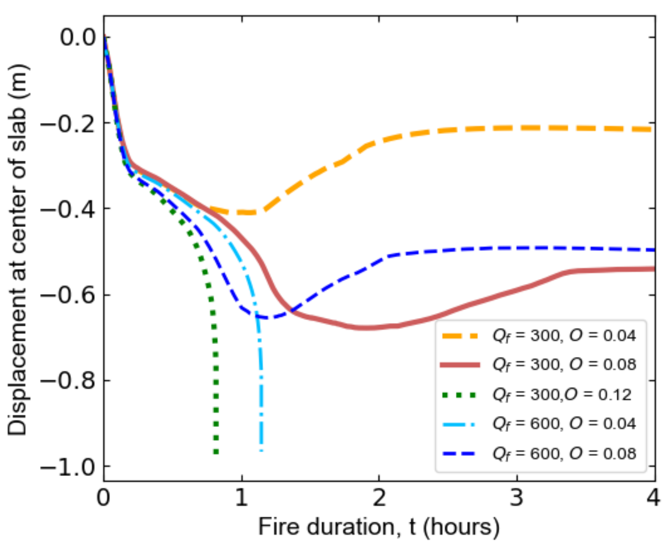

Figure 6: Displacement-time history at center of slab panel for different LHS realizations.

5.3 *Residual deformation and residual capacity*

The damage to the slab panel after the fire exposure is represented by residual deflections. Figure 7 displays the probability density function (PDF) for the residual vertical displacements at mid-span for the slab panels which survived the fire burnout.

A mean residual displacement of approximately 327 mm is obtained for the slab panel, while the maximum observed residual displacement was 850 mm. Based on Ni and Gernay (2021), the maximum residual displacement for reparability of slab panels is estimated as 152.5 mm (equivalent to 1/60 of the slab length). Thus, from this observation, most slab panels are considered irreparable. This is important information for the structural fire design: although the slab panel has a high probability of surviving the fire, it is highly likely that the demolition and reconstruction of the slab panel after the fire will be required.

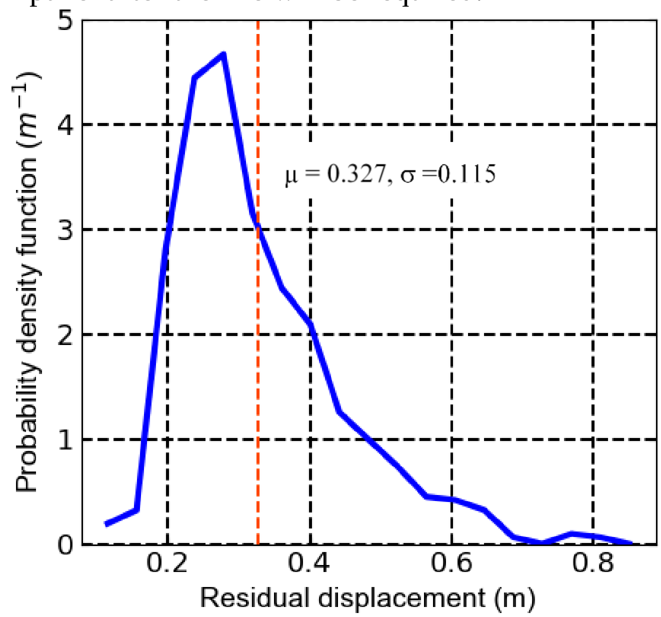

Figure 7: Probability density function (PDF) for the residual displacements of fire-damaged slab panels after 24 hours.

The residual structural capacity after fire exposure is evaluated by applying an incremental mechanical load on the fire-damaged structure according to the procedure described in Figure 1. The residual capacity is evaluated only for the LHS samples of the slab panels which remained stable during the fire exposure and after the cooling phase. Figure 5 shows the CDF developed for the residual capacity together with initial capacity of the slab panel based on the considered uncertainties. The mean residual capacity of the slab panel is observed to be 17.40 kN/m^2, while the mean load capacity at the ambient temperature was 21.13 kN/m^2. Thus, the mean load capacity of the structure decreases by approximately 18% for the fire exposure of the slab panel. Also, the structural load capacity with 99% exceedance probability is reduced to 10.77 kN/m^2 after fire exposure (down from 16.91 kN/m^2). The total characteristic load for the slab panel is 7.61 kN/m^2 (5.42 kN/m^2 imposed load and 2.21 kN/m^2 permanent load). Thus, the structure can be considered safe based on the evaluated residual capacity for the assumed uncertainties. The above suggests that the residual capacity is sufficient for continued use of the composite panel, but that large residual deformations will nevertheless require the panel to be replaced post-fire.

Figure 8 shows the scatter plot for the ratio of residual to ambient load capacity of the slab panel for the 762 realizations that survive up to burnout. Figure 9 highlights that the residual capacity of the slab panel varies from 1 to 0.4 of the ambient load capacity. A higher fire load density results in a lower residual capacity ratio, but large scatter exists. The cases of the slab panel with a fire load higher than 800 MJ/m^2 experienced failure during the fire independently of other uncertain parameters. For fire loads exceeding 700 MJ/m^2, all surviving slabs experienced a decrease in residual capacity of at least 30%, because of the severity of the fire action. For smaller fire load, the dispersion in residual capacity is very large as other factors also play a role.

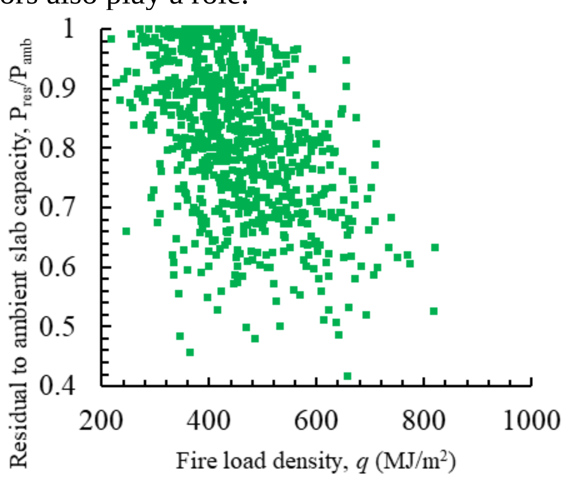

Figure 8: Residual to ambient load capacity ratio as a function of the fire load density (768 LHS realizations that survived the fire burnout).

6 CONCLUSIONS

In this study, the performance during and after fire of a composite slab panel comprising concrete slab and steel beams has been investigated considering natural fire exposure. The probability of failure of the slab panel was estimated to be 23.8% considering a characteristic load of 7.61 kN/m^2 (5.42 kN/m^2 imposed load and 2.21 kN/m^2 permanent load). For the cases that did not fail during the fire exposure, a mean residual displacement of 327 mm was observed. This exceeds a typical threshold for repairability of such structures (1/60 of the slab length = 152.5 mm), indicating that the slab panel will likely need to be demolished and replaced after a compartment fire. Besides permanent deflections, another important consideration for post-fire repairability and re-usability is the residual capacity of the structure. The load capacity of the slab panel with 99% exceedance probability (i.e. the 1% quantile) was reduced from 16.91 kN/m^2 at ambient temperature to 10.77 kN/m^2 after fire exposure, i.e. an additional 36% decrease due to the fire event. In some cases, the slab panel showed a reduction of up to 60% of the capacity after fire exposure. Based on the current study, it can be concluded that the continued use of a composite slab panel post-fire is likely governed by residual deformations. In order to evaluate residual deformations, a transient structural fire evaluation is required. The study further functions as a proof-of-concept for the probabilistic evaluation of structural performance during and after fire in the design phase. Such evaluation ultimately enables to determine an optimized structural fire design upfront.

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