Judgment-Based Fragility Assessment of Reinforced Concrete Buildings Exposed to Fire

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# 1    INTRODUCTION

Fires can cause substantial damage to buildings, both non-structural and structural, as evidenced by multiple fire-induced structural failures of buildings in recent decades (e.g., [1]) and the substantial cumulative costs of fires reflected in fire statistics internationally (the economic loss due to fire reaches 1% of the Gross Domestic Product in developed countries) [2]. Whilst current codes and design guidance (e.g. [3]) allow structural engineers to design for the principal performance driver in fire – namely life safety – comparatively little thought or guidance is typically given (with some notable exceptions) during the structural design phase of a building to the mitigation of direct and indirect economic losses, cultural and historical losses, reputational damage, or environmental losses that significant structural fires may cause. Furthermore, if the structural damage is known, there is relatively little information available in the literature on the repair and strengthening of fire-damaged structures [4].

Holistic, quantified 'loss' estimation for structures under extreme or accidental loads is not a novel concept, however, and there has been a recent trend towards developing probabilistic frameworks for structural fire loss estimation (e.g., [5]). This is typically undertaken in line with the Pacific Earthquake Engineering Research (PEER) framework that exists for seismic loss mitigation (e.g., [6]). An essential component of any loss estimation procedure is the quantification of a building's *fragility*; i.e. the likelihood of the building experiencing damage of a given magnitude, if a hazardous event occurs (e.g. earthquake, tsunami, fire, etc.). In the PEER framework, a building's fragility is assessed by explicitly incorporating the multiple sources of uncertainty that are prevalent in any variable loading situation, in principle also including fire loading. Sources of uncertainty include the characteristics of a building fire (e.g. the fire duration and temperature, or the location of ignition) as well as of the building itself (e.g. the ventilation conditions, construction materials, properties of the critical elements, and so on).

The PEER framework handles the problem of multiple sources of uncertainty by focusing on the construction of two key interrelationships. The first of these relates a fire *intensity* measure (*FIM*) with a measure of the structural *response* (*SRM*). The second relates this *SRM* with a discrete fire *damage state* (*DS*); determined using a specific damage scale relevant to the building class in question. These two interrelationships are then coupled to construct a set of fragility curves that correspond to discrete damage states included in the damage scale (see Fig. 1). A fragility curve is a continuous fire intensity-to-damage

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relationship that expresses the probability that a building of the examined class will suffer damage corresponding to a specific damage state (or greater) for a given fire intensity measure. Symbolic fragility curves for a hypothetical 4-state damage scale are illustrated in Fig. 1.
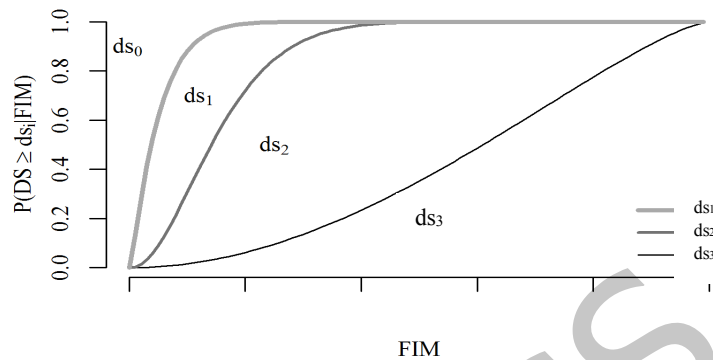


Fig. 1: Fragility curve for a hypothetical 4-state damage scale ranging from no damage to structural collapse.

A building's fragility can be assessed empirically from: post-event data (e.g., empirical seismic fragility assessment by Rossetto and Elnashai [7]); analytically, based on simulating the building's performance (e.g., analytical seismic fragility assessment by Rossetto and Elnashai [8]); expert judgment (e.g. expert-judgement seismic fragility assessment by Jaiswal et al. [9]), or some combination of these methods. To date, full-frame response research in structural fire engineering has concentrated almost exclusively on steel-framed, steel-concrete composite buildings, and only limited research has been performed to quantify the fire fragility of cast-in-place reinforced concrete (RC) structures [10]. This may be due to the relative simplicity of structural steel materials as compared with concrete, the clear economic advantages of taking a rational approach to structural fire engineering for these types of structures, and the well characterized real life fires that have occurred and large scale experimental programmes that have been undertaken in composite steel frame structures (e.g. [11]). This has enabled validation of complex modelling of such structures under multiple credible thermal loading scenarios, and has provided a reasonable amount of data, both analytical and empirical, that can be used in the fire fragility assessment for steel-framed buildings. Such a rich set of empirical data is not available for reinforced concrete structures exposed to fire, and this prevents detailed and confident validation of models for the full-frame response of RC structures in real fires. Recently, Lange et al. [5] adopted the PEER framework approach in order to analytically assess the annual fire cost of a composite building. In their study, the fire intensity is measured in terms of peak compartment temperature, which has been determined by the use of the Eurocode parametric fire, and the response in terms of the deflection of the slab. Repair costs, times and casualties have been associated with thresholds of the deflection mainly based on assumptions.

Given the costs and difficulties associated with large scale tests on real buildings, and the absence of any fully validated capability to model the full frame response of concrete

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structures, another means of generating input data is needed. This paper adopts a novel approach, eliciting judgments from internationally leading experts in the field of cast-in-place reinforced concrete structures in fire to generate the data necessary to create PEER-type fragility assessments for concrete structures in fires. This paper presents the anonymized results of 13 international experts and resulting analyses from an expert elicitation workshop held in Shanghai in June 2014, the results from which aided in the construction of the first ever elicited expertise-based fragility curves for the floor slabs and supporting columns of a generic, mid-rise, open-plan, cast-in-place reinforced concrete frame exposed to fire,

## 2 JUDGMENT-BASED FRAGILITY ASSESSMENT METHODOLOGY

Fragility assessments require large amount of data for multiple building construction types and materials. Four main variables are central to the proposed fire fragility assessment procedure, namely: a. definition of the building class, b. a relevant damage scale, and appropriate measures of c. fire intensity and d. structural response. Whilst in the current paper the building class has been defined by the authors (i.e. cast-in-place concrete frames) to necessarily limit the scope of inquiry, the remaining variables were analysed and quantified using expert elicitation, as described below. The 13 international experts were asked during the elicitation workshop to judge the responses of a generic mid-rise cast-in-place reinforced concrete frame when exposed to different fire intensities, and then to judge the level of response that would be required to cause a given level of damage. The resulting fire intensity-to-structural response and structural response-to-damage relationship judgments from each expert are then analysed to form useful data to be used within a fragility assessment, following the philosophy of the PEER framework. These two relationships are then combined (see procedure in section 4) to understand the intensity-to-damage fragility assessment of cast-in-place concrete structures in fire.

The expert elicitation and judgement analysis, and fragility assessment methods are detailed in the following sections.

## 3 EXPERT ELICITATION METHOD

The main challenge in any expert group elicitation is how best to combine the various experts' opinions when – almost invariably - the experts have different levels of competence when it comes to judging uncertainties. In general, there are two types of expert elicitation approaches: a. mathematical, and b. behavioural [12]. Behavioural approaches attempt to achieve some level of consensus among the experts, who are allowed to interact and review their judgments during elicitation workshops in light of the opinions of other experts. However, an unwanted tendency to conformity rather than genuine agreement has been observed when using this approach (e.g. [13]). By contrast, mathematical approaches limit direct interaction amongst the experts at the point when they make their judgments. These judgments are treated as subjective probabilities of an uncertain quantity, and are then combined mathematically, either by performance-based weighting of each expert's estimates (e.g. [14]) or through the use of Bayesian statistics (e.g. [15]).

In general, for a complex and uncertain problem such as quantified structural fire fragility analysis, if a single expert is asked to provide their judgment of the expected value of a variable, and also to quantify their uncertainty, most will typically provide undependable estimates of the mean, often accompanied by unrealistically small uncertainties (here, expressed as 90% credible intervals). By contrast, if a large group of experts is elicited and their opinions are weighted equally, a good estimate of the mean of the variable is usually obtained, but the associated uncertainty is wide in most cases. Overall, mathematical approaches to expert elicitation are regarded as being more reliable, more reproducible, and fairer than behavioural approaches for aggregating expert opinions [9, 12, 14]. In the current study, one such approach, Cooke's method [14], is adopted.

## 3.1 COOKE'S METHOD

Cooke's method has been widely applied for expert elicitation in a number of diverse applications, e.g. [16]. As a first step, Cooke's method requires a group of experts to answer 'seed' questions, the values of which are all realisable subject matter data; however, experts are not expected to know these values precisely, but are expected to be able to bracket them closely with suitable credible intervals. Gauging the relative ability of each expert to quantify these uncertainties, accurately and informatively, then allows the experts' judgments to be calibrated (i.e. weighted) when they are asked to quantify other, unknown variables in the second stage of elicitation. Generally, however not always, this produces an outcome distribution with formally quantified uncertainty that falls somewhere between the extremes of inaccurate individual judgments and excessively vague combinations of collective views. Recently, Jaiswal et al. [9] studied the seismic fragility of a number of building types using this procedure for expert group elicitation where the experts were asked to relate ground motion intensity measures directly to the probability of damage.

It should be noted that to encourage experts to state their true engineering beliefs in the elicitation process, individual experts provided their judgments confidentially to the neutral facilitator, and their identities are not attached to any of the findings presented here. This is an important procedural aspect of a structured expert judgment elicitation, aimed at reducing induced biases, such as might arise in conservative public expression of views.

The experts' ability to perform accurate judgements is assessed by measuring their performance over a series of seed items, for which they provide three distribution-defining quantiles (5%ile, 50%ile, 95%ile) motivated by informed judgement. The true realization for each question is a single value that is not immediately available to the experts, but represents a value they can be expected to capture within a meaningful credible interval, corresponding to a 90% credible interval in the current case, together with an appropriate median value.

From their responses to the set of seed items, the ability of each expert to gauge the uncertainty around an unknown quantity is assessed by determining how well-calibrated and informative the experts' opinions are, as shown in Fig. 2. If most of the 'true' answers of the seed questions fall within an expert's 90% intervals, then that expert is considered to be well calibrated, provided that the number of 'true' values above and

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below his or her elicited median values is approximately equal. In Fig. 2, the more an expert's empirical distribution approaches a wide uniform distribution, the easier it is to capture a true value; however, on the item in question, that expert is less informative than one with a narrower credible range. Usually in an expert elicitation, the set of seed items comprises between about eight and 20 questions; this provides a fair and discriminating statistical test of individual experts' performances. The Classical Model algorithm penalizes poorly calibrated, over-opinionated, or less-informative experts with low weights (see Fig. 2).

Thus, Cooke's Classical Model objectively calculates a performance-based weighting for each expert by combining the two aforementioned metrics, namely their calibration (statistical accuracy) and their information score. The procedure is described in detail in the literature [16].
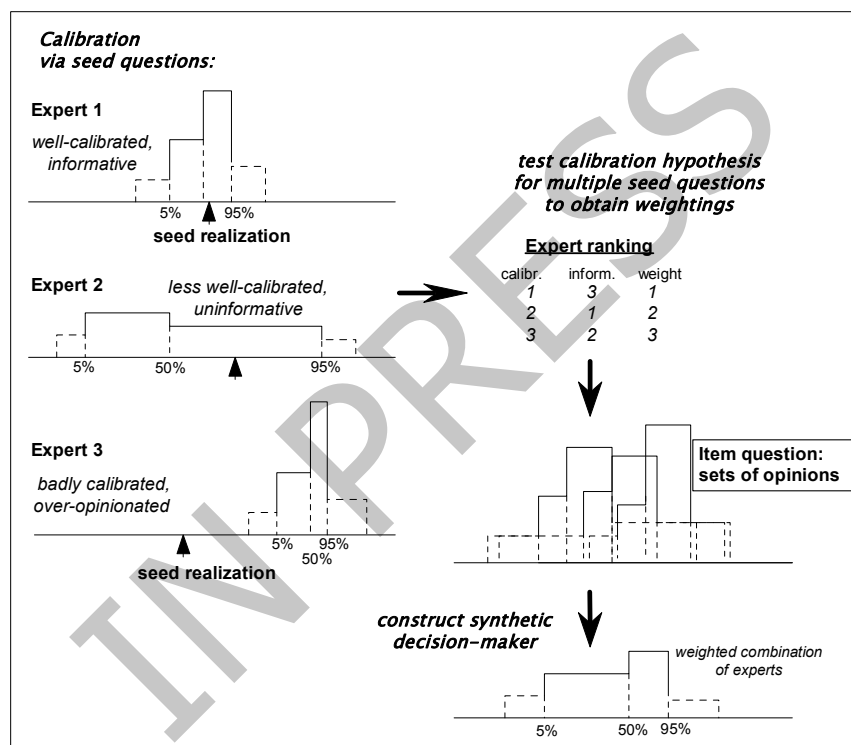
Fig. 2: Schematic showing ranking of experts based on their measured information and calibration scores.

## 3.2 EXPERT ELICITATION WORKSHOP

Based on knowledge of the structural fire engineering community, the authors identified 40 experts from the list of participants at the 7th International Conference on Structures on Fire (Shanghai, China, 2014) who were then invited to attend a one-day workshop following the conference. Thirteen experts agreed to participate, and information regarding their backgrounds was gathered using a pre-workshop survey. Thirteen experts are considered to be a large enough panel for a constructive elicitation [17]. The experts' responses to the pre-workshop survey showed that they had cumulative experience of fire response of 217 RC buildings of different heights, structural systems and ages. This

experience was mostly research-based, as opposed to practical design or post-fire repair work. The experts had participated in the construction or design of only 23% of these buildings. All experts had experience of experimental or analytical methods for investigating the performance of RC elements in fire, but only 25% had ever assessed (or visited) a real fire-damaged RC building. All the experts were novices in formalized expert elicitation.

The workshop was divided into two sessions. During the first session, each of the thirteen experts assessed the uncertainties associated with the seed items. In the second session, the experts were invited to provide their uncertainty judgments on the 'target' questions, presuming that the whole 1st-floor area of a generic RC building (shown in Fig. 3) is uniformly exposed to fire. The uncertainty around each variable was quantified by providing expert judgments on the values with 5%, 50% and 95% probabilities of exceedance.

Questions aiming to quantify uncertainty around 16 seed variables were prepared and covered a wide range of related subject matter issues, including: a. general fire loss estimation, b. the structural performance of RC structural elements in fires, c. the development of fires in compartments, and d. the material properties of RC and steel members exposed to fire. The seed variable realizations were known to the facilitator; however, crucially - as noted above - the experts were not expected to know the precise answers but to be able to provide credible ranges that captured the answers reliably and informatively. The experts' performance in quantifying uncertainty around these questions determined their weightings for the target questions.

Questions aiming to determine the uncertainty distributions of 48 'target' variables were also formulated. Twenty-four target questions aimed at understanding the *structural response-to-damage relationship* and quantifying uncertainties in the spalling, deflection, residual capacity, and peak rebar temperature thresholds at three damage states (i.e. $ds_1$, $ds_2$ and $ds_4$) for both slabs (12 questions) and columns (12 questions). The remaining questions were aimed at understanding the *fire intensity-to-structural response relationship* and to quantify uncertainties in the aforementioned four response measures for slabs (12 questions) and columns (12 questions) when exposed to equivalent durations of the ISO 834 [18] standard fire of 30mins, 60mins and 90mins, respectively.

In the current study, experts' opinions are pooled according as the item weighting scheme to provide the most rational combination of the judgments of the expert group on the quantities of concern for this study. For context, these results are compared (in Section 5) with those that would be obtained by equally weighting all experts' judgments.

In what follows, a proposed fire fragility assessment procedure is presented and illustrated for the construction of elicited expertise-based fragility curves for the floor slabs and supporting columns of a generic, mid-rise, open-plan, cast-in-place reinforced concrete frame exposed to fire.

## 4 FRAGILITY ASSESSMENT METHODOLOGY

The fire fragility of a building class is assessed by coupling the *fire intensity-to-structural*

*response relationship* with the *structural response-to-damage relationship*, in line with the framework proposed by PEER for seismic fragility assessments of different classes of buildings (e.g. [6]):

$$P\left(DS \geq ds_i \mid FIM = x\right) = \int_{SR} P\left(DS \geq ds_i \mid SRM = y\right) f\left(SRM = y \mid FIM = x\right) dy \tag{1}$$

where $DS$ is a random variable representing the structural fire damage suffered by a building with specified characteristics; $ds_i$ is a given level of this damage; $FIM$ is a random variable representing the fire intensity measure; $x$ is a given fire intensity measure level; $SRM$ is a random variable representing the structural response measure of the building; $y$ is a given structural response level. It is evident from Eq. (1) that the assessment of fire fragility requires:

a. determination of the main variables, namely: the building class, a relevant damage scale, and appropriate structural response and fire intensity measures;

b. construction of the fire intensity-to-structural response and the structural response-to-damage relationships, which involve quantification of the uncertainty in predicting the structural response for given fire intensity measure levels and determining the structural response thresholds for given damage states; the uncertainty is quantified herein through elicited experts' judgments, by combining their opinions using Cooke's method; and

c. construction of novel fire fragility curves for cast-in-place RC frames by coupling the two above-mentioned relationships.

The approach adopted to determine each of these is described in what follows.

## 4.1 MAIN VARIABLE CLASSIFICATION

### 4.1.1 Building class

The fire losses, as well as the strategies to mitigate these losses, depend on the building characteristics, most notably the construction material and building height. Buildings have historically been classified for fire safety purposes according to their size and occupancy type, the size of fire rescue equipment (e.g. ladders), presumed or calculated egress times, and the emergence of defend-in-place strategies, particularly in taller or more complex buildings [19, 20].

The current study is focused on the fire fragility assessment of mid-rise cast-in-place reinforced concrete frames, due to the paucity of any specific attention in the structural fire fragility literature. The variability in the selected class (e.g. the height of buildings ranges from 4 to 7 stories or the different sizes of buildings) is expected to unmanageably increase the complexity of the expert elicitation. For this reason, the selected class is characterised by the generic building depicted in Fig. 3, which is considered a typical modern, open-plan, mid-rise concrete cast-in-place structure, of scale and dimensions that might be expected for an open-plan building in the UK. The inter-storey height is 3.75m and the nominal thickness of the floor slabs is 150mm. The dimensions of the internal and external columns are $400 \times 400 \text{mm}^2$ and $400 \times 250 \text{mm}^2$, respectively. The

characteristics of the concrete and reinforcement steel are presented in Table 1. It should be noted that the selected building is loosely based on the reinforced concrete structure tested during the Cardington Concrete frame tests in the UK [21].

### 4.1.2 Damage scale

A quantified damage assessment of a structure exposed to fire requires a definition of an appropriate structural damage scale. A damage scale suitable for the purposes of fragility assessment consists of a number of discrete damage thresholds, which ideally [22]:

a. consider all possible damage mechanisms that a building of a given class is likely to experience when exposed to fire;

b. provide clear and, to the extent possible, unambiguous descriptions of the expected damage in each state;

c. associate each damage state with quantifiable thresholds of one or more structural response parameters that allow the interpretation of the damage state from structural analyses, fire tests, observations of real fires, or expert elicitations; and

d. relate each state with a level of repair *and* an associated repair cost to replacement cost ratio.

However, quantifiable damage scales such as described above are not yet available within the structural fire safety engineering community; current design assessments are generally based on pass/fail criteria related to elements' ability to carry sufficient loads for a prescribed duration of exposure to a standard fire [18] when tested in a fire testing furnace. This binary structural fire compliance criterion is impractical in assessing the potential fire losses in concrete structures, which requires a more detailed classification of damage.

The Concrete Society [4] has previously proposed a qualitative scale for assessing damage sustained by the four main types of RC structural elements (i.e. slabs, columns, beams, and walls) exposed to fire, and their associated repair strategies and methods (see Table 2). This damage scale meets criteria (a), (b) and (d) outlined above, since it consists of five clearly defined states of increasing severity, ranging from *none to extensive damage*, and each state is associated with a level of repair. Nonetheless, this scale falls short of the needs of fragility assessment since it does not relate each damage state to thresholds of quantifiable measures of structural response, i.e. (c) above.

Table 1: Characteristics of slabs and columns assumed in the current expert elicitation.

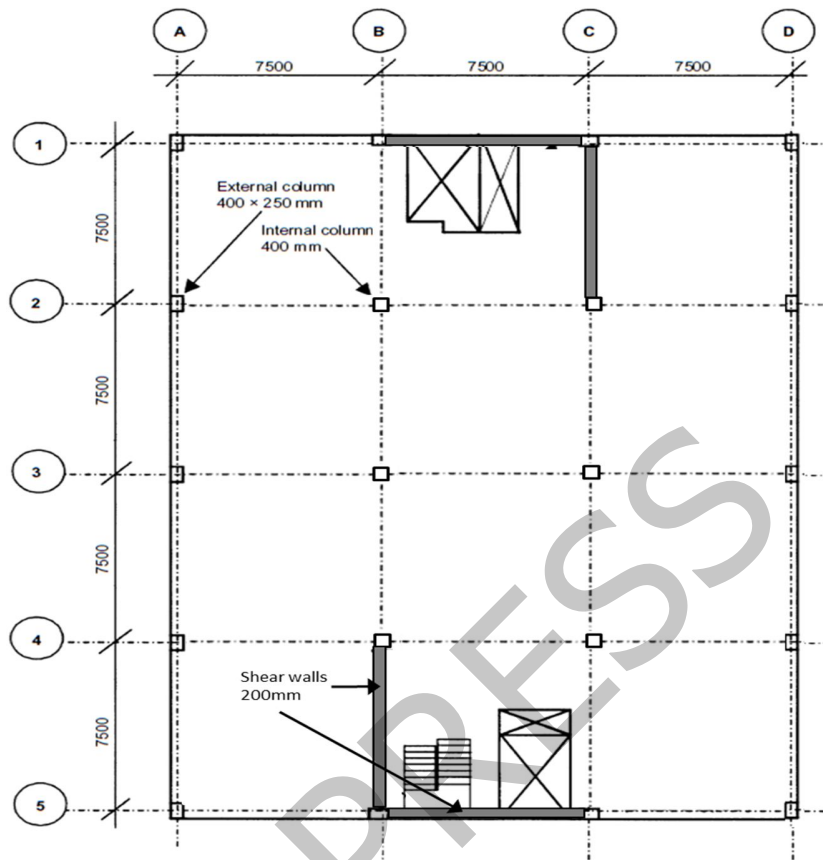| Element | Concrete | | | | Steel |
| | Strength | Moisture content (%) | 48mm permeability of (in $m^2$) | Cover in mm | ø (in mm) |
|---|---|---|---|---|---|
| Slab | C37 | 3.8 | $6.75 \cdot 10^{-17}$ | 20 | 12-16 |
| Col. | C85 | 4.2 | $1.92 \cdot 10^{-19}$ | 40 | |

Fig. 3: Plan of generic RC office building assumed in the current expert elicitation.

In the absence of any other available guidance in this area, The Concrete Society's [4] damage scale is applied herein in order to classify the fire damage sustained by individual RC structural elements based on observations of the surface appearance of the concrete, i.e. the condition of plaster, the colour of the surface and the level of crazing, and its structural condition in terms of qualitative descriptions of spalling, cracking and deflection or distortion. This paper also seeks to address this shortcoming by developing novel structural response-to-damage relationships that quantify the thresholds of the four selected structural response measures described in the next section (i.e., spalling, deflection, residual capacity, and peak rebar temperature) for each damage state.

### 4.1.3 Structural response measures

Appropriate and quantifiable structural response measures should be identified which are capable to measure the impact of the fire on the selected structural elements. In the current study, four structural response measures have been selected based on conventional criteria used (however subjectively) within the construction industry, namely:

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a. the percentage (%) of the exposed surface area of a given element which has spalled due to heating, such that it could be classed in a given damage state;

b. the peak rebar temperature (in $^{o}$C) associated with $ds_i$, again in the absence of spalling.

c. the level of deflection, $D$, measured after cooling, which would be associated with a given damage state, and stated as:

$$D = \frac{L}{X} \qquad (2)$$

where $L$ is the length (span) of the element; $X$ is a parameter which determines the deflection in the mid-span on a slab or the end point of a column (i.e. drift);

d. the percentage of the residual capacity of the examined element that can be associated with a given damage state, $ds_i$, on the assumption that the element sustains no other structural damage, i.e., spalling or deformation. Note that the residual capacity is measured as the ratio of total remaining capacity to original capacity regarding the axial load capacity for columns and flexural capacity for slabs (i.e. shear capacity is ignored in this initial study); and

It should be noted that the first two measures (i.e, (a) and (b)) are determined at section level and the latter two at element level. Measure (a) is chosen because spalling is widely regarded (rightly or wrongly) as a key parameter in assessing fire damage to concrete buildings [4]; parameters (b-d) are selected because they are often used as end-point criteria in standard structural fire tests on isolated structural elements [18, 23, 24], and because they can be quantifiably assessed in experiments and through structural analysis.

### 4.1.4 Fire intensity measures

The fire intensity measure is the key input to a fire fragility analysis, since it quantifies the hazard. It has a probabilistic distribution due to the varying nature of fire hazards (e.g. varying fire loads, ventilation, compartment boundaries, etc). The aim of the fire intensity measure is to provide a single parameter that can encompass many variations of the hazard, such that these can be directly compared and associated with one another. For instance, Lange et al. [5] used a fire intensity measure consisting of peak compartment temperature when studying the structural fire fragility of steel-framed composite buildings. Clearly there can be many variations of fires that might lead eventually to the same peak compartment temperature, and it is likely that the thermal path to peak temperature (i.e. rate of heating) is important for the structural fire response. Other *FIM* choices might be: fire duration, total heat released, etc; however, all have specific inadequacies. Lange et al. [5] used peak fire temperature in conjunction with a PEER framework assessment of a steel framed building; this *FIM* is reasonable for unprotected steel framed buildings within a range of likely heating rates, since steel has high thermal conductivity and it is common to assume for structural fire analysis that unprotected steel temperatures are similar to the compartment gas temperatures in a fire.

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Table 2: Characteristics of the damage scale adopted in the current study based on a damage scale proposed previously by The Concrete Society [4].

| DS | Surface Appearance of concrete | | | Description |
| | Condition of plaster/ finish | Colour* | Crazing | |
|---|---|---|---|---|
| $ds_0$ | Unaffected or beyond extent of fire | | | |
| $ds_1$ | Some peeling | Normal | Slight | Damage primarily cosmetic in nature, which does not impact on the design or repair of the structural fabric of concrete building. |
| $ds_2$ | Substantial loss | Pink/Red | Moderate | A small amount of damage has been experienced by the element to the effect that some small scale remedial action is required to enhance the element's remaining ability to perform its structural function(s). |
| $ds_3$ | total loss | Pink/Red Whitish grey | extensive | The element has experienced a significant, but not catastrophic, amount of damage to the effect that, with significant remedial action, it can be reinstated to perform its structural functions |
| $ds_4$ | destroyed | whitish grey | surface lost | The damage caused by the fire is so extensive that it is no longer viable to repair and reuse the element and replacing the element with a new element is the only option. The building has not suffered a disproportionate collapse. |

Determining a suitable intensity measure for reinforced concrete structures is less straightforward for two main reasons: a. concrete has comparatively low thermal conductivity, which introduces an important time dependency into the thermal, and thus structural, responses of RC elements and structures, and b. RC structures incorporate steel reinforcing bars, which are important in the response of elements and structures and which have different thermal and mechanical properties. Thus, for RC structures *FIM*s will also depend on the effect that a hazard has on the structure and the *SRM* being assessed (i.e. for some choices of the *SRM*, peak rebar temperatures might be a suitable *FIM*; however, for other *SRM* choices the area under the time versus compartment gas temperature curve might be more appropriate).

Ideally, a large database of experimental and real fire structural response observations and data would be available to inform decisions on which *FIM*s and *SRM*s are most suitable for concrete buildings; however, there is a scarcity of such data – thus computational analysis and expert opinion must be relied upon.

The intensity measure chosen in this initial study is based on Ingberg's [25] original time-equivalence concept, which forms the original basis of structural fire resistance ratings globally, and which states that real and standard fires are equivalent if the area under the real temperature-time fire curve – above a baseline temperature of 150°C (Area B in Fig. 4) – is equal to the area under the standard temperature-time fire curve, for instance from ISO 834 [18] or similar – again above a baseline temperature of 150°C (Area A in Fig. 4).

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This gives an equivalent exposure time when compared to the standard fire curve ($T_s$). Ingberg [21] originally proposed this area-based equivalence concept as a means of relating the standard temperature-time curve, used in furnace testing to demonstrate fire resistance, with the time-temperature history experienced in real fires for a given building type and occupancy. It is the simplest parameter that can be used in any attempt to capture both peak temperature and duration of heating (both of which are important parameters for most structural materials and systems) in a fire; however, the concept is not without flaws since it fails to distinguish between a short, hot fire and a long, cool fire with the same area – such fires might result in significantly different structural outcomes. Nonetheless, Ingberg's equivalence concept is a reasonable first choice due to its historical relevance and explicit relationship to fire resistance ratings still prescribed in modern building codes.
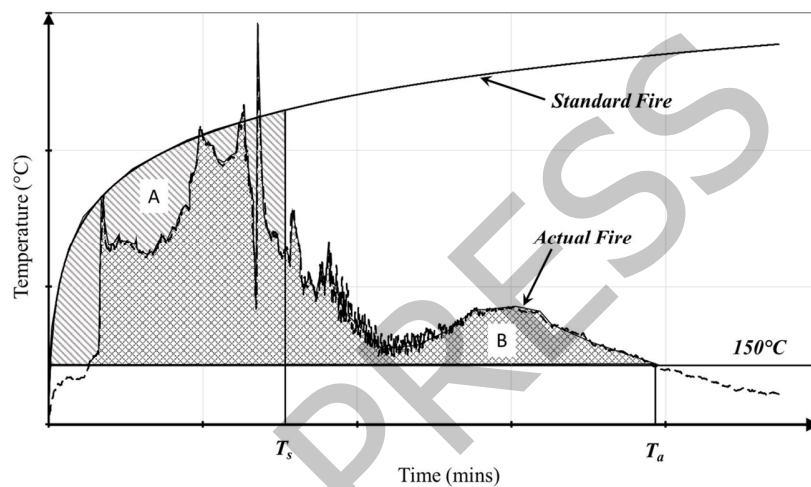


Fig. 4: Schematic showing Ingberg's [25] equivalent temperature-time fire curve concept.

## 4.2 RELATIONSHIP UNCERTAINTIES

### 4.2.1 Uncertainty in fire intensity-to-structural response relationships

In general, the value of any measure of structural response over a class of buildings is uncertain due to the variability in both the fire intensity and in the building characteristics. In this study, a generic building is adopted, therefore the fire intensity-to-structural response relationships reflect mainly the fire uncertainty. For a given level of FIM, the exact level of the structural response is not known precisely, but can be characterized by a probability distribution function, selected according to the properties of the structural response measure. From the definitions of the structural response measures in Section 4.1.3, spalling and residual capacity are measures whose values fall in the range (0, 1). For this reason, these structural responses, for given fire intensity levels, are assumed to follow beta distributions. In the case of deflection and peak reinforcement bar temperature, values are strictly positive and therefore they are considered to follow lognormal distributions, widely used in the earthquake fragility engineering field (e.g. [26, 27]):

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$$f\left(SRM = y \mid FIM = x\right) = \begin{cases} \text{pdf.beta}\left(\alpha, \beta\right) \\ \text{pdf.lognormal}\left(\lambda, \zeta\right) \end{cases} = \begin{cases} \dfrac{\Gamma\left(\alpha+\beta\right)}{\Gamma\left(\alpha\right)\Gamma\left(\beta\right)} x^{a-1}\left(1-x\right)^{\beta-1} \\ \dfrac{1}{x\zeta\sqrt{2\pi}} e^{-\frac{\left(\ln(x)-\lambda\right)^2}{2\zeta^2}} \end{cases} \quad (3)$$

where *SRM* is the structural response measure; *y* is a given structural response level; $\alpha$ is the shape parameter; $\beta$ is the rate parameter; $\lambda$ is the lognormal mean; $\zeta$ is the lognormal standard deviation *FIM* is the fire intensity measure; *x* is a given fire intensity level; $f(.)$ is the probability density function, pdf; $\Gamma(.)$ is the gamma function. The unknown parameters (i.e. $\alpha$, $\beta$, $\lambda$ or $\zeta$) are estimated by fitting an appropriate probability distribution to the empirical distributions obtained by combining the opinions of experts using the Classical Model proposed by Cooke [16].

### 4.2.2 *Uncertainty in structural response-to-damage relationships*

The construction of a damage scale relevant to an RC structural element requires the estimation of a number (four in the current paper) of structural response measure thresholds for each damage state. However, the exact values of these thresholds are also uncertain (see Fig. 1). Thus, the structural response-to-damage relationships focus on the estimation of the mean of each threshold, as well as the uncertainty around the threshold of a structural response measure for a given *damage state*. As with the discussion in Section 4.2.1, these relationships are expressed in terms of probability distributions, whose shapes depend on the properties of each structural response measure. Here, the spalling or residual capacity thresholds are assumed to follow beta distributions, and the deflection or peak rebar temperature thresholds are considered to follow lognormal distributions:

$$f\left(DS \ge ds_i \mid SRM = y\right) = \begin{cases} \text{cdf.beta}\left(\alpha, \beta\right) \\ \text{cdf.lognormal}\left(\lambda, \zeta\right) \end{cases} = \begin{cases} \dfrac{\Gamma\left(\alpha+\beta\right)}{\Gamma\left(\alpha\right)\Gamma\left(\beta\right)} \displaystyle\int_0^x t^{a-1}\left(1-t\right)^{b-1} dt \\ \Phi\left(\dfrac{\ln(y)-\lambda}{\zeta}\right) \end{cases} \quad (4)$$

where *DS* is the damage state; $ds_i$ is a given damage state; and $P(.)$ is the cumulative probability function. The parameters of these distributions are estimated by fitting the appropriate distribution to the empirical cumulative probability distribution functions (CDFs) obtained through the Cooke's method [16].

### 4.3 CONSTRUCTION OF FRAGILITY CURVES

Having determined the shapes of the fire intensity-to-response and response-to-damage relationships, the fragility curves are then obtained from Eq.(1). In the current study, fragility curves are constructed by numerically coupling the aforementioned two relationships, using a method proposed by Porter and Kiremidjian [27]. According to Porter and Kiremidjian [27], for a specified fire intensity level (*FIM = x*), the probability

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that a building will reach or exceed a given damage state is estimated by a Monte Carlo technique which requires the following steps, illustrated by an example iteration and also depicted in Fig. 5:

a. A random number, $u$, is generated from a uniform distribution ranging between [0, 1]:

$$u \sim \text{uniform}\,(0,\,1) \tag{5}$$

Essentially, $u$ represents a random level of the cumulative distribution of the structural response given $x$ (see Eq. (3)). In Fig. 5a, $u = 0.55$.

b. The structural response level, $y$, with cumulative probability equal to $u$ is estimated from the appropriate fire intensity-to-response relationship (see Eq. (3)), the corresponding structural response level, $y$, is calculated. From Fig. 5a, $y$ can be estimated as:

$$y = \left(P(SR \leq y \mid FIM = x) = u = 0.55\right)^{-1} \tag{6}$$

c. For $y$, the probability that a damage state will be reached or exceeded is estimated from the response-to-damage relationships (see Eq. (4)). From Fig. 5b, the probabilities of exceedance for the three damage states can be determined as:

$$
\begin{aligned}
P(DS \geq ds_1 \mid SRM = y) &= 0.95 \\
P(DS \geq ds_2 \mid SRM = y) &= 0.48 \\
P(DS \geq ds_3 \mid SRM = y) &= 0.04 \\
P(DS \geq ds_4 \mid SRM = y) &= 0.00
\end{aligned} \tag{7}
$$

d. A new random number, $w$, is generated:

$$w \sim \text{uniform}\,(0,1) \tag{8}$$

Depending on its value, the damage state sustained by the building under consideration is thus determined. For example, in Fig. 5b, $w = 0.2$ is considered, for which:

$$P(DS \geq ds_3 \mid SRM = y) \leq w = 0.02 \leq P(DS \geq ds_2 \mid SRM = y) \tag{9}$$

Eq. (9) indicates that the building in this iteration sustained $ds_2$.

A large number of iterations, $N$ (e.g., $N = 10{,}000$), of these four steps is required. Finally, the probability that a building will sustain a damage level $DS \geq ds_i$ is estimated as:

$$P(DS \geq ds_i \mid FIM = x) = \frac{\displaystyle\sum_{i=i}^{n} N_{ds_i}}{N} \tag{10}$$

where $N_{ds_i}$ is the number of iterations for which $DS = ds_i$.

## 4.4 DATA ANALYSIS PROGRAM – EXCALIBUR

The experts' responses to both the seed and target questions are as the input data for the EXCALIBUR expert elicitation software [28]; this is a bespoke software package used for analysing expert judgments and ascribing expert weights according to the Classical Model.

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In order to create an empirical distribution that completely spans a set of expert responses within credible interval bounds, EXCALIBUR calculates tail extensions – beyond the lowest and highest quantiles provided by any individual in the elicitation (in this case 5th and 95th percentiles) – termed the 'intrinsic range' (for the mathematical details, see [14]). In doing this, the program may return small negative values at very low quantiles for variables that should take only non-negative measures (i.e. spalling and residual capacity). As a generic tool, the EXCALIBUR program cannot recognize that certain variables may only take non-negative values, or that others may have strict upper bounds (e.g. 100%). Therefore, when the program produces marginal or infeasible values, the resulting distributions are re-normalized by the analyst and post-processed to condition the data to valid values.
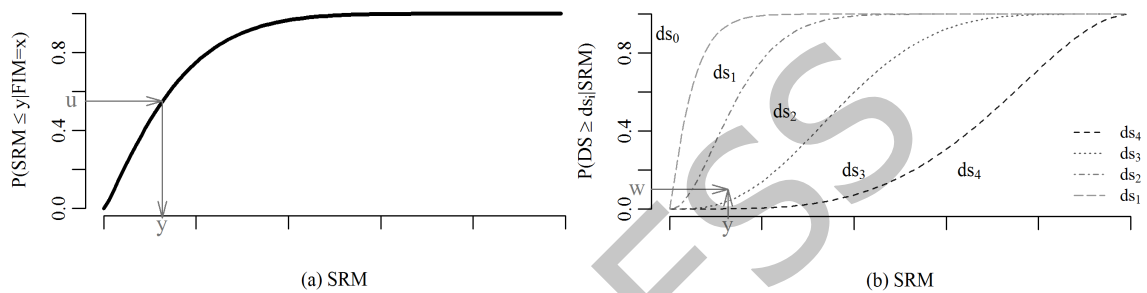


Fig. 5: Schematic representations of the methodology used for the construction of fragility curves in the current paper.

## 5 RESULTS

Empirical CDFs for the three damage state threshold values of the four structural response measures, and the response measure levels for the three fire intensity levels are thus produced from the experts' weighted responses. It should be noted that only the final weighted CDFs are presented in this study (preserving the essential anonymity of the experts, mentioned above). In addition, the empirical CDFs of the $ds_3$ threshold values for the four SRMs are determined by linearly interpolating the corresponding $ds_2$ and $ds_4$ thresholds.

In the present study, an additional distribution structure constraint is imposed to ensure that the three curves retain the expected relative ordering over their whole cumulative distributions, and do not overlap or cross one another at certain percentiles. For example, for spalling damage all percentiles of $ds_1$ are constrained to be lower than those for $ds_2$; whereas $ds_4$ quantiles are constrained to be higher. This is achieved by conditional re-sampling the EXCALIBUR output empirical distributions with the necessary inequality constraints.

The resulting empirical cumulative distributions are then fitted to the two continuous CDFs (Eq.(3) and Eq.(4)), according to the characteristics of each variable. These continuous distributions for the two relationships are then coupled to produce fragility curves.

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## 5.1 STRUCTURAL RESPONSE – TO – DAMAGE STATE RELATIONSHIPS

Table 3 depicts the fire damage scale suitable for the examined building class, which includes the qualitative as well as quantitative (i.e., parameters of the fitted continuous probability distribution functions) descriptions of each damage state. Fig. 6 compares the CDFs of the thresholds for these two element types, and Table 4 compares their means and their 90% interval; this measures the size of the uncertainty in the thresholds for each damage state. It should be noted that the 90% confidence interval of absolute deflection of the slab or column can be obtained from the 90% confidence interval of relative deflection using Eq. (2). The reader should bear in mind that the relative deflection value with a 5% probability of being exceeded corresponds to the deflection value with 95% probability of being exceeded. The curves show the expected shapes, e.g. the CDF of spalling for $ds_1$ falls to the left of the other two curves, indicating the smallest overall level of spalling. Similarly, the CDF of $ds_4$ is to the right of the other two, indicating a larger overall percentage of spalling than for $ds_2$ or $ds_1$.

### 5.1.1 Comments on the weighting method

For context, two alternative expert-judgement pooling schemes are assessed here: the first provided by the Classical Model where the CDFs of the experts' judgements are objectively combined on the Cooke's method empirical performance basis, and secondly, by simple equal weight pooling (i.e. by averaging uncertainty distributions over all experts uniformly). By comparing the credible ranges present in the two approaches, it is normally possible to assess whether there is general consensus on a particular target item within the selected panel of experts. Where there are significant differences of opinion, the performance-based weight pooling scheme determines an optimal solution, which minimizes the scatter that otherwise can be extreme if all disparate views are accorded an equal weighting.

For 13 out of the 24 thresholds (see Tables 5 and 6), using Cooke's weighting scheme results in less uncertainty around the thresholds, as compared to equal weighting for all experts. In particular, the uncertainty regarding relative deflection, for columns, is considerably different between the two weighting schemes, with the Cooke's performance-based expert weighing scheme reducing both the uncertainty and the mean values, as compared with the equally weighted experts' opinions (see Tables 5 and 6). This results in the CDFs (see Fig. 6e) for the three damage states obtained by Cooke's weighting scheme being steeper and to the left of their equal weighting counterparts, thus demonstrating the benefits of Cooke's method in reducing the influences of extreme outlier opinions.

The advantage of Cooke's method can also be seen for cases where it indicates mean thresholds that are similar (sometimes greater) to those derived by equal weighting: there is less uncertainty associated with performance-based solutions than with the equal weights solutions. For example, the mean $ds_1$ threshold for the residual capacity for columns is practically identical (difference $\leq$ 10%) for the two weighting methods, however the uncertainty in the Cooke's method is considerably smaller (see Tables 5 and 6). This results in the CDFs (see Fig. 6i) for the two damage state curves obtained by Cooke's weighting scheme being steeper than their equal weighting counterparts, again

highlighting the ability of the weighted judgments to more tightly constrain the uncertainty than if the whole group is weighted equally.

In other cases, Cooke's weighting scheme leads to similar or larger uncertainty around the damage thresholds than equal weights. For example, the uncertainty around the $ds_4$ threshold for the peak rebar temperature of both columns and slabs obtained by the Cooke's weights is larger than for equal weights. This suggests that the highest-weighted experts believe it is difficult to constrain the uncertainties on these thresholds on the basis of present knowledge and experience. Similarly, the highest-weighted experts gave relatively wide uncertainties around the $ds_1$ residual capacity thresholds for slabs and the $ds_4$ residual capacity thresholds for columns.

These are noteworthy findings, and indicate that further research is needed to better understand the *SRM*-to-*DS* relationships for these response variables. This demonstrates also one of the diagnostic strengths of this form of structured expert pooling, which can often be missed by more subjective or informal group approaches.

### 5.1.2 Comments on different SRMs

The experts judged that half of the *SRM*-to-*DS* relationships regarding spalling, residual capacity and peak rebar temperature are not greatly dependent on the type of structural element; they are similar for both columns and slabs. The thresholds of spalling for slabs and columns are judged practically identical (difference $\leq$ 10%) for all three damage states. This is counter-intuitive in the authors' opinion, since spalling is widely believed to be more likely in columns due to the existence of pre-compressive stresses in these elements.

With regards to residual capacity, the best-weighted experts did not believe that there was a significant difference between $ds_4$ and $ds_1$ thresholds for slabs and columns. Although the differences in the $ds_2$ mean thresholds for residual capacity appears to be negligible, the uncertainty is larger for slabs. Similarly, there is essentially no difference between the $ds_2$ and $ds_1$ thresholds for peak rebar temperature for slabs and columns. However, the CDFs for *slab* peak rebar temperatures corresponding to $ds_4$ appear shifted to the right of the CDFs obtained for *columns* (see Fig. 6m). This indicates that the experts believe that the thresholds are on average higher for slabs than for columns (i.e. slabs can withstand higher temperatures before reaching $ds_4$ damage) but have greater associated uncertainties. It is noteworthy that uncertainty is greatest in the three above-mentioned structural response measures (i.e. spalling, residual capacity and peak rebar temperature) for $ds_4$ and smallest for $ds_1$, as indicated by shallow and steep curves, respectively. This is expected, given the difficulty in predicting the structural performance of RC elements that sustain severe damage, and result in them being judged as being in the most severe damage state.

A different picture is noted for the relative deflection threshold, for which the uncertainty is largest for $ds_1$ and smallest for $ds_4$. For all three damage states, the mean as well as the uncertainty in the relative deflection thresholds appear to be larger for slabs and smaller for columns (see Tables 5 and 6). There is an obvious question as to what the effects are on the deflection thresholds for slabs and columns. For instance, there is 90% probability that the $ds_4$ deflection thresholds for slabs ranges from $L/222$ to $L/7$. For the 7500mm

span, this means that the deflection threshold ranges from 34mm to 1113mm, which is very wide indeed. By contrast, for the columns, with height $L$=3750mm, the $ds_4$ deflection threshold is smaller and ranges from 39mm to 268mm. Overall, the $ds_1$ deflection threshold is smaller for slabs (i.e., threshold ranges from 0mm to 10mm with 90% probability) than for columns (i.e., range from 0mm to 28mm with 90% probability). It is clear that the deflection response of RC structures under heating is not at all well known, and that there is considerable disagreement amongst the experts who have been elicited during the current study.

## 5.2   FIRE INTENSITY-TO-STRUCTURAL RESPONSE RELATIONSHIPS

For each fire intensity value, beta and lognormal cumulative distributions are fit to the cumulative distributions of spalling and residual capacity, and deflection and peak rebar temperature, respectively. The resulting CDFs (depicted in Fig. 7), based on Cooke's weighting scheme follow the expected order, e.g. the CDF of deflection for $T = 30$min falls to the left of the other two curves, indicating the smallest overall level of deflection. Similarly, the CDF of the deflection at $T = 90$min appears to the right of the other two, indicating a larger overall percentage of deflection. The mean structural response levels for the three damage states and the width of the 90% confidence intervals of each distribution are also presented in Table 7 for slabs and Table 8 for columns.

### 5.2.1   Comments on the weighting method

The effect of the weighting scheme on the CDFs of the structural response parameters for given levels of fire intensity is discussed in this section. In 11 out of the 24 obtained CDFs, the uncertainty around the structural response levels for a given fire intensity appears to be wider when the experts' opinions are weighted the same. By comparing the width of the 90% interval, the best experts appear to be able to provide cogent uncertainties on the peak rebar temperatures of both slabs and columns for all three intensity levels. They are also able to rationalize the uncertainty around residual capacity for slabs.

However, the uncertainty produced by the two weighting schemes appears to be almost identical when considering the residual capacity of columns for all three $FIM$ levels. This may be due to the assumption that the residual capacity of a concrete slab in sagging depends primarily on damage to the steel reinforcement; the latter is reasonably easily quantified, whereas the residual capacity of a concrete column in compression depends to a great extent on damage to the concrete, which is both more difficult to quantify and more variable compared with fire damage to steel reinforcement. This indicates that the best-performing experts judge this uncertainty is not well-constrained and that more research is required to better understand the factors influencing fire-induced damage in columns.

With regards to spalling, the best-performing experts constrain the uncertainty in spalling of both slabs and columns for $FIM = 90$min, but appear find it difficult to similarly limit the uncertainty in spalling for any other $FIM$ level. This may reflect a view amongst the experts that spalling is reasonably assured for more severe fires, whereas for less severe fires the likelihood of spalling is comparatively unknown. This is somewhat surprising

given that when concrete elements are exposed standard fire curves, peak rebar temperature spalling occurs relatively early on (within the first 20 mins) [29]. For fires with the same lower *FIM* level that have a greater overall duration (i.e. a long cool fire compared to a short hot fire of the same equivalent area), the level of spalling is unknown, but could be assumed to be less severe due to lower thermally induced strains.

Finally, the uncertainty around the relative deflection is systematically larger for Cooke's method than for the equal weighting scheme. This means that the best-performing experts believe that for a given *FIM* level the deflection of the slabs as well as of the columns is expected to be lower than for the whole group. For example, for *FIM* = 30min the experts feel that there is a 90% probability that the deflection of the slab ranges from 0.4mm to 6.4mm according as Cooke's weighting scheme, which is significantly lower than the 0.7mm to 45mm deflection predicted by applying equal weighting.

### 5.2.2 *Comments on SRMs*

The uncertainty in peak rebar temperature, residual capacity, and deflection increase as the fire intensity measure (*FIM*) levels increase, with the greatest uncertainties for $T$ = 90min, and smallest for $T$ = 30min. This suggests a profound uncertainty associated with the likelihood of spalling amongst the experts; an uncertainty that is clearly borne out within the concrete spalling research community [30]. Uncertainty increases as FIMs increases for two reasons. First, concrete has many complex thermal and mechanical relationships at high temperatures (i.e. above 300°C). Complexities such as movement of moisture, differential thermal expansion of aggregates and cement paste, overall thermal expansion of elements and frames, restraint forces and thermal stresses, cause considerable uncertainty as heating increases. Second, the *FIM* used is a time equivalence of a real fire (see Fig. 4). This means that a short, hot fire and a long, cool fire that have the same areas under the real fire curve, could have the same time equivalent *FIM* when the definition of *FIM* used herein is applied. However, due to the material properties of concrete, and in particular its low thermal conductivity, a long cool fire will create more uniform temperatures and, on average, higher temperatures within the concrete than a short, hot fire; albeit with less risk of spalling according to the available literature [31] .

More intense fires (i.e. $T$ = 90mins) will increase the proportion of concrete at higher temperatures and increase the level of complexities, and thus the uncertainty in predicting the response and damage.

The comparison of the results for slabs and columns (see Fig. 7i-m and Tables 7 and 8) shows that the developed intensity-to-response relationships for slabs are significantly different from those for columns. This contradicts observations regarding the damage thresholds, where most *SRM*-to-*DS* relationships are seen to be similar for slabs and columns.

A closer look at the differences between these relationships serves to identify a systematic difference in the means or uncertainties. For instance, the CDFs (see Fig. 7m) for the peak rebar temperature for slabs are shifted to the right of their counterparts for columns for all three fire intensity levels. This indicates that for the same fire intensity level, slabs are on average expected to suffer higher overall peak rebar temperature than columns.
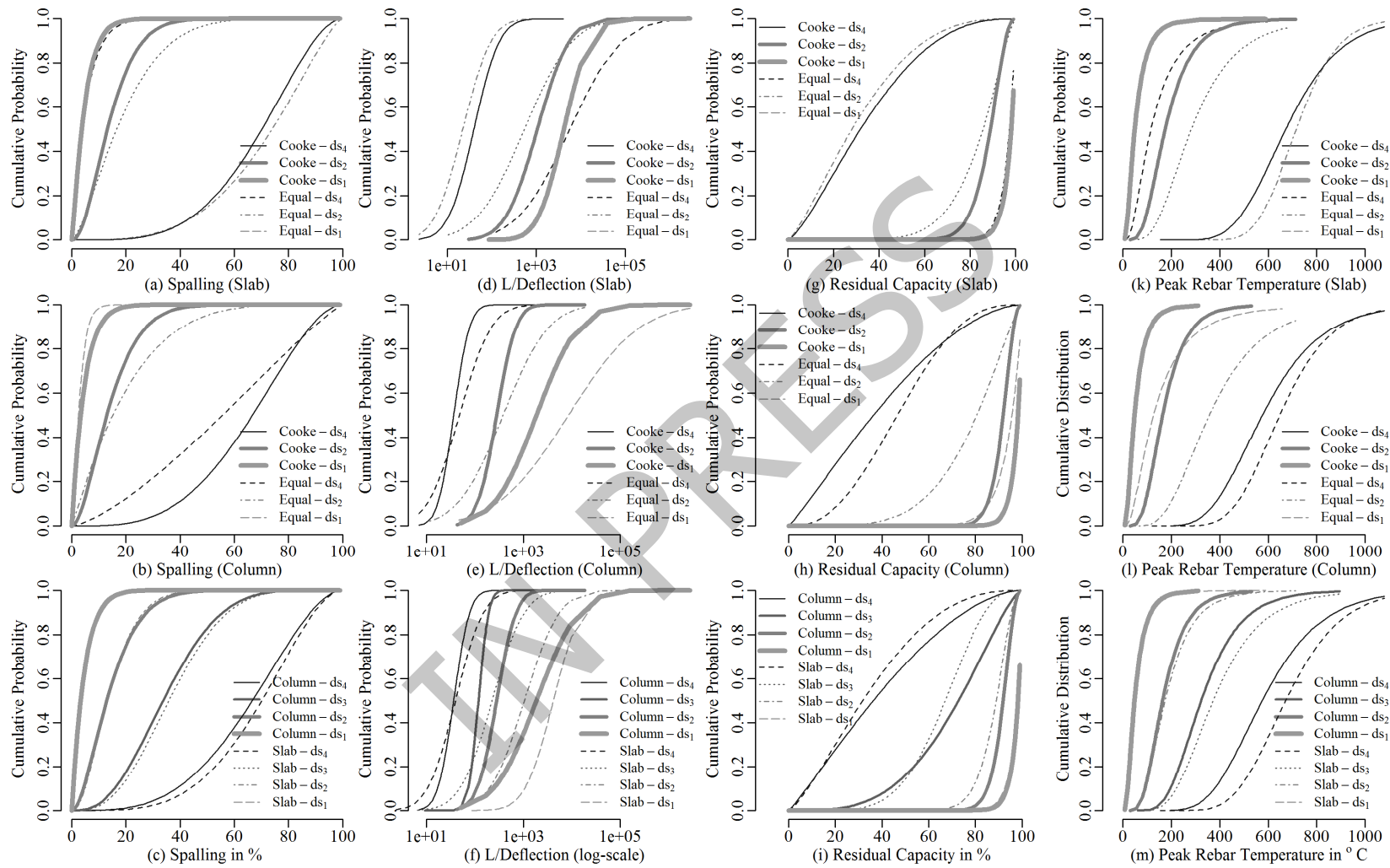
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Fig. 6: *SRM*-to-*DS* functions for columns and slabs using Cooke weights and equal weights.

Table 3: Fire damage scale for the open-plan, mid-rise, cast-in-place RC buildings.

| C | Surface Appearance of Concrete | | | Description | Element | Spalling | | L/Deflection | | Residual Capacity | | Peak Rebar Temp. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | *Condition of plaster/ finish* | *Colour* | *Crazing* | | | $\alpha$ | $\beta$ | $\lambda$ | $\zeta$ | $\alpha$ | $\beta$ | $\lambda$ | $\zeta$ |
| $ds_0$ | Unaffected or beyond extent of fire | | | | - | - | - | - | - | - | - | - | - |
| $ds_1$ | Some Peeling | Normal | Slight | Damage primarily cosmetic in nature, which does not impact on the design or repair of the structural fabric of RC buildings. | Slab | 1.19 | 23.90 | 8.34 | 1.07 | 24.97 | 0.79 | 3.89 | 0.71 |
| | | | | | Col. | 1.00 | 21.35 | 7.53 | 1.60 | 30.45 | 0.84 | 3.88 | 0.68 |
| $ds_2$ | Substantial Loss | Pink/ Red | Moderate | A small amount of damage has been experienced by the element to the effect that some small scale remedial action is required to enhance the element's remaining ability to perform its structural functions. | Slab | 2.33 | 13.64 | 6.96 | 1.31 | 20.36 | 2.83 | 5.16 | 0.49 |
| | | | | | Col. | 1.96 | 11.41 | 5.64 | 0.73 | 34.47 | 3.32 | 5.10 | 0.46 |
| $ds_3$ | Total Loss | Pink/Red Whitish grey | Extensive | The element has experienced a significant, but not catastrophic, amount of damage to the effect that, with significant remedial action, it can be reinstated to perform its structural functions. | Slab | 3.35 | 5.74 | 5.42 | 1.21 | 6.61 | 3.40 | 5.92 | 0.41 |
| | | | | | Col. | 3.09 | 5.77 | 4.77 | 0.38 | 3.53 | 1.52 | 5.78 | 0.39 |
| $ds_4$ | Destroyed | Whitish grey | Surface Lost | The damage cause by the fire is so extensive that it is no longer viable to repair and reuse the element and replacing the element with a new element is the only option. The building has not suffered a disproportionate collapse. | Slab | 4.25 | 1.99 | 3.68 | 1.05 | 1.37 | 2.60 | 6.51 | 0.26 |
| | | | | | Col. | 3.63 | 1.98 | 3.59 | 0.60 | 1.12 | 1.75 | 6.36 | 0.33 |

Table 4: Mean, 5% and 95% probability of exceedance of the four SRMs thresholds for each damage state.

| DS | Element | 5% | 95% | mean | 5% | 95% | mean | 5% | 95% | mean | 5% | 95% | mean |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Spalling | | | L/Deflection | | | Residual Capacity | | | Peak Rebar Temp. | | |
| $ds_0$ | Slab | - | - | - | - | - | - | - | - | - | - | - | - |
| | Col. | - | - | - | - | - | - | - | - | - | - | - | - |
| $ds_1$ | Slab | 0.38 | 13.06 | 4.75 | 722.13 | 24068.88 | 7357.69 | 90.16 | 99.92 | 0.97 | 15.28 | 157.91 | 63.20 |
| | Col. | 0.24 | 13.11 | 4.48 | 134.05 | 25956.29 | 6716.82 | 91.54 | 99.91 | 0.97 | 15.93 | 147.68 | 60.99 |
| $ds_2$ | Slab | 3.42 | 30.91 | 14.62 | 121.74 | 9109.59 | 2489.40 | 75.26 | 96.62 | 0.88 | 77.82 | 391.55 | 196.93 |
| | Col. | 2.83 | 32.62 | 14.68 | 84.95 | 934.54 | 367.50 | 82.67 | 97.26 | 0.91 | 77.17 | 348.30 | 182.08 |
| $ds_3$ | Slab | 13.64 | 63.55 | 36.87 | 31.15 | 1650.29 | 469.61 | 40.71 | 87.55 | 0.66 | 190.21 | 730.63 | 405.32 |
| | Col. | 12.02 | 61.82 | 34.88 | 63.07 | 222.46 | 127.47 | 35.08 | 95.33 | 0.70 | 169.87 | 615.82 | 349.20 |
| $ds_4$ | Slab | 36.38 | 92.79 | 68.07 | 7.07 | 221.93 | 68.57 | 4.87 | 73.77 | 0.35 | 434.97 | 1037.84 | 695.77 |
| | Col. | 31.27 | 91.90 | 64.78 | 13.60 | 97.46 | 43.55 | 4.12 | 83.42 | 0.39 | 335.76 | 986.07 | 607.09 |

Table 5: Response-to-Damage functions for slabs using Cooke weights and equal weights for expert pooling.

| SR | DS | Cooke weights | | Equal weights | | | | Diff. (in 1-$X_{Equal}/X_{Cooke}$ %) | |
|---|---|---|---|---|---|---|---|---|---|
| | | mean | 90% range | Q5% | Q95% | mean | 90% range | 90% range | mean |
| Spalling (in %) | $ds_4$ | 68 | 56 | 21 | 94 | 62 | 73 | -30% | 9% |
| | $ds_2$ | 15 | 27 | 3 | 58 | 25 | 55 | -99% | -71% |
| | $ds_1$ | 5 | 13 | 0 | 18 | 6 | 18 | -40% | -27% |
| Relative Deflection L/Deflection | $ds_4$ | 69 | 215 | 3 | 353 | 94 | 350 | -63% | -37% |
| | $ds_2$ | 2489 | 8988 | 19 | 3936 | 1018 | 3918 | 56% | 59% |
| | $ds_1$ | 7358 | 23347 | 86 | 140512 | 43635 | 140426 | -501% | -493% |
| Residual Capacity (in %) | $ds_4$ | 35 | 69 | 7 | 88 | 45 | 81 | -18% | -30% |
| | $ds_2$ | 88 | 21 | 72 | 97 | 86 | 25 | -15% | 1% |
| | $ds_1$ | 97 | 10 | 91 | 99 | 96 | 9 | 9% | 1% |
| Peak Rebar Temp (in ºC) | $ds_4$ | 696 | 603 | 431 | 1007 | 681 | 576 | 4% | 2% |
| | $ds_2$ | 197 | 314 | 148 | 801 | 393 | 653 | -108% | -99% |
| | $ds_1$ | 63 | 143 | 36 | 455 | 172 | 419 | -194% | -172% |

Table 6: Response-to-Damage functions for columns using Cooke weights and equal weights for expert pooling.

| SR | DS | Cooke weights | | Equal weights | | | | Diff. (in 1-$X_{Equal}/X_{Cooke}$ %) | | (in 1-$X_{Column}/X_{Slab}$ %) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | mean | 90% range | Q5% | Q95% | mean | 90% range | 90% range | mean | 90% range | mean |
| Spalling (in %) | $ds_4$ | 65 | 61 | 10 | 94 | 54 | 84 | -38% | 16% | -7% | 5% |
| | $ds_2$ | 15 | 30 | 1 | 49 | 19 | 48 | -62% | -29% | -8% | 0% |
| | $ds_1$ | 4 | 13 | 0 | 8 | 3 | 7 | 44% | 34% | -1% | 6% |
| Relative Deflection L/Deflection | $ds_4$ | 44 | 84 | 7 | 346 | 99 | 339 | -305% | -126% | 61% | 36% |
| | $ds_2$ | 367 | 850 | 23 | 6414 | 1662 | 6391 | -652% | -352% | 91% | 85% |
| | $ds_1$ | 6717 | 25822 | 103 | 702425 | 311114 | 702321 | -2620% | -4532% | -11% | 9% |
| Residual Capacity (in %) | $ds_4$ | 39 | 79 | 16 | 79 | 47 | 63 | 21% | -20% | -15% | -13% |
| | $ds_2$ | 91 | 15 | 48 | 97 | 78 | 50 | -240% | 15% | 32% | -4% |
| | $ds_1$ | 97 | 8 | 82 | 100 | 94 | 18 | -116% | 4% | 14% | 0% |
| Peak Rebar Temp (in ºC) | $ds_4$ | 607 | 650 | 427 | 983 | 669 | 557 | 14% | -10% | -8% | 13% |
| | $ds_2$ | 182 | 271 | 164 | 774 | 398 | 610 | -125% | -119% | 14% | 8% |
| | $ds_1$ | 61 | 132 | 35 | 455 | 171 | 420 | -219% | -181% | 8% | 4% |

Tables 7 and 8 and Fig. 7 show the relationships between the structural response and the intensity of the fire, and show that the experts expect higher relative mean deflections with higher *FIM*s, but are less certain about this structural response, for columns as compared to slabs for the same *FIM*. The experts also expect columns to lose more strength than slabs for the same *FIM*, but are again less certain regarding the response of the columns than the slabs. Finally, they believe that peak rebar temperatures in columns are on average lower than for slabs for the same *FIM*. However, the comparison of the level of uncertainty around the peak rebar temperature of slabs and columns for a given FIM does not show any trend. Overall, there is generally less certainty amongst experts on the response of columns, and hence columns are thus considered more critical for further research.

Contrary to the other *SRM*s, the experts appeared to be more certain about the spalling response of columns compared to slabs, and in general the experts expect spalling to occur for both slabs and columns. This is important, since the majority of design codes [3] assume that spalling will not occur, meaning that either design codes ought to be updated or that a better understanding of how to prevent spalling from occurring and its consequences for structural response ought to be developed [29].

## 5.3    FIRE INTENSITY-TO-STRUCTURAL RESPONSE RELATIONSHIPS – FRAGILITY CURVES

Fragility curves are obtained by coupling the intensity-to-structural response and response-to-damage relationships using the Monte Carlo procedure outlined in Section 3.7. The probability of the damage sustained by the studied generic RC building reaching or exceeding each of the three considered damage states is numerically determined for *FIM* = 30min, 60min and 90min. For each *FIM* level, a structural response level is randomly generated from the appropriate *FIM*-to-*SRM* relationship (see Fig. 5). For this latter level, the damage state sustained by the building is then randomly generated from the appropriate *SRM*-to-DS relationship (see Fig. 5). 1,000 such iterations are performed, and the probabilities of reaching or exceeding $ds_1$, $ds_2$ and $ds_4$ are estimated for the three FIM levels. The three points are then connected with a piecewise multi-linear curve (see Fig. 8 and Fig. 9), with which to assess:

a.  whether slabs are considered more vulnerable than columns, for an open-plan mid-rise RC building;
b.  if the shape of the fragility curves for a given structural element depends on the chosen structural response measure; and
c.  if a specific set of fragility curves can be identified as more reliable for possible use in quantifying the structural fire fragility of mid-rise cast-in-place RC concrete frames.

In general, when comparing fragility curves corresponding to a given damage state for two structural elements, the curve that lies most to the right indicates the least fragile element. Comparison of the fragility curves corresponding to the three damage states for slabs and columns in Fig. 8 indicates that whether slabs or columns are deemed to be relatively more or less fragile depends on the structural response measure used to construct the fragility curves. If spalling or residual capacity is used, then columns appear to be more fragile than slabs. The opposite is true if deflection or peak rebar temperature is adopted as response measures.

Fig. 8 also shows that the greatest risk of damage is deemed to be from spalling, since the probabilities of exceeding each of the damage states is higher for spalling as compared to other SRMs, at the three levels of *FIM* assessed in this elicitation. It should also be noted that with spalling as the SRM, for a *FIM* of 30 minutes, the probability of exceeding $ds_2$ is around 60% for both slabs and columns. This means that the experts expect to see some spalling in the *majority* of fires in modern cast-in-place concrete buildings. This is significant for the reasons already noted, i.e. that in general spalling is not considered by structural designers, except in rare cases such as tunnels.

Fig. 9 shows substantial differences in the shape of the fragility curves for the four response measures, depending on the damage states being assessed. The largest differences are in the values of the fragility curves corresponding to $ds_1$ and $ds_2$ for $T =$ 30min. For instance, the probability that a column of an open plan RC building will sustain damage equal to or above $ds_2$ if exposed to a fire with intensity $T = 30$min ranges from 3.5% to 62.5% for deflection and spalling structural response measures, respectively. By contrast, the differences in the curves corresponding to $ds_4$ for all four response measures reduce notably, indicating better agreement amongst experts as to the likelihood of severe damage under this FIM. This picture is reversed for a fire intensity of $T = 90$min. In this case the differences in the fragility curves for the four response measures are small, indicating that, irrespective of the response measure, the structural element is almost certainly damaged after 90min of equivalent fire exposure. For example, the probability that a column will sustain damage equal or above $ds_1$, if exposed to fire intensity $T = 90$min, ranges from 93% to 100%.

The probability that a structural element will sustain damage $ds_2$ or above given 90min of exposure in the fire is also very high, although the uncertainty due to the selection of response measure is higher. For example, the probability that a slab will sustain damage equal or above $ds_2$ if exposed to fire intensity $T$=90min ranges from 86% to 100%.

Finally, the probability that a structural element will sustain damage $ds_4$ or above is notably influenced by the structural response parameter used in the construction of the curve. For example, the probability that a column will sustain damage equal or above $ds_4$ if exposed to fire intensity $T = 90$min ranges from 37% to 80%. This shows that either the knowledge of how structures are damaged, rather than failing, due to a fire is low and more research is required, or that the specific FIM of equivalent areas of fire above 150°C that has been chosen in the current study is inappropriate (or insufficient) for evaluating the structural fire fragility of RC structures.

## 6  CONCLUSIONS

The constructive and successful expert elicitation pursued in the present work leads to the quantification of the uncertainty in *SRM*-to-*DS* and *FIM*-to-*SRM* relationships for slabs and columns in a generic mid-rise open plan cast-in-place RC frame building, and the subsequent endeavour to construct fire fragility curves for this building class by coupling these two relationships. From the elicitation, analysis, and construction of fire fragility curves, the following can be concluded:

a. An initial fire damage scale (see Table 3 and Table 4) for slabs and columns of the selected building class, which can be used to aid design decision and the analytical fragility assessment, has been developed. The scale uses the four damage states developed by the Concrete Society [4] and uses the expert opinions pooled with Cooke's model to quantify spalling, deflection, residual capacity and peak rebar temperature thresholds for each damage state. It is evident that international experts in this area judge that the uncertainties around certain damage thresholds are difficult to delimit, especially for the extreme damage state $ds_4$, and that more experimental, analytical, and judgment based research is needed to better understand the relationship of these response thresholds with the observed damage.
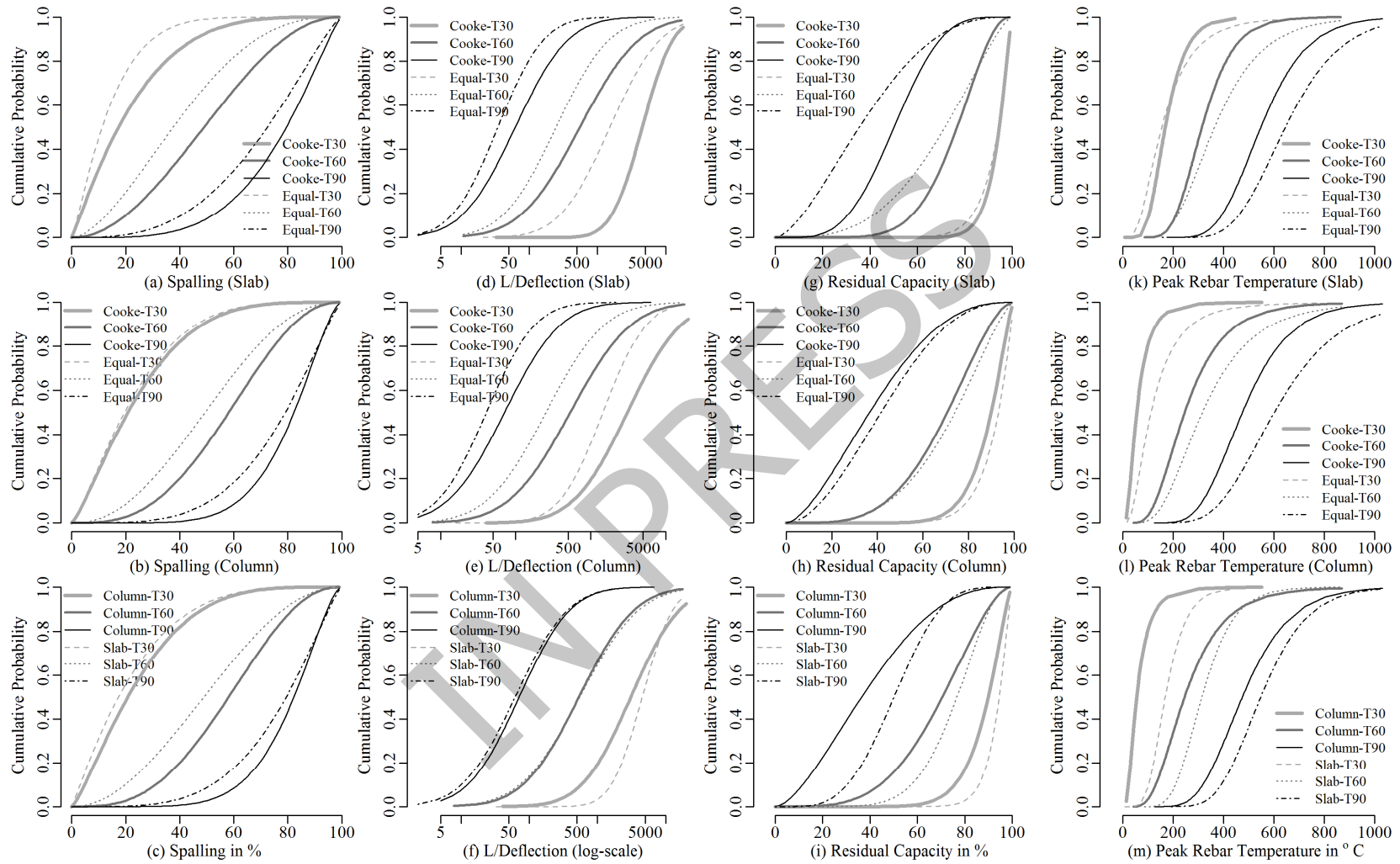
Fig. 7: *FIM*-to-*SRM* functions for columns and slabs using Cooke weights and equal weights.

Table 7: *FIM*-to-*SRM* functions for slabs using Cooke weights and equal weights for expert pooling.

| SR | FIM | Cooke weights | | | | | | | | Equal weights | | | | Diff. (in 1-$X_{Equal}$/$X_{Cooke}$ %) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Beta | | Lognormal | | Q5% | Q95% | mean | 90% range | Q5% | Q95% | mean | 90% range | 90% Range | mean |
| | | *A* | *β* | *λ* | *ζ* | | | | | | | | | | |
| Spalling (in %) | *T*=30min | 1.07 | 3.96 | - | - | 2 | 54 | 21 | 53 | 1 | 36 | 14 | 35 | 34% | 33% |
| | *T*=60min | 2.13 | 2.21 | - | - | 14 | 85 | 49 | 71 | 8 | 76 | 39 | 68 | 4% | 20% |
| | *T*=90min | 4.15 | 1.30 | - | - | 44 | 97 | 76 | 54 | 32 | 96 | 69 | 64 | -19% | 9% |
| Relative Deflection L/Deflection | *T*=30min | - | - | 8.42 | 0.82 | 1172 | 16254 | 6327 | 11099 | 167 | 13670 | 3704 | 13503 | -22% | 41% |
| | *T*=60min | - | - | 6.31 | 1.58 | 41 | 7402 | 1918 | 7362 | 24 | 2551 | 676 | 2527 | 66% | 65% |
| | *T*=90min | - | - | 4.13 | 1.41 | 6 | 633 | 168 | 627 | 5 | 258 | 73 | 253 | 60% | 57% |
| Residual Capacity (in %) | *T*=30min | 19.72 | 1.44 | - | - | 83 | 99 | 93 | 17 | 78 | 99 | 92 | 21 | -24% | 1% |
| | *T*=60min | 7.89 | 2.62 | - | - | 52 | 93 | 75 | 41 | 33 | 95 | 69 | 63 | -54% | 8% |
| | *T*=90min | 5.13 | 5.11 | - | - | 25 | 75 | 50 | 49 | 7 | 75 | 38 | 68 | -39% | 24% |
| Peak Rebar Temp (in ºC) | *T*=30min | - | - | 5.12 | 0.39 | 89 | 317 | 181 | 228 | 57 | 416 | 185 | 359 | -57% | -2% |
| | *T*=60min | - | - | 5.75 | 0.32 | 186 | 527 | 329 | 341 | 186 | 711 | 395 | 526 | -54% | -20% |
| | *T*=90min | - | - | 6.31 | 0.26 | 355 | 844 | 567 | 489 | 415 | 1001 | 668 | 586 | -20% | -18% |

Table 8: *FIM*-to-*SRM* functions for columns using Cooke weights and equal weights for expert pooling.

| SR | FIM | Cooke weights | | | | | | | | Equal weights | | | | Diff. (in 1-$X_{Equal}$/$X_{Cooke}$ %) | | Diff. (in 1-$X_{Column}$/$X_{Slab}$ %) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Beta | | Lognormal | | Q5% | Q95% | mean | 90% range | Q5% | Q95% | mean | 90% range | 90% range | mean | 90% range | mean |
| | | α | β | λ | Z | | | | | | | | | | | | |
| Spalling (in %) | T=30min | 1.30 | 4.07 | - | - | 3 | 57 | 24 | 54 | 3 | 55 | 23 | 52 | 4% | 4% | -2% | -14% |
| | T=60min | 3.31 | 2.47 | - | - | 25 | 87 | 57 | 62 | 16 | 83 | 49 | 68 | -10% | 14% | 13% | -16% |
| | T=90min | 6.92 | 1.74 | - | - | 55 | 97 | 80 | 41 | 43 | 97 | 76 | 54 | -32% | 5% | 24% | -5% |
| Relative Deflection L/Deflection | T=30min | - | - | 8.03 | 1.30 | 361 | 26354 | 7219 | 25994 | 260 | 8176 | 2526 | 7915 | 70% | 65% | -134% | -14% |
| | T=60min | - | - | 6.28 | 1.48 | 47 | 6106 | 1599 | 6059 | 24 | 2633 | 697 | 2608 | 57% | 56% | 18% | 17% |
| | T=90min | - | - | 4.26 | 1.36 | 8 | 667 | 180 | 660 | 6 | 285 | 82 | 279 | 58% | 54% | -5% | -7% |
| Residual Capacity (in %) | T=30min | 11.35 | 1.56 | - | - | 71 | 98 | 88 | 28 | 75 | 99 | 91 | 25 | 11% | -3% | -65% | 5% |
| | T=60min | 4.28 | 1.97 | - | - | 37 | 93 | 69 | 56 | 37 | 96 | 71 | 58 | -4% | -3% | -37% | 8% |
| | T=90min | 1.67 | 2.68 | - | - | 7 | 76 | 38 | 68 | 11 | 78 | 43 | 67 | 1% | -13% | -39% | 24% |
| Peak Rebar Temp (in ºC) | T=30min | - | - | 4.00 | 0.67 | 17 | 172 | 69 | 155 | 35 | 357 | 143 | 322 | -108% | -107% | 32% | 62% |
| | T=60min | - | - | 5.50 | 0.49 | 109 | 550 | 276 | 441 | 139 | 715 | 357 | 576 | -31% | -29% | -29% | 16% |
| | T=90min | - | - | 6.16 | 0.31 | 283 | 793 | 498 | 510 | 348 | 1036 | 635 | 688 | -35% | -28% | -4% | 12% |
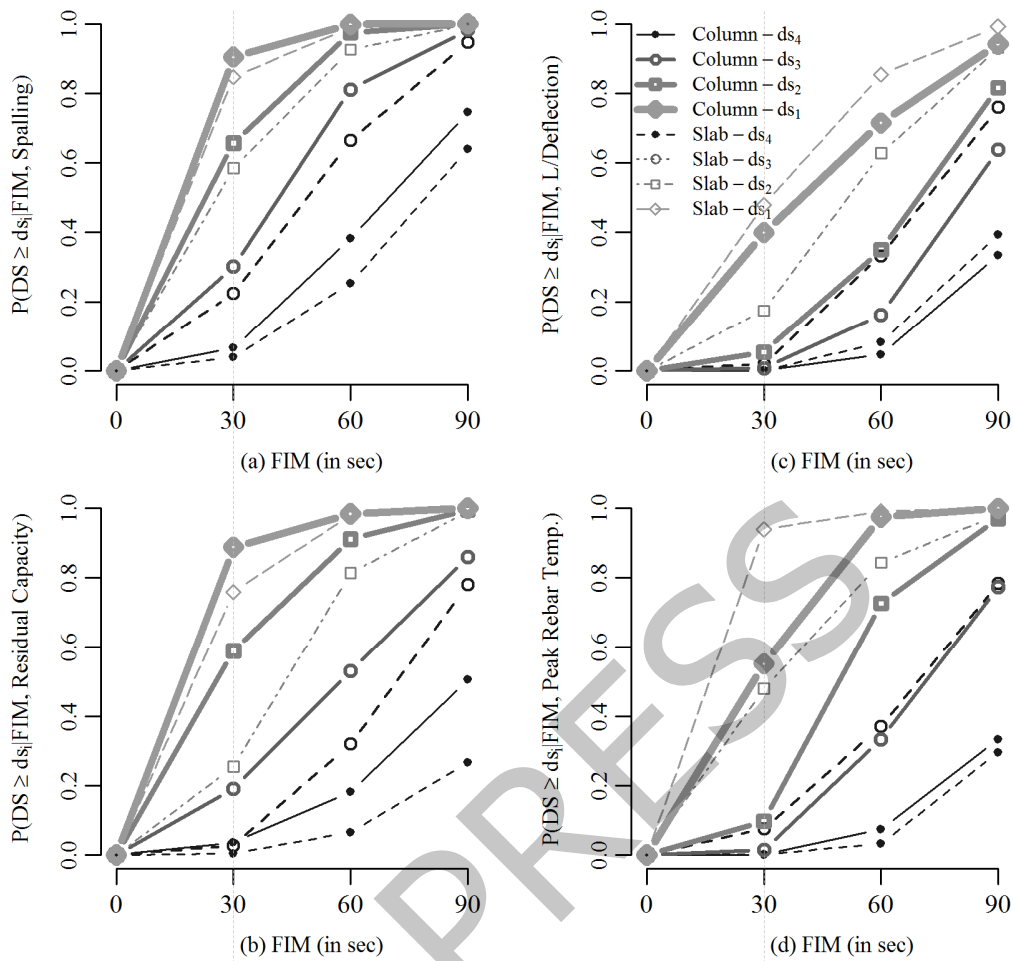
Fig. 8: Fragility curves corresponding to three damage states for (a) spalling, (b) residual capacity, (c) deflection and (d) peak rebar temperature constructed for slabs and columns.

**b.** Increases in the fire intensity substantially increase uncertainties around deflections, residual capacity, and peak rebar temperature. The results of the expert elicitation have clearly highlighted profound uncertainties within the structural fire engineering community as to the effects of fires on systems of RC structural elements (real buildings) as compared with isolated structural elements exposed to fires in furnace tests (regulatory/compliance tests). A great deal of additional research is needed before the full-structure response of concrete buildings in fire can be predicted with any real confidence.

c. The results of the expert elicitation show that even for relatively low fire intensity levels (i.e. FIM = 30mins) experts *expect spalling* to occur and to cause considerable damage; columns are deemed to be more susceptible than slabs. Since most design codes assume that spalling does not occur, it is recommended that either spalling of concrete in fire is integrated into structural analysis procedures, or that concrete mixes are designed so as to minimise the risk of spalling for every design in which damage from spalling must be avoided.

The next steps in research in this area will be the determination of thresholds for other types of structural elements (i.e. walls and beams), and the further development of the relatively rudimentary scales presented here. Research focus can be concentrated on the damage assessment of individual members and on heading towards a global damage scale that can be used to assess the overall system performance of a building. This would allow possible losses due to fire to be more credibly and quantitatively assessed.

## ACKNOWLEDGEMENTS

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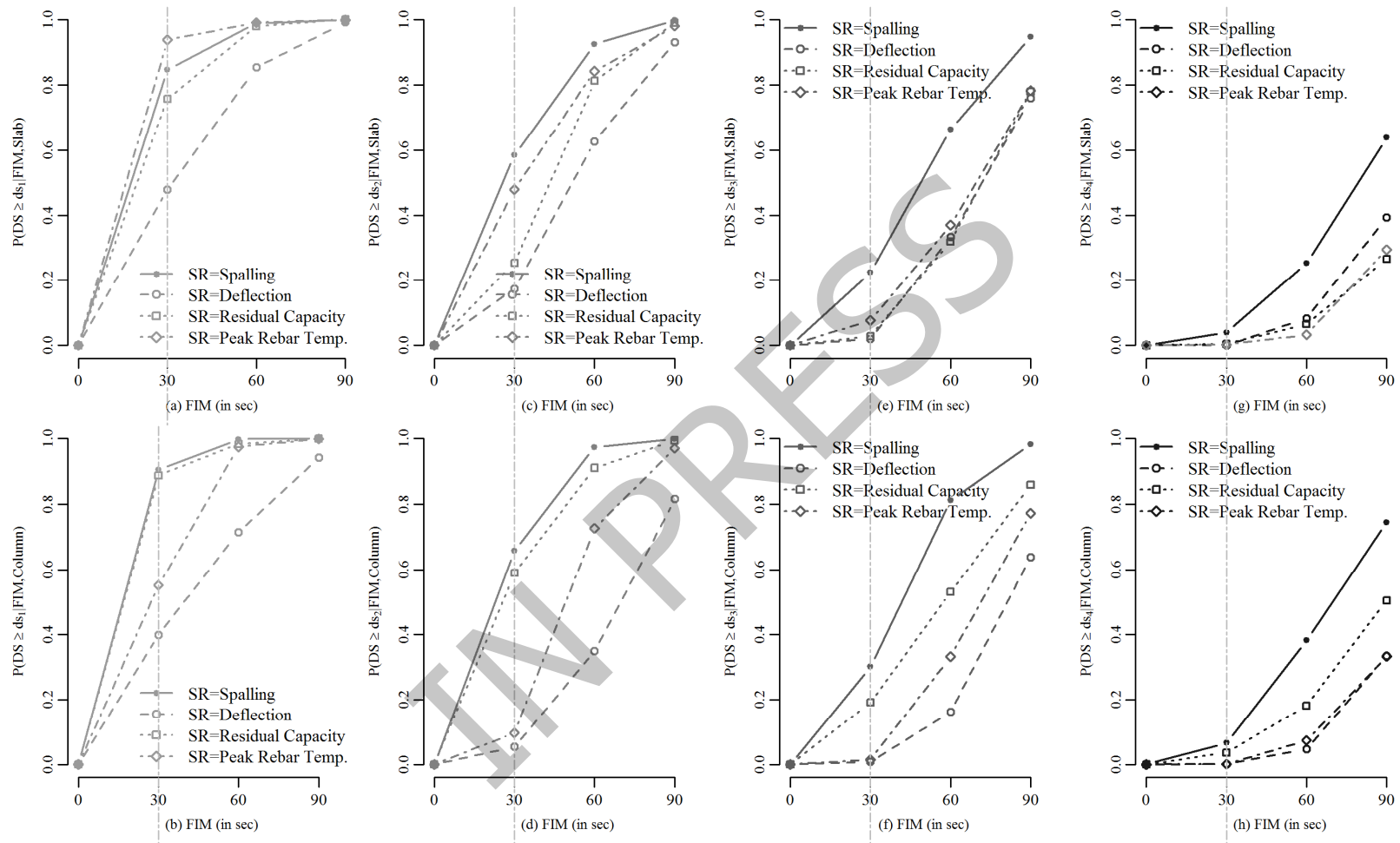
Fig. 9: Effect of structural response measure in fragility curves for slabs and columns.

APPENDIX

This appendix contains the seed and target questions asked to the participants. Examples of individual expert's responses are included for both seed and target questions.

A.1 SEED QUESTIONS

The experts were asked to provide for each seed question three values corresponding to the 5%, 50% and 95% percentile. The sixteen seed questions which were used in the expert elicitation workshop with their correct answers are presented in what follows. Two examples of experts' individual responses are also presented in what follows.

A.1.1 Seed Question 1

Can you estimate the combined direct and indirect losses suffered by UK buildings exposed to fire every year as a percentage of the Gross Domestic Product (GDP)?

- Direct loss: Direct material loss to the structural and non-structural elements of buildings and their contents.
- Indirect loss: Consequent losses due to loss of production, profit or employment.

Response: 0.21% [1]

A.1.2 Seed Question 2

A Finish study including data from 2,959 fires, collected between 1996-2002, found that 23% of ignitions started in the living rooms of apartment houses.

What is the % of ignitions which started in kitchens?

Response: 24.5% [2]

A.1.3 Seed Question 3

Following the introduction of a New Zealand national building code in 1991, mandatory performance-based design for fire safety was adopted in 1993, with a set of new prescriptive documents for use in 'acceptable solutions'. A survey of approving authorities was carried out in 1998 to quantify the major changes during the first five years of implementation of the new code. A considerable number of designers were found to be using the freedom of the new legislation to make significant departures from the prescriptive documents. Also, the new code environment created a sudden demand for educated professionals for design, review, and regulation.
According to this survey (i.e. in 1998), what percentage of all designs at the time were being prepared by poorly qualified consultants attempting to do fire engineering?

Response: 30% [3]

A.1.4 Seed Question 4

At 9:40 am on 28 July 1945, in thick fog a B-25 bomber accidentally struck the north face of the 79th floor of the Empire State Building, then the tallest building in the world. The plane ripped an 18-foot-wide 20-foot-high hole in the outer wall of the building. The force of the impact propelled one of the two engines through the building and out the opposite wall; it fell 900ft and struck the 12th-story roof of a building across 32nd Street, where it ignited a fire. The other engine fell into an elevator shaft of the Empire State Building and down to the sub-cellar. In the Empire State Building on the impact floor, fuel from the ruptured tanks ignited.

How long did it take to extinguish the fire? Please state your units of time!

Response: 40min [4]

A.1.5 Seed Question 5

In general, fires which cause damage to buildings have three distinct stages: the growth period, the flashover and decay. Whether a fire develops to its flashover stage (in other words whether it becomes significant for the examined building) depends on a number of variables (e.g. fire load, ventilation, presence of detection systems or presence of suppression systems).

Given that for every 100 ignitions occurring in schools only 2 fires are expected to develop to the flashover stage, how many flashovers are expected for 100 ignitions occurring in dwellings?

Response: 10 ignitions [3]

A.1.6 Seed Question 6

The compressive strength of normal strength concrete (C35) is influenced by heating and cooling regimes. Let's assume a large number of cylinders, with dimensions: 100mm diameter and 200mm height, are exposed to different heating regimes for 3 hours and then they are either water-cooled or air-cooled.

If the average loss of compressive strength for air-cooled cylinders heated in an oven at 300ºC is 12%, what is the average loss of compressive strength for water-cooled cylinders heated in an oven at 600ºC?

Response: 57% [5]

A.1.7 Seed Question 7

In the afternoon of 6th April 2000, a fire broke out on the 12th floor of a University building in Novi Sad (Serbia). The 13-storey building was built in 1962 (an illustration of a typical floor plan is shown in Fig. A.1). The fire spread rapidly from the 8th to the 13th floor, covering a total area of 2,400 m². The rapid vertical spread of the fire was attributed to:

- the lack of fire protection arrangements (e.g. lack of vertical fire resistant separating walls, non-insulated installation openings and staircases etc);
- the large amount of flammable materials, i.e. approximately 60 kg of wood/m$^2$;
- strong wind during the fire (13-15m/s); and
- hindered fire brigade action due to difficult access.

The duration of the fully developed fire in each floor was approximately between 25 and 35min. Severe damages had been noted in all six affected floors.
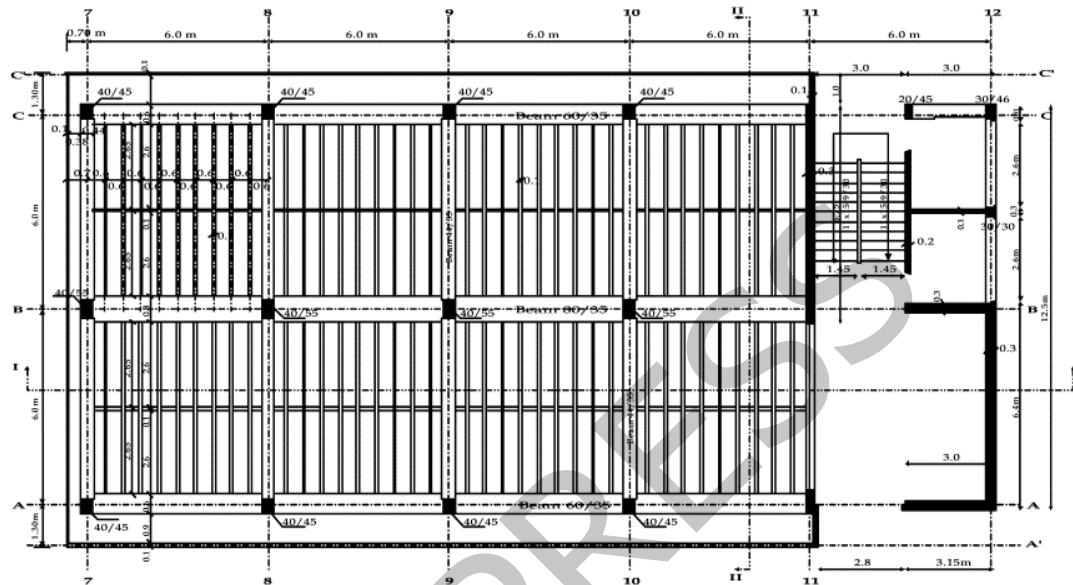


Fig.A.1: Plan of a typical floor of the University building in Novi Sad [6].

If the compressive strength of the concrete from columns not exposed to the fire is on average 55MPa, can you estimate the compressive strength of the columns which were exposed to the fire?

Response: 35 MPa [6]

A.1.8 Seed Question 8

Neves et al. [7] studied the mechanical properties of reinforcement steel bars (A400 NR, Φ 6 mm, Φ 12 mm, Φ 20 mm) and prestressing steel cables (central tendon: Φ 5.5mm). The length of all specimens was 23 cm.

Samples were placed, in batches of 10, inside a furnace. The furnace was set to reach a defined temperature, and once reached; it was maintained for 1h. This was considered to achieve a temperature uniformity tested specimens. After heating, specimens were air cooled and their residual tensile strength was measured.

The residual tensile strength of the reinforcement bars heated up to 700°C was found to be on average 20% lower than the tensile strength of the bars. Could you judge the average reduction in residual tensile strength of the pre-stressing steel samples heated up to 700°C?

3

$$\text{reduction in \%} = \left(1 - \frac{\text{Tensile strenght heated}}{\text{Tensile strength unheated}}\right) * 100\%$$

Response: 60% [7]

## A.1.9 Seed Question 9

McCaffrey et al. [8] studied the onset of the flashover. Tests were performed in order to examine the effect of room dimensions on the rate of energy required for a flashover, conservatively considered as the temperature rise at the upper part of the room equal to $\Delta T = 500$ degrees Celsius.

The following assumptions regarding the compartment were considered:
- The walls of the compartment had gypsum lining.
- The effective heat transfer coefficient through ceiling/walls is $0.0055$ k$W/m^2K^{-1}$
- The compartment had a single opening with ventilation factor:

$$Width_{Opening} Height_{Opening} \sqrt{Height_{Opening}} = 2 m^{5/2}$$

For the compartment's dimensions were equal to 2m width x 4m length x 2.4 m height, the rate of energy required for a flashover at the characteristic time $t_c = 1,000$ seconds is 450 k$W$.

What is the rate of energy (in k$W$) required for a flashover at $t_c = 1,000$ seconds for a compartment with dimensions 4m width x 8m length x 2.4 m height?

Response: 875 k$W$ [8]

## A.1.10 Seed Question 10

Kirby et al. [9] experimentally examined the behaviour of *"natural"* fires in large compartments. This required the construction of a large compartment and the use of 33 equally spaced wooden cribs in order to ensure uniform fuel load density ($20kg/m^2$ equivalent to $380.1$ MJ/$m^2$). The dimensions of the compartment and the position of the cribs are shown in Fig.A.2 and Fig.A.3.

The fire was intentionally started at the rear of the compartment and it rapidly (aprox. 17 min) engulfed the whole compartment (i.e. became a fully developed fire). After 20 min, the average temperature of the 11 thermocouples (see Figure 3) at the front of the compartment, near the opening (row #10, near the opening) was 1050°C and the average temperature at the rear of the compartment (row 2) is 36% lower:

$$difference\ in\ T\ after\ 20\ min = \left(1 - \frac{T_{rear}}{T_{front}}\right) * 100\% = 36\%$$

Can you judge the average temperature in the rear after 60min, if the average temperature in the front of the compartment is 600°C?
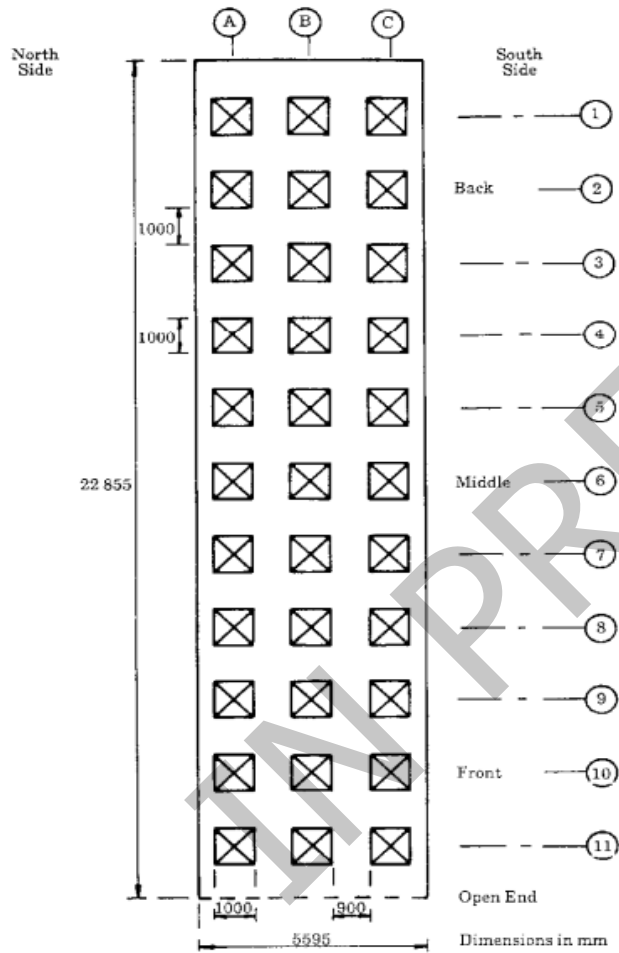
Response: 850°C [9]



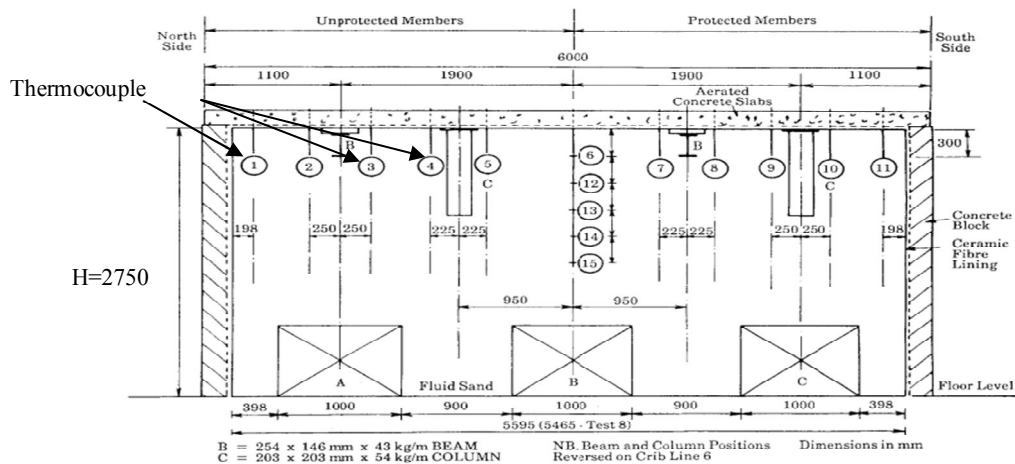Fig. A.2: Compartment plan showing the position of the cribs with the back and front measuring stations.

Fig. A.3 Section through the compartment cribs 2, and 10 showing the position of the monocouple locations.

## A.1.11 Seed Question 11

Stern-Gottfried et al. [10, 11] studied the temperature distribution of fully developed, post-flashover fires in a large compartment. They showed that there is heterogeneity in the temperature field due to the depth of the compartment relative to the position of the vents.

The study involved the statistical analyses of data collected from 8 Cardington tests. These tests were conducted in a compartment with dimensions 12m length x 12m width x 3m height. The load fuel packages were uniformly positioned across the floor. The ignition and the subsequent burning can also be considered uniform. Sixteen thermocouple trees consisting of 4 thermocouples each were placed on a uniform gird as presented in Fig.A.5. The fuel packages were either made or wood or wood and plastic (see Fig.A.4). The ventilation openings were either only on the front (F in **Error! Reference source not found.**) or the front and the back (FB in Fig.A.4). With regard to the lining, three types of insulation were considered:
  • Insulating (I in Fig.A.4)
  • Intermediate insulation (I+ Fig.A.4)
  • Highly Insulating (HI in Fig.A.4)

The statistical analyses of the compartment's temperature data for different fire times showed that when there is only one opening (i.e. test 1,2,3 and 8) the standard deviation of the temperature for the 120min of fire is on average 88ºC. What is the average standard deviation for the tests (4-7) with two openings?

| Test number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|---|
| Fuel type | W | W | W+P | W | W+P | W | W+P | W+P |
| Insulation type | I | HI | HI | HI | HI | I+ | I+ | I+ |
| Opening location | F | F | F | F+B | F+B | F+B | F+B | F |

Fig. A.4: Summary of test conditions in Cardington test (Stern-Gottfried et al., [10, 11]).
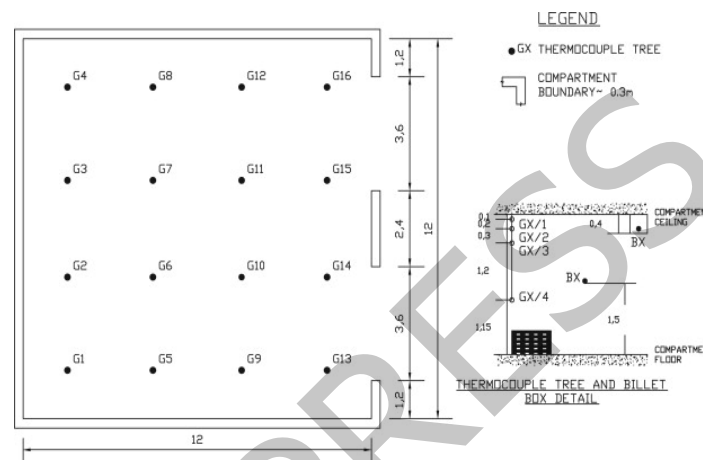
Response: 51°C [10, 11]



Fig.A.5 Experimental layout of Cardington Tests. Locations of the 16 thermocouple trees (each with 4 thermocouples in height) are noted as black dots (Stern-Gottfried et al., 2010).

A.1.12 Seed Question 12

The structural behaviour of a floor system was assessed during a NZ standard fire resistance test. The structural system comprises a composite floor with concrete slab on a profiled steel deck, supported on primary and secondary steel beams, designed to act compositely with the floor slab. In the example, only the slab and secondary beam elements are given, these being the relevant design components:
* normal weight concrete slab, 120mm thick on Diamond Hi-Bond.
* secondary beams are at 2.8m centres.
* secondary beam size, grade is 310UB40, Grade 300.
* secondary beam is composite with the floor slab.
* secondary beams are unprotected against fire.
* connections to secondary beams are WP30.
* beam span is 8.3m.
* dead load, G=2.4 kPa.
* basic live load, Q=2.5 kPa.
* the live load combination factor for the ultimate limit state is 0.4.

Please use your judgement to quantify what beam limiting temperature (in °C) would be determined for a beam in a simply supported condition, if NZS 3404 [Clause 11.5] were used?

Response: 711°C [12]

A.1.13 Seed Question 13

The residual stiffness of an RC beam (defined from the load vs deflection curves) exposed to fire is influenced by the thickness of the cover. Assuming that high strength concrete (C55) is used and the beam (see figure below) was tested under a standard time-temperature curve (ISO 834) for 90min (see Fig. A.6).

If there is a 40% reduction in stiffness for thickness cover 40mm, which is the reduction (in %) in stiffness for thickness cover 50mm?

$$\% \ reduction \ in \ stiffness \ after \ t \ min = \left(1 - \frac{Stiffness_t}{Stiffness_{Unheated}}\right) * 100\%$$
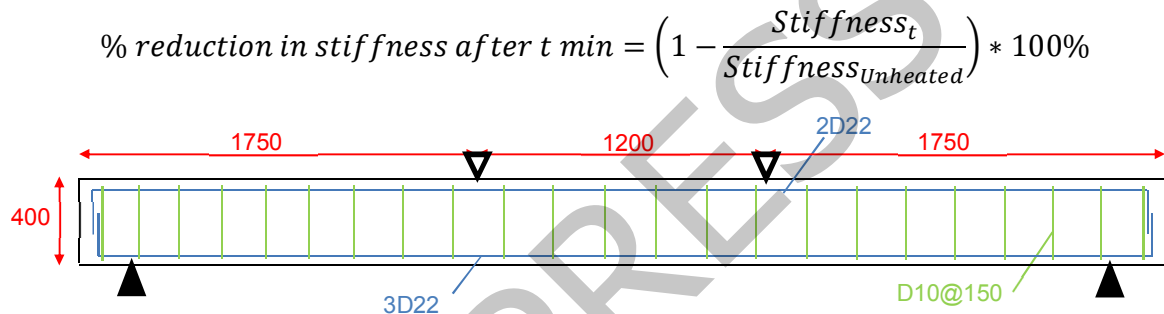

Fig. A.6: Section of RC beam and load pattern.

Response: 61% [13]

A.1.14 Seed Question 14

A T-beam, with the characteristics presented below, is considered (see Fig.A.7). The full beam is was tested under a standard time-temperature curve (ISO 834)75min. The web and the bottom flange of the T-beam are exposed to fire. This caused spalling in the bottom flange and many alligator cracks at the surface of the web. The beam is left to cool down and residual load bearing capacity is measured by testing then two point loads, as shown in the figure below, are applied until failure.

Can you estimate the residual bending moment of the beam at the support B, if you know that the elastic bending moment at support B for the ultimate load ($F=F_u$= 141 $kN$) is 70.39 $kNm$?

where, compressive strength of concrete = C30
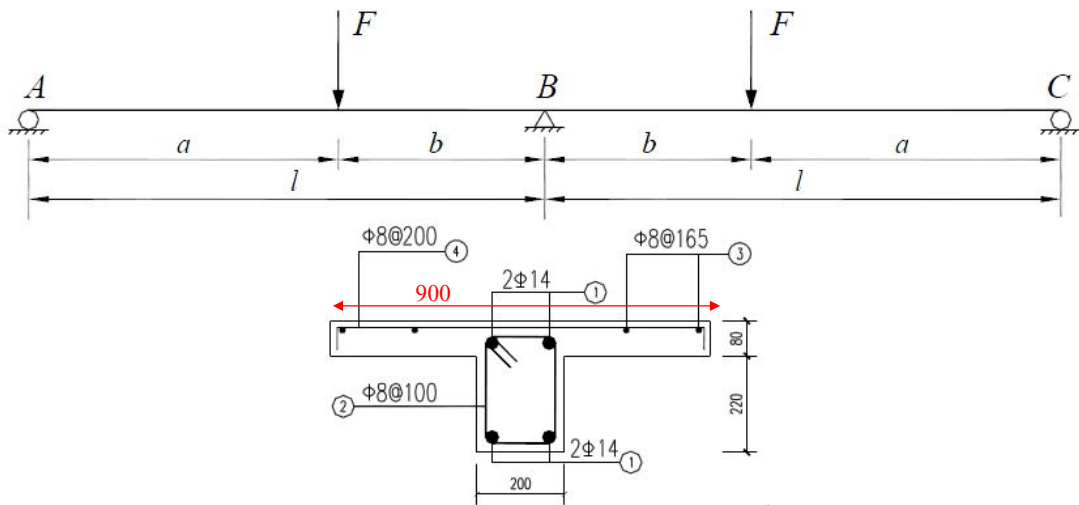$l$ = 2.6 $mm$
$a$ = 0.64*$l$

Fig. A.7: Set up of experiment and details of beam cross section [14].

Response: 41.48kNm [14]

A.1.15 Seed Question 15

Yaqub and Bailey [15] examined the effect of the cross-sectional shape on the post-heated RC columns.

Two square and two circular columns having length equal to 1,000mm and the cross-sectional dimensions presented in Fig.A.8 have been considered. One square and one circular unheated column were subjected to an axial compressive load until they failed with a sudden and explosive compressive brittle failure. The remaining two were heated to a uniform temperature until it reached 500°C. The columns were heated at this temperature until the centre of the column also reached 500 °C. The columns were then allowed to cool down naturally. No spalling was noted. The columns were finally subjected to an axial compressive load until they failed with a gentle crushing ductile failure.
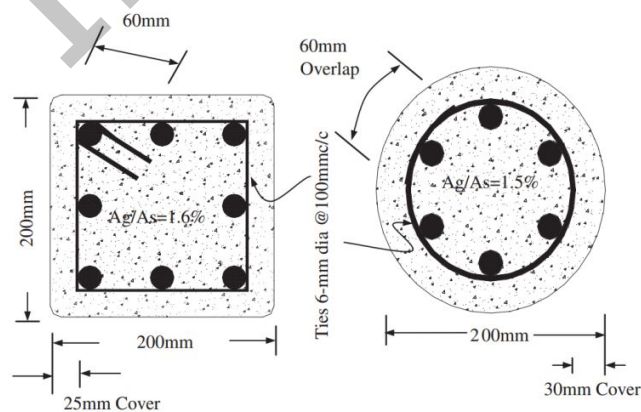


Fig. A.8: Details of the cross-sections used in [15].

The secant stiffness of the unheated square column is 3,854 k*N/mm*. The secant stiffness of the post-heated square column is 653 k*N/mm*. If the secant stiffness of the unheated circular column is 2,755 k*N/mm*, can you judge the secant stiffness of the post-heated circular column?

$$\text{Secant Stiffness} = \frac{F_{\text{Ultimate axial compressive load}}}{Displacement_{\text{At midheight of the column for } F_{\text{Ultimate axial compressive load}}}}$$

Response: 758 k*N/mm* [15]

A.1.16 Seed Question 16

A full scale 7-storey RC building was constructed in 1998 at the BRE laboratories in Cardington [16]. The building was designed according to Eurocode 2 and BS 8110. The plan of the building can be seen in Fig. A.10. The floor was constructed with C37 normal-weight concrete.

A fire compartment with a floor area of $225m^2$ on the ground floor was constructed, which represented a realistic scenario of an office fire.

The recorded temperatures throughout the compartment 300mm below the soffit of the slab are presented in Fig.A.9.
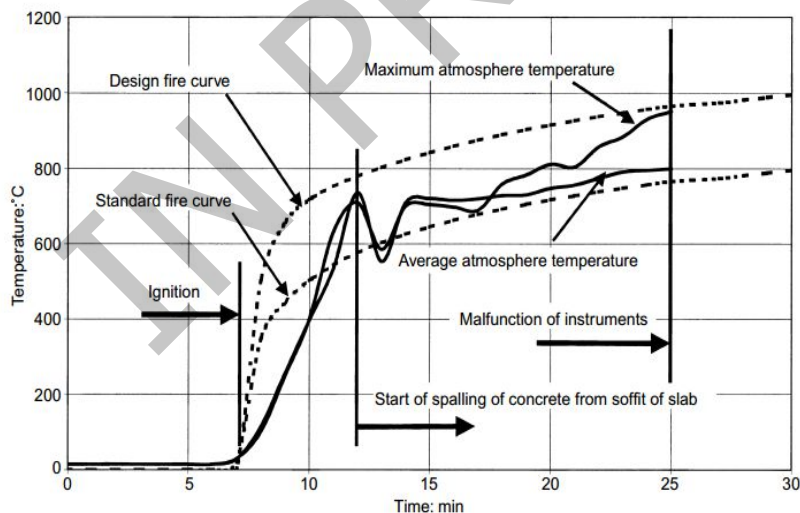


Fig. A.9: Recorded temperatures over time.

After 20min of fire, the average vertical displacement of the slab in points V4, V8 and V11 is 17mm. What is the vertical displacement of the slab if averaged over points V6, V9 and V13?
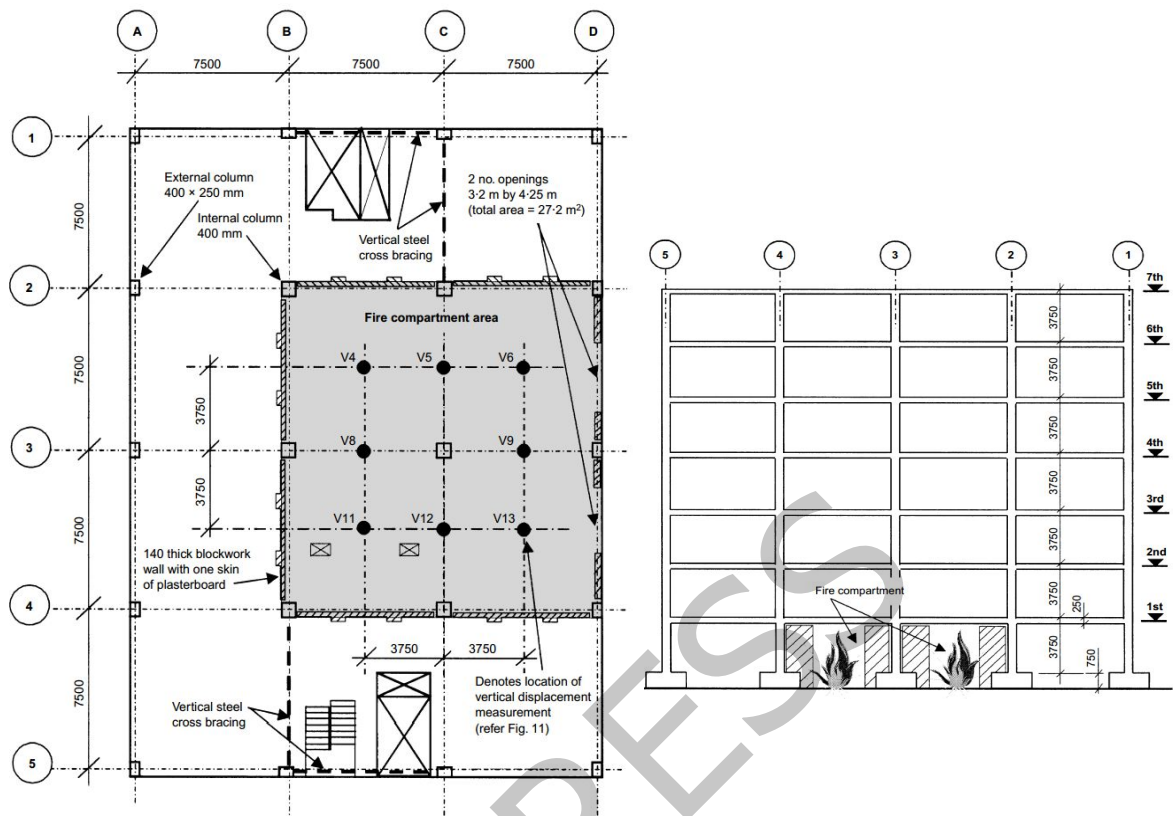
Response: 52.3 mm [16]

Fig. A.10: Plan (left) and cross-section (right) of the building showing the location of the fire compartment [16].

A.1.17 Examples – responses to seed questions

For illustrative purposes, the individual responses of the 13 experts to two seed questions. The anonymity of the participants is ensured by randomising the order of the individual responses the two target questions. In simple terms, the first response depicted in Fig.A.11(a) has been provided by a different expert than the first response depicted in Fig.A.11.(b). A typical response pattern is depicted in Fig.A.11(a), where the individual responses to seed question 4 are represented. It can be seen that 5 experts managed to capture the correct value. Expert 12 appears to have captured the correct value using the narrowest intervals. The remaining four experts provided much wider confidence intervals (e.g., EXP08). Fig.A.11(b) depicts the response to seed question 14, where no expert managed to capture the correct value.
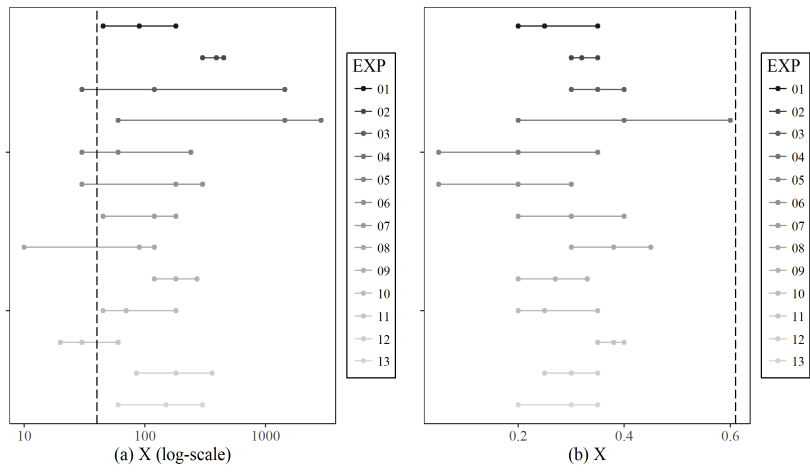
Fig. A.11: Individual responses for the seed questions (a) 4 and (b) 14.

A.2 TARGET QUESTIONS

The representative concrete structure (see Fig. A.2) is a cast-in place seven story braced frame, open plan office building with no sprinklers, incorporating columns, flat slabs and shear walls. A section through the building and a floor plan are given in figures 1 and 2. The building consists of four by three 7.5 m bays, with an inter-storey height of 3.75 m, and cast-in place 200 mm thick reinforced concrete shear walls to resist lateral loads. The floors are nominally 250 mm thick, constructed in grade C37 concrete, and designed as a flat slab, supported on 400 mm square internal columns, and 400 x 250 mm square edge columns. The reinforcement is traditional loose bar; varying between 16 mm and 12 mm diameters, with 12 mm diameter shear reinforcement around the columns.

The floor is constructed using a C37 normal-weight concrete, with a cube strength at 28 days of 61 N/mm$^2$. The moisture content of the water is 3.8% and has a 48mm permeability of $6.75 \times 10^{-17}$ m$^2$.

The columns and shear walls are constructed using grade C85 high-strength concrete WITHOUT polypropylene fibres, with a 28-day cube strength of 102 N/mm$^2$. The moisture content of the concrete is 4.2% by mass, and has a 48mm permeability of $1.92 \times 10^{-19}$ m$^2$.

Cover to the reinforcement is 20mm, 20mm, and 40mm for the slab, walls, and columns, respectively. The concrete contains limestone aggregate micro-silica supplementary cementing material.

The factored applied load to the floor plate <u>during</u> the fire situation is 3.25 kN/m$^2$.

The fire that occurs is assumed to occur simultaneously over the whole floor area.

The answers concern the elements exposed by fire on the 1$^{st}$ floor (british sytem).

*A.2.1 Response Measure-to-Damage relationships*

In this elicitation we will be concentrating on three different damage states from Table A.1, namely Damage States 4, 2 and 1, on the assumption that Damage State 3 can be inferred from understanding Damage States 2 and 4 more rationally.

A detailed description of each damage state is provided in what follows.

**Damage State 1 is primarily cosmetic in nature and does not impact on the design or repair of the structural fabric of concrete buildings;**

**Damage State 2 is considered as the state where a small amount of damage has been experienced by the element to the effect that some small scale remedial action is required to enhance the element's remaining ability to perform its structural function(s).;**

Damage State 3 is considered as the state where the element has experienced a significant, but not catastrophic, amount of damage to the effect

that, with significant remedial action, it can be reinstated to perform its structural functions; and

**Damage State 4 is considered the worst case where the damage caused by the fire is so extensive that it is no longer viable to repair and reuse the element and replacing the element with a new element is the only option. The building has not suffered a disproportionate collapse.**

TABLE A.1: DAMAGE CLASSIFICATION SCALE FOR REINFORCED CONCRETE ELEMENTS

| Damage State (DS) | Element | Surface Appearance of concrete | | | Structural condition | | |
|---|---|---|---|---|---|---|---|
| | | Condition of plaster/ finish | Colour* | Crazing | Spalling | Deflection/ distortions | Residual capacity |
| 0 | Any | Unaffected or beyond extent of fire | | | | | |
| 1 | Column | Some peeling | Normal | Slight | | | |
| | Wall | | | | | | |
| | Slab | | | | | | |
| 2 | Column | Substantial loss | Pink/Red | Moderate | | | |
| | Wall | | | | | | |
| | Slab | | | | | | |
| 3 | Column | total loss | Pink/Red Whitish grey | extensive | | | |
| | Wall | | | | | | |
| | Slab | | | | | | |
| 4 | Column | destroyed | whitish grey | surface lost | | | |
| | Wall | | | | | | |
| | Slab | | | | | | |

* Colour is dependent on type of aggregate used – Pink/Red seldom seen in calcareous concretes

The aim of the following questions is to ascertain your judgment on what levels of response of the element the expert would expect to observe given that it has been placed within one of the above four damage states.

A.2.1.1 SLAB
The experts are asked to provide their opinions regarding the SRM thresholds.

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| | 5% | 50% | 95% |
|---|---|---|---|
| *DAMAGE STATE 4* | | | |
| **Q1**: what percentage of the exposed surface area of the element will show signs of spalling such that it would be classed within the Damage State 4? (%) | | | |
| **Q2**: how much deflection (after cooling) would you associate with Damage State 4 for the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q3**: given no other structural damage to the element (i.e. no spalling or deformations), what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you associate with Damage State 4? (%) | | | |
| **Q4**: What peak rebar temperature corresponds to this damage state in the absence of spalling?(°C) | | | |
| *DAMAGE STATE 2* | | | |
| **Q5**: what percentage of the exposed surface area of the element will show signs of spalling such that it would be classed within the Damage State 2? (%) | | | |
| **Q6**: how much deflection (after cooling) would you associate with Damage State 2 for the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q7**: given no other structural damage to the element (i.e. no spalling or deformations), what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you associate with Damage State 2? (%) | | | |
| **Q8**: What peak rebar temperature corresponds to this damage state in the absence of spalling?(°C) | | | |
| *DAMAGE STATE 1* | | | |
| **Q9**: what percentage of the exposed surface area of the element will show signs of spalling such that it would be classed within the Damage State 1? (%) | | | |
| **Q10**: how much deflection (after cooling) would you associate with Damage State 1 for the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q11**: given no other structural damage to the element (i.e. no spalling or deformations), what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you associate with Damage State 1? (%) | | | |
| **Q12**: What peak rebar temperature corresponds to this damage state in the absence of spalling?(°C) | | | |

A.2.1.1 COLUMN

The experts are asked to provide their opinions regarding the SRM thresholds.

| | 5% | 50% | 95% |
|---|---|---|---|
| *DAMAGE STATE 4* | | | |
| **Q13**: what percentage of the exposed surface area of the element will show signs of spalling such that it would be classed within the Damage State 4? (%) | | | |
| **Q14**: how much deflection (after cooling) would you associate with Damage State 4 for the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q15:** given no other structural damage to the element (i.e. no spalling or deformations), what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you associate with Damage State 4? (%) | | | |
| **Q16:** What peak rebar temperature corresponds to this damage state in the absence of spalling?(ºC) | | | |
| *DAMAGE STATE 2* | | | |
| **Q17**: what percentage of the exposed surface area of the element will show signs of spalling such that it would be classed within the Damage State 2? (%) | | | |
| **Q18**: how much deflection (after cooling) would you associate with Damage State 2 for the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q19:** given no other structural damage to the element (i.e. no spalling or deformations), what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you associate with Damage State 2? (%) | | | |
| **Q20:** What peak rebar temperature corresponds to this damage state in the absence of spalling?(ºC) | | | |
| *DAMAGE STATE 1* | | | |
| **Q21**: what percentage of the exposed surface area of the element will show signs of spalling such that it would be classed within the Damage State 1? (%) | | | |
| **Q22**: how much deflection (after cooling) would you associate with Damage State 1 for the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q23:** given no other structural damage to the element (i.e. no spalling or deformations), what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you associate with Damage State 1? (%) | | | |
| **Q24:** what peak rebar temperature corresponds to this damage state in the absence of spalling?(ºC) | | | |

*A.2.2 Intensity-to-Response Measure relationship*

The intensity measure chosen is based on the principle that the area under any ACTUAL temperature-time fire curve above a baseline of 150°C (Area A) can be equated to the same area under the STANDARD temperature-time fire curve from *ISO-834* above a baseline of 150°C (Area B), from which we can calculate the equivalent exposure time to the standard fire curve ($T_s$). This equivalency will be used as the Intensity Measure (*IM*) for the following question.
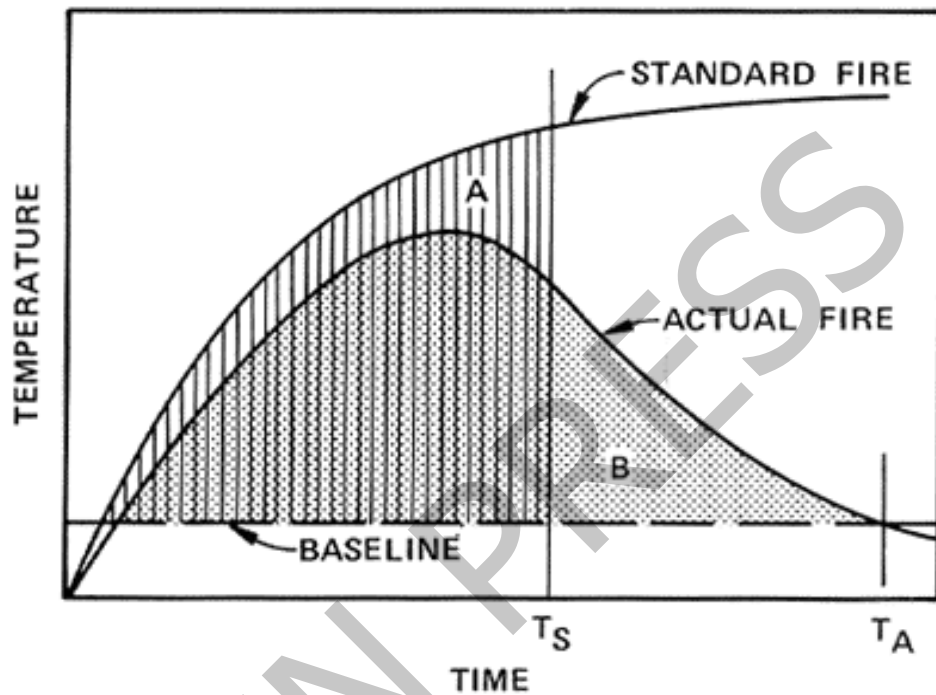


The *IM, $T_s$*, will be defined at three levels; A; B; and C, representing progressively greater severity, and have values of 30, 60, and 120 minutes equivalent exposure to the standard fire curve, respectively. The fire is assumed to occur over the whole floor area simultaneously.

The aim of the following questions is to ascertain your judgement on what level of response is likely to occur given the three different levels of fire intensity.

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A.2.2.1 SLAB

| GIVEN THAT THE SLAB HAS BEEN EXPOSED TO IM_A--- TS = 120 MINUTES | 5% | 50% | 95% |
|---|---|---|---|
| **Q25**: what percentage of the element will show signs of spalling? (%) | | | |
| **Q26**: how much deflection after cooling would you expect to observe in the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q27**: what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you judge will remain? (%) | | | |
| **Q28**: What rebar-temperature do you expect in the absence of spalling? | | | |
| GIVEN THAT THE SLAB HAS BEEN EXPOSED TO IM_B --- TS = 60 MINUTES | | | |
| **Q29**: what percentage of the element will show signs of spalling? (%) | | | |
| **Q30**: how much deflection after cooling you would expect to observe in the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q31**: what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you judge will remain? (%) | | | |
| **Q32**: What rebar-temperature do you expect in the absence of spalling? | | | |
| GIVEN THAT THE SLAB HAS BEEN EXPOSED TO IM_C --- TS = 30 MINUTES | | | |
| **Q33**: what percentage of the element will show signs of spalling? (%) | | | |
| **Q34**: how much deflection after cooling would you expect to observe in the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q35**: what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you judge will remain? (%) | | | |
| **Q36**: What is the rebar temperature if there is no spalling? | | | |

A.2.2.2 COLUMN

| GIVEN THAT THE COLUMN HAS BEEN EXPOSED TO IM_A--- TS = 120 MINUTES | 5% | 50% | 95% |
|---|---|---|---|
| **Q37**: what percentage of the element will show signs of spalling? (%) | | | |
| **Q38**: how much deflection after cooling would you expect to observe in the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q39**: what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you judge will remain? (%) | | | |
| **Q40**: What rebar-temperature do you expect in the absence of spalling? | | | |
| GIVEN THAT THE COLUMN HAS BEEN EXPOSED TO IM_B --- TS = 60 MINUTES | | | |
| **Q41**: what percentage of the element will show signs of spalling? (%) | | | |
| **Q42**: how much deflection after cooling you would expect to observe in the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q43**: what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you judge will remain? (%) | | | |
| **Q44**: What rebar-temperature do you expect in the absence of spalling? | | | |
| GIVEN THAT THE COLUMN HAS BEEN EXPOSED TO IM_C --- TS = 30 MINUTES | | | |
| **Q45**: what percentage of the element will show signs of spalling? (%) | | | |
| **Q46**: how much deflection after cooling would you expect to observe in the element (mid-span for slab; end displacements for column/wall)? Please give X in form of Length/X. | | | |
| **Q47**: what percentage of residual capacity (i.e. axial load capacity for column/wall, flexural capacity for slab) would you judge will remain? (%) | | | |
| **Q48**: What is the rebar temperature if there is no spalling? | | | |

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A.2.3 Examples – responses to target questions

The experts are invited to provide their opinions on three values of each target variable corresponding to the 5%, 50% and 95% probability of being exceeded. Two examples of the individual responses of the 13 experts compared with their pooled opinions using Cooke's weighting scheme as well as equal weighting are presented in Fig.A.12. The anonymity of the participants is ensured by randomising the order of the individual responses the two target questions. In simple terms, the first response depicted in Fig.A.12(a) has been provided by a different expert than the first response depicted in Fig.A.12.(b). Fig.A.12.(a) depicts the $ds_1$ residual capacity threshold for columns. When equal weighting is used, the threshold appears to be associated with a rather large uncertainty. By contrast, the highest-weighted experts (Cooke_W) managed to better constrain the uncertainty. The opposite can be observed in Fig.A.11.(b), which depicts the responses for the residual capacity of columns for T=30min. In this case, the Cooke's weighting scheme appears to produce larger uncertainty than its equal weighting counterpart. This suggests that the highest-weighted experts believe it is difficult to constrain the uncertainties on this variable on the basis of present knowledge and experience.
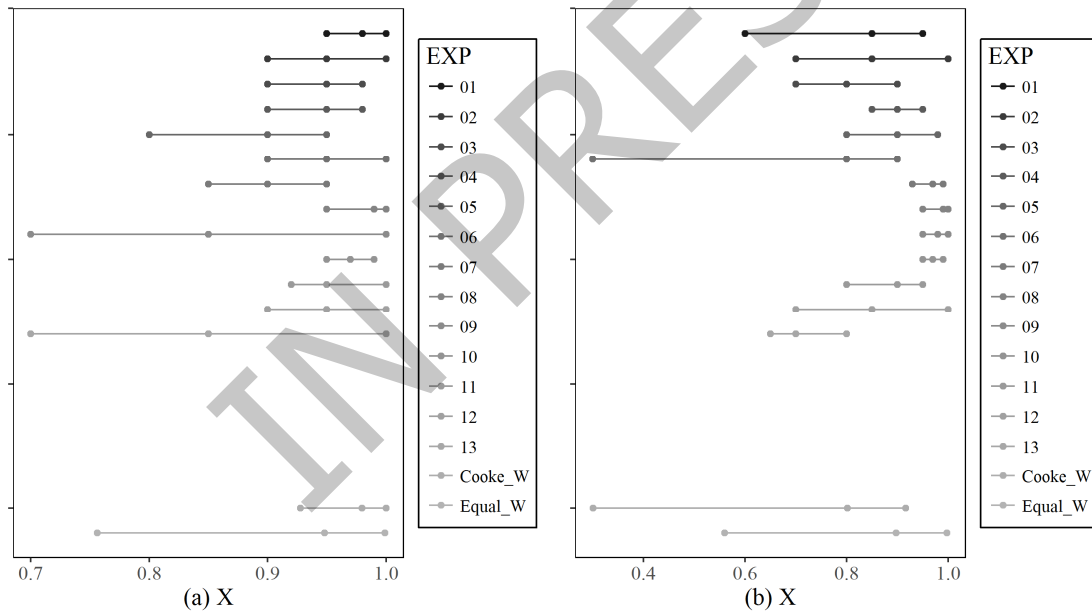


Fig. A.12: Individual responses as well as the pooled opinions using the Cooke's weights and equal weights for two target questions (a) the residual capacity threshold of a column for $ds_1$ and (b) the residual capacity of a column for T = 30min.

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