




Review of Current Practice in Probabilistic Structural Fire Engineering: Permanent and Live Load Modelling

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Abstract. Probabilistic analysis is receiving increased attention from fire engineers, assessment bodies and researchers. It is however often unclear which probabilistic models are appropriate for the analysis. For example, in probabilistic structural fire engineering, the models used to describe the permanent and live loads differ widely between studies. Through a literature review, it is observed that these diverging load models are based on surveys conducted between 1893 and 1976 and that widely adopted assumptions, such as the rule for combining permanent and live loads into the total load effect, are commonly adopted based on precedent. The diverging current models however relate to mostly the same underlying datasets and basic methodologies. Differences can be attributed (largely) to specific assumptions in different background papers, which have become consolidated through repeated use in research papers and adoption in background documents to codes. By reviewing the studies underlying currently applied probabilistic load models in structural fire engineering, a consolidated probabilistic load model is proposed in this paper. It is concluded that the total load effect is ideally described by $K_E(G + Q)$, with K_E the model uncertainty for the load effect, G the permanent load, and Q the imposed load. The model uncertainty K_E can be described by a lognormal distribution with mean equal to unity and coefficient of variation (COV) of 0.10. The permanent load is preferably modelled by a normal distribution with mean equal to the nominal permanent load, and a COV which can either be assessed on a project specific basis, or can be set to 0.10 for a first assessment. For common occupancies (office, residential), the

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live load is preferably modelled by a Gamma distribution. The mean live load can be taken as 0.2 times the nominal, and the live load COV can be taken as 0.60 for large load areas ($> 200 \text{ m}^2$) and 0.95 for smaller load areas ($< 100 \text{ m}^2$). Comparison between the failure probabilities of steel and concrete columns subject to fire, considering the proposed consolidated model and two currently commonly used models, indicates that relative differences of the probability of failure can be in the order of 10%. Live load models for evacuation routes and warehouses require specific study and are outside the scope of the review.

Keywords: Permanent load, Live load, Probabilistic analysis, Structural fire engineering

List of Symbols

G	Permanent load (nominal value G_{nom} , characteristic value G_k)
Q	Imposed load (nominal value Q_{nom} , characteristic value Q_k)
δ	Coefficient of variation
μ	Mean value
σ	Standard deviation
χ	Load ratio = $Q_k/(Q_k + G_k)$

Abbreviations

APIT	Arbitrary point in time
COV	Coefficient of variation
PMC	Probabilistic Model Code
PSFE	Probabilistic structural fire engineering

1. Introduction

Traditionally, fire engineering is based on the experience of the profession [1]. In prescriptive approaches, the experience drives the updating of design requirements or guidance in the wake of fire events [2]. When using (deterministic) performance-based design approaches, the definition of scenarios, acceptance criteria, and input values are fundamentally based on the experience [3]. Experience-based approaches can be considered efficient and reasonable in common cases for which performance has been repeatedly observed, and where the possibility of disastrous consequences is negligible. However, for innovative design solutions, or when considering situations with the potential for high consequences (e.g. structures with higher reliability classes), a full probabilistic approach is necessary to demonstrate adequacy of the design [3].

This realization, as well as expected cost reductions and genuine research interests, have fuelled a series of research projects and applications in structural fire engineering (SFE), e.g. [4–17]. The appropriate models for the stochastic input variables of the probabilistic analysis are however not always clear to the user performing probabilistic SFE (PSFE). Also, the identification of stochastic variables varies widely between studies. To partially alleviate this, a review of probabilistic models used to describe the permanent and imposed loads in structural fire engineering has been conducted. Preliminary results were presented by Van Coile et al. [18] at the 2019 CONFAB conference. In the current contribution this

review is revisited and further elaborated, notably by a more in-depth literature review and analysis. By reviewing models used in previous studies and their background, permanent and imposed load models for PSFE are recommended.

It should be noted that: (i) The recommended load models are based on historical data (no new surveys were conducted); (ii) Overcrowding near emergency exits during the evacuation process is not considered and the recommended load models have not been derived to apply for evacuation routes; and (iii) Imposed loads in warehouses are excluded from the final recommendation as they represent a special case. The presented study is directed towards probabilistic structural fire analyses and the discussions are mainly focused on research in this field. The discussion and results can, however, also be of immediate relevance to the analysis of other accidental design situations. At the end of the paper, the effect of the load model assumptions on failure probability evaluations is highlighted using examples of steel and concrete columns subjected to fire.

2. Stochastic Models in SFE

2.1. Problem Statement

Fire ignition is a rare event and many fires are extinguished or suppressed early on by the occupants of the building or active measures (e.g. sprinklers). SFE is concerned with fires which progress beyond this barrier, often referred to as 'structurally significant' fires. The probabilistic structural analysis starting from the occurrence of a structurally significant fire thus assesses a conditional failure (or survival) probability of the structure given the occurrence of the significant fire.

A range of consequential input variables for SFE are stochastic in nature (i.e. uncertain). These can be grouped broadly in two categories: uncertainty in materials (resistance), and uncertainty in actions (both thermal and mechanical). With respect to the uncertainty in materials, the thermal and mechanical properties of building materials are subject to large scatter, both at normal design temperatures and at elevated temperatures. For elevated temperatures limited models exist. For a review and recent developments, see e.g. [19, 20]. The uncertainty in actions relates to thermal actions (i.e., the thermal exposure) on the one hand, and mechanical actions on the other hand. For guidance on the thermal actions and their probabilistic modelling, see [21].

Focussing on the mechanical actions, these are traditionally subdivided into permanent actions and imposed (or variable) actions. Their variability with time is an aspect of particular relevance for SFE. In design for normal conditions, the load variability is considered by a (characteristic or design) load with a low probability of being exceeded during the lifetime of the structure. This ensures that building structures are designed both safely and (largely) economically, as a balance has been sought between the costs of premature failures and the costs of additional up-front (safety) investments when establishing the design requirements [22]. Naturally, the day-to-day probability of occurrence of such high (design) load value is low, just as for the day-to-day probability of occurrence of a struc-

turally significant fire. Taking into account both events simultaneously would result in very onerous fire design requirements, as the coincidence rate is very low. Hence, the reduced safety and combination factors in the Eurocode [23] and in the ASCE design format (load and resistance factor design) [24] lessen the required load under consideration for structural fire design compared to that of normal design conditions as is also the case for other accidental design situations. The above highlights that application of appropriate probability distributions of permanent and imposed actions for probabilistic structural fire analyses is crucial for a correct evaluation of (conditional) failure probabilities in PSFE.

2.2. Literature Review of Load Models in PSFE

The PSFE literature contains application of different permanent load and imposed load models. These studies make limited explicit reference to the issue of time variability of the loads. Most state (directly or indirectly) that their load models correspond with arbitrary point in time (APIT) dead (permanent) and live (imposed) loads, e.g. [6–8, 14, 15]. The study by Ellingwood [16] is a notable exception, going to some depth in explaining underlying processes of load variability.

A non-exhaustive overview of applied permanent load models is given in Table 1, where μ is the mean value, δ the coefficient of variation (COV), and G_{nom} the nominal permanent load. The nominal permanent load results from the nominal dimensions and density of both the structure itself and finishes. The references for the applied models in the PSFE studies are not always fully clear. Regularly a set of references is given at once to justify a list of stochastic variables. Based on the overview of Table 1, the studies agree on the use of a normal distribution with COV of 0.10. With respect to the mean permanent load, however, two sets of models are discerned. One school of thought considers a mean value equal to 1.05 G_{nom} , whereas the other specifies the mean as directly equalling G_{nom} . The first school has Ellingwood's 2005 study [16] as a common point of reference, whereas the second school relates to formulations underlying the Eurocodes, see Sect. 3.2 for further background.

Table 2 gives an overview of imposed load models applied in PSFE literature, showing a larger variation in models. The imposed load model used by Devaney [10] and Van Coile et al. [11] is the distribution model for the 50-year maximum value of the imposed load (i.e. 50 year 'reference period') [30]. This description is commonly used for reliability evaluations in normal design conditions to evaluate the probability of failure over this long reference period (often considered equal to the design life), and thus does not correspond with APIT loads. It has the advantage that the reliability index at the start of fire exposure (i.e. while the structure has not yet been heated) is the same as the commonly applied (50-year reference period) reliability index for normal design conditions. The physical interpretation of the resulting failure probability during fire is however not intuitive. It is unclear if the same applies to [12, 13], as no explanation could be found in the references themselves.

The other applied live load models relate to APIT load. One family again originates from the work of Ellingwood [16]. Another series of studies [14, 15] model

Table 1
Permanent Load Model in PSFE Studies

Study	Distribution	μ/G_{nom}	δ	Reference
[6, 7]	Normal	1.05	0.10	[8, 16]
[8]	Normal	1.05	0.10	[16, 25]
[9]	Normal	1.05	0.10	[16]
[16]	Normal	1.05	0.10	[26, 27]*
[10]	Normal	1.00**	0.10	[28, 29]
[11, 14, 15]	Normal	1.00	0.10	[30]
[12, 13, 17]	Normal	1.00	0.10	Not given

*Further refers to 'additional published data'; values listed as 'typical'

**Implied by listed data

Table 2
Imposed Load Model in PFSE Studies

Study	Distr.	μ/Q_{nom}	δ	Reference
[6]	Gamma	0.24	0.80	[8, 16, 25]
[6]	Gamma	0.24	0.60	[8, 16, 25]
[8]	Gamma	0.24	***	[16, 25]
[9]	Gamma	0.24 to 0.50	0.60	****
[16]	Gamma	0.24 to 0.50	0.60	[26, 27]*
[10]	Gumbel	0.60	0.35	[17, 29]
[11]	Gumbel	0.60	0.35	[30] (50y)
[12]	Gumbel	0.60	0.50	Not given
[13]	Gumbel	0.52**	0.50	Not given
[14, 15]	Gumbel	0.2	1.10	[30] (5y)
[17]	Gumbel	0.2	1.10	*****

*Further general references as 'additional published data'; values listed as 'typical'

**Implied by listed data

***Variable in function of the tributary floor area [25]

****Reference is made to a 2011 email by B.R. Ellingwood

*****Eurocode background document, no reference given

the APIT load by a Gumbel distribution with a 5-year reference period, by assuming that the imposed load can be modelled by a rectangular wave renewal process with a 5-year return period. The 5-year return period corresponds with the expected time between renewals (changes in use and users [31]) for office buildings [30]. The Gumbel model has the advantage of allowing for a direct calculation of the imposed load model for fire conditions from the imposed load model applied for normal design reliability analysis (in the Eurocode framework). On the other hand, the Gumbel distribution has the disadvantage of having a non-zero probability of negative values. The same 5-year Gumbel distribution is applied without reference in the background document to the Eurocode fire design guidance [17]. Further background to the live load models is discussed in Sect. 4.2.

The models listed in Tables 1 and 2 are not sufficient to compare current load models in PSFE, as additional stochastic factors are taken into account when combining the permanent and imposed load effects. For example, Guo et al. [6] specify the total load w through Eq. (1), referencing [8, 25], with the stochastic parameters A , B , and E listed in Table 3. The same total load model is applied in [7, 8]. Iqbal and Harichandran [8] clarify that the factors A and B refer to uncertainty in the transformation of loads into load effects, and that E refers to uncertainty in structural analysis, and refer to Ravindra and Galambos [25] for the origin of these stochastic factors. Ravindra and Galambos [25] for their part refer to Eq. (1) as an assumption, and explain that A and B are to be interpreted as characterizing the difference between computed and actual internal forces in the structure, while E is intended to characterize deviations introduced by characterizing a 3D structure into elements or sub-systems and other simplifying assumptions (such as boundary conditions). They however do not mention a distribution type for these variables, and indicate that the mean values and COVs (as listed in Table 3) were ‘chosen’ and ‘assumed’ as ‘reasonable estimates based on data and judgements’, with further reference to a UK live load survey [32]. It is interesting to note that Ravindra and Galambos consider $\mu_G = G_{nom}$ and $\delta_G = 0.04$, raising the question whether the factors E and A are (partially) taken into account (in duplicate) as part of the alternative permanent load model applied in the later studies (e.g. [6]), see Table 1.

$$w = E(AG + BQ) \quad (1)$$

In [6, 7, 9, 15, 16] on the other hand, the total load model of Eq. (2) is used, where K_E is the model uncertainty for the load effect. No reference is given for the normal model applied in [12]. The lognormal model uncertainty in the other studies is based on the review by Holický and Sýkora [30], with reference to the JCSS (Joint Committee on Structural Safety) Probabilistic Model Code [33] (except [17] where no reference is given). The Probabilistic Model Code (PMC) makes a distinction in recommended COV for K_E in function of the load effect (axial load or moment). For frames, a COV of 0.1 is the higher value across the load effects (bending, shear, axial force). Only for moments in plates, the recommended value of COV is higher at 0.2 [33]. The PMC provides no indication as to the origin of these values.

$$w = K_E(G + Q) \quad (2)$$

Other total load formulations were (implicitly) applied in other references, e.g. Hamilton [9] applied a single professionalism factor which also takes into account uncertainty with respect to the resistance model (material strength retention factor), while Devaney applied $G + Q$ directly without addition of further stochastic model or combination parameters. The widely cited paper by Ellingwood [16] does not specify a clear formulation for the total load model.

Table 3
Load Combination Parameters as Applied in Total Load Models

Parameter	Distribution	μ	δ	Applied in
E	Normal	1.0	0.05	[6–8]
A	Normal	1.0	0.04	[6–8]
B	Normal	1.0	0.20	[6–8]
K_E	Normal	1.0	0.10	[12]
K_E	Lognormal	1.0	0.10	[11, 14, 15, 17]

It should be noted that the factors in Table 3 were originally developed for normal design reliability evaluations. Their appropriateness for addressing uncertainty in case of fire has not been addressed explicitly.

2.3. Effect of Different Literature Models on Total Load Distribution

To assess the effect of the different load models on the implied total load effect, the total load formulations are compared. To make a direct comparison possible without introducing further assumptions, the load ratio χ and total nominal load P_{nom} are defined through Eq. (3), where the nominal values Q_{nom} and G_{nom} correspond to the characteristic values of Q_k and G_k when using the Eurocode methodology. Further writing the stochastic permanent load G as $G_{nom} \cdot g$ and the stochastic imposed load Q as $Q_{nom} \cdot q$, the ratio ζ of the total load w to the nominal total load P_{nom} is given by Eqs. (4) and (5) for the total load models of Eqs. (1) and (2) respectively. Note that G and Q are stochastic variables, while G_{nom} and Q_{nom} are deterministic values. The variation in G and Q is thus taken into account through the stochastic variables g and q with μ and δ as listed in Tables 1 and 2. The Eqs. (4) and (5) have been evaluated using 10^8 crude Monte Carlo Simulations (MCS), for different load ratios χ , applying the distribution models as applied in [7] ('Model A', in black) and [15] ('Model B', in red), respectively. As a reminder, Ref. [7] adopted the total load model of Eq. (1), while Ref. [15] adopted that of Eq. (2). The obtained cumulative density functions (CDF) and complementary CDF (cCDF) for the total nominal load factor ζ are given in Fig. 1, along with the corresponding probability density functions (PDF) in Fig. 2.

For failure probability evaluations of structures, large values of ζ are of interest. It is clear from Fig. 1 that there is a notable difference between both models. For example, assuming a load ratio of 0.5, the total nominal factor that has a probability of exceedance of 10^{-3} is equal to 1.08 with Model A and 1.22 with Model B. The magnitude of this difference justifies further efforts in evaluating the background of the load models, and proposing a consolidated load model for future reference. This is completed in the following sections, with special emphasis on the issue of time dependency and on providing further background to the load models.

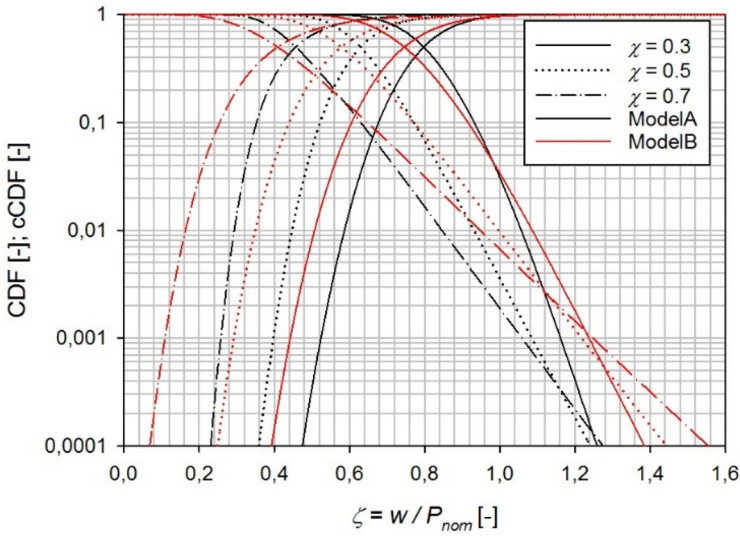


Figure 1. CDF and cCDF for the total nominal load factor ζ according to Eqs. (4) and (5), for different load ratio χ . Applying the distribution models of [7] ('Model A') and [15] ('Model B') respectively.

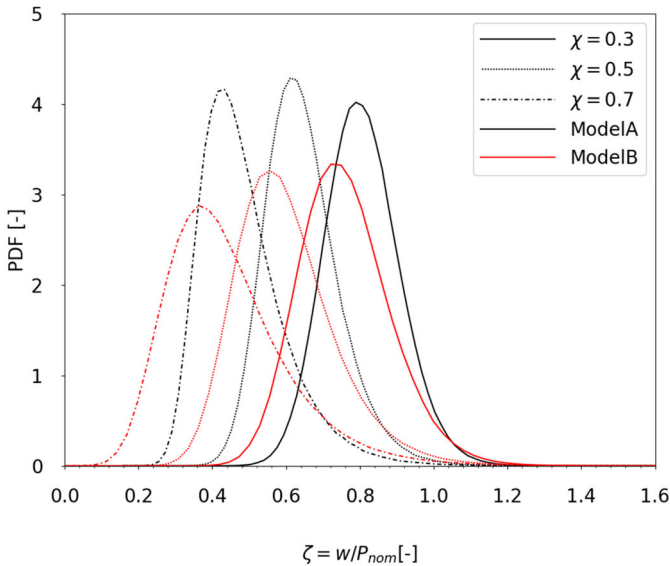


Figure 2. PDF for the total nominal load factor ζ according to Eqs. (4) and (5), for different load ratio χ . Applying the distribution models of [7] ('Model A') and [15] ('Model B') respectively.

$$\chi = \frac{Q_{nom}}{Q_{nom} + G_{nom}} = \frac{Q_{nom}}{P_{nom}} \quad (3)$$

$$\frac{w}{P_{nom}} = E(A(1 - \chi)g + B\chi q) = \zeta \quad (4)$$

$$\frac{w}{P_{nom}} = K_E((1 - \chi)g + \chi q) = \zeta \quad (5)$$

3. Permanent Load Model

3.1. TIME Dependency

The permanent actions result from the self-weight of the structural elements and finishes, and can be considered time-invariant [26, 34, 35]. Hence, for the stochastic model of the permanent load no distinction needs to be made between normal design conditions and the fire situation, when neglecting possible combustion of finishes or structure (as might be the case for a combustible frame, like mass timber). Thus, models applied for normal design are directly applicable, and qualify as APIT permanent loads.

Neglecting combustion and therefore reduction of loads is a standard and conservative approximation. There are currently no models available to readily take this potential load-reducing effect into account. It can also be noted that the occurrence of fire in a structure implies that the structure did not fail due to gravity loads prior to fire. This means that the probability of very high load realizations is lower than considered in the prior distribution for design calculations. Adjusting the load to take into account no prior failure due to large gravity load realizations is commonly neglected.

3.2. Model Background

In principle, the uncertainty associated with the self-weight l of an element can be calculated considering uncertainty related to the material density γ and the element's volume V [34]. In general terms, the density of the material is described by a random field, i.e. the density varies throughout the material [34]. For reasonably homogenous materials, an average (but stochastic) density can be considered for the entire element [35]. This average density can be considered to be normally distributed, with mean values and COV listed for common building materials in the PMC [35]. The PMC further lists geometric deviations of structural sections as normal random variables with positive mean, but states (in a simplifying manner) that the mean value of the volume can be calculated directly from the mean value of the dimensions, and that the mean dimensions can be considered equal to their nominal value [35]. In their summary overview of appropriate stochastic models for structural reliability analysis, Holický and Sýkora [30] note that the mean vol-

ume will generally be slightly larger than the nominal value. This is because the expected deviations from the nominal dimensions are slightly positive, as noted above. For concrete elements, for example, this exceedance of the nominal volume by the mean volume will be less than 1% (taking into account the models for dimensional deviations in [35]). As demonstrated in [36], also for steel elements the average relative cross-section area is slightly higher than the nominal value.

With both γ and V described by a normal distribution, the self-weight l is in principle not normally distributed. However, when the coefficients of variation (COV) of the volume and density are small (which is generally the case), the resulting self-weight loads can nevertheless be assumed to be described by a normal distribution [34]. This has also been adopted in the JCSS PMC [35]. Considering Taylor expansion, the mean value of the self-weight μ_l is given by $\mu_\gamma \cdot \mu_V$. The coefficient of variation δ_l can be estimated from [30]:

$$\delta_l^2 = \delta_V^2 + \delta_\gamma^2 + \delta_V^2 \delta_\gamma^2 \quad (6)$$

Values of μ_γ and δ_γ , as given in the PMC, are listed in Table 4, together with indicative values for δ_l as listed in [30].

When multiple materials or components contribute to the permanent load effect, the addition of normally distributed variables needs to be taken into account. If the constituent self-weights l_i are assumed as independent variables (with mean values μ_{l_i} and standard deviation σ_{l_i}), the overall permanent load is described by a normal distribution, with mean values μ_G and standard deviation σ_G given by:

$$\mu_G = \sum_i \mu_{l_i} \quad (7)$$

$$\sigma_G = \sqrt{\sum_i \sigma_{l_i}^2} \quad (8)$$

3.3. Probabilistic Model

Considering the background discussion above, the permanent load can be described by a normal distribution, with a mean value slightly larger than nominal value, and a COV which can be evaluated from the contributions of the different (independent) elements based on Eqs. (7) and (8). It is reasonable to apply Eqs. (7) and (8) to calculate the COV of permanent load when evaluating an existing building. For general reliability studies and code calibration purposes, detailed assumptions on floor build-up and construction materials should, for the purpose of generality, be avoided. Thus, a generally applicable COV is preferable.

Holický and Sýkora indicate that application of the PMC recommended values commonly results in a COV from 0.03 to 0.10 for permanent loads [30]. This also in alignment with the recommendations from *fib* (*Fédération internationale du*

Table 4
Distribution Parameters for the Density of Common Building Materials [35] and Indicative Values for the Coefficient of Variation of the Self-Weight, δ_l [30]

Material	μ_γ (kN/m ³)	δ_γ	δ_l
Steel	77	0.01	0.032
Concrete	24	0.04	0.045
Masonry	Not specified	0.05	0.064
Timber	*	0.10	0.101

*Listed in [35] for 12% moisture content: spruce: 4.4; fir: 4.4; pine: 5.1; larch: 6.6; beech: 6.8; oak: 6.5

béton) Bulletin 80 [37]. Taking into account the models currently applied in PSFE (Table 1), there is a pronounced preference for a COV of 0.10 as a generally applicable recommendation for the permanent load model.

The JCSS simplifying approximation of taking the mean volume as the nominal volume implies that the mean permanent load does not exceed the nominal value. It is considered preferable to neglect a 1% (order of magnitude) exceedance, than to overestimate the mean value by setting μ_G equal to $1.05 G_{nom}$. The underestimation of the mean permanent load by setting $\mu_G = G_{nom}$ is from an overall perspective compensated by the practical choice for a COV of 0.10. Consider for example a model (i) $\mu_G = 1.05 G_{nom}$, COV = 0.07, and (ii) $\mu_G = 1.00 G_{nom}$, COV = 0.10. The 3% highest realizations of G are larger in model (ii) than in model (i).

In conclusion, the permanent load effect G is recommended to be described by a normal distribution, with mean equal to the nominal permanent load effect G_{nom} , and COV of 0.10. As mentioned earlier, this does not take into account the reduction of the permanent load as a consequence of a possible combustion of finishes or structure.

4. Live Load Model

4.1. Time Dependency

ive loads arise from a range of items, from building occupants and their possessions, to movable items, like furniture. This movable characteristic of such loads makes time dependency particularly pertinent for the live load model. The total live load can be broken down into two components: (1) a sustained component, and (2) an intermittent or transient component [16, 27, 30, 31, 38]. The sustained load refers to the weight of furniture and heavy equipment, with variations usually coming from changes in building use [31]. Also normal occupancy load is included in the sustained load, e.g. [23]. The intermittent live load, on the other hand, relates to exceptional events, such as overcrowding [23], or the stacking of objects during refurbishing [31]. With respect to overcrowding, the CIB (Conseil International du Bâtiment) report [39] refers to a possible link to emergency situa-

tions, such as a loading scenario in the vicinity of a fire escape. Very limited data is available on this intermittent load. Whilst both the sustained load and the intermittent load vary with time, a component of the sustained load is ever present—albeit its magnitude could vary. Figure 3 illustrates the difference between the sustained and intermittent live load, adapted from [16].

For PSFE applications, the APIT live load is of interest, as noted earlier. As the occurrence of the intermittent (transient) live load is by its conceptualization rare, it generally does not need to be taken into account simultaneously with fire exposure [16]. While this can be considered sufficient for the general floor area of most buildings (e.g. offices, residential buildings), care should be taken whenever the live load profile of the building has specific occurrence patterns or a particular likelihood of overcrowding (e.g. sports stadia), or when considering buildings with high reliability requirements (e.g. high-rise structures).

The possible overcrowding near emergency exits and on evacuation routes during the evacuation process needs to be mentioned. In those situations extreme values of live load can occur. This load is however (partially) compensated by the necessary absence of furniture in those areas. This aspect requires a separate in-depth analysis before conclusions can be drawn and is not considered further here. The models specified below are not developed to apply to evacuation routes.

Another case where the more detailed analysis of the intermittent live load is of particular interest is during construction and refurbishment, as the fire risk can be higher (with for example various fire safety precautions not yet installed or operational). Also those situations are considered special cases beyond the scope of the current review.

4.2. Model Background

Ellingwood [16] is commonly referenced in PSFE applications to justify describing the APIT live load Q by a Gamma distribution, with mean value between 0.24 and 0.50 Q_{nom} and COV of 0.60 (see Table 2). Ellingwood himself further refers to [26, 27].

In [26], Ellingwood and Culver consider data obtained in a 1974 to 1975 US survey of loads in office buildings (excluding people) and assess the equivalent uniformly distributed APIT load Q . Only mean values and standard deviations are listed, as reproduced in Table 5. This data includes a nominal personnel load of 81 N/m². Personnel load was not measured directly as part of the survey. Ellingwood states that no significant difference with UK [32] data published in the early 1970s could be discerned and the dependency on floor area was stated to be statistically weak [26]. The agreement between the US and UK data is confirmed through Table 6 where summary results of the UK survey [32] mentioned by Ellingwood are presented, as analysed by Peir [40]. The values indeed show very little difference with those in Table 5, but it must be mentioned that the values in Table 5 are the product of a curve fitting procedure by Ellingwood. The actual data that was presented graphically in [26] shows a large scatter. Considering a nominal live load of 50 psf (2.39 kN/m²), the data in Table 5 corresponds to $\mu_Q = 0.23 Q_{nom}$. Generally, an additional 15 psf (0.72 kN/m²) for “partition

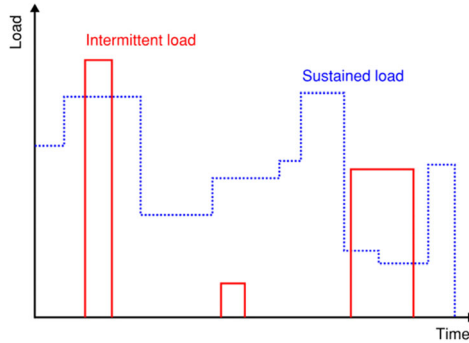


Figure 3. Components of live load—sustained and intermittent load (the total live load represents the sum of these two parts), adapted from [16].

Table 5 Sustained Office Live Load Data [26]

Area (m ²)	μ (psf)	μ (kN/m ²)	δ
18.6	11.6	0.56	0.85
37.2	11.6	0.56	0.68
92.9	11.6	0.56	0.55
185.8	11.6	0.56	0.50
464.5	11.6	0.56	0.47

Table 6 Sustained Office Live Load Data [32]

Area (m ²)	μ (psf)	μ (kN/m ²)	δ
1.1	11.8	0.56	1.28
1.4	11.8	0.56	1.10
2.4	11.8	0.56	0.98
5.2	11.8	0.56	0.82
14.0	11.8	0.56	0.69
31.2	11.8	0.56	0.56
58.0	11.8	0.56	0.51
111.2	11.8	0.56	0.45
192.2	11.8	0.56	0.38

load” is considered which would make the data in Table 5 correspond to $\mu_Q = 0.18 Q_{nom}$. When considering Q_{nom} equal to 2 kN/m² or 3 kN/m² as in EN 1991-1-1 [29], the ratio is 0.28 and 0.19 respectively. A Gamma distribution was

assumed by Ellingwood based on [41, 42]. McGuire [41] references Peir [40] for showing that the live load on different areas can be presented well by a Gamma distribution. Peir, in both of his mentioned works [40, 42], introduces the Gamma distribution as the best graphical fit for the data presented in the UK survey of office loads [32].

The study by Chalk and Corotis [27] published in 1980 is more extensive and lists the mean APIT sustained live loads and variability for different occupancy types, taking into account data from multiple surveys (summary results presented in Table 7). Comparison with the office load data listed in Table 5 confirms the order of magnitude values. Furthermore, Chalk and Corotis also apply a Gamma distribution in their calculations, seemingly referencing [43]. However, as part of the review study presented here, no mention of the Gamma distribution could be found in [43].

Iqbal and Harichandran [8] refer to Ravindra and Galambos [25] for a tributary area dependent bilinear formulation of the COV for office buildings. The formulation listed in [8] however decreases rapidly for larger floor areas (e.g. the COV reduces to only 0.07 at 465 m² vs. 0.47 in Table 5). Ravindra and Galambos again refer to [42] for the live load modelling, and refer to other papers for the background of the area-dependent COV formulation. As part of the review presented here, the bilinear equation of the COV in function of floor area could not be found in any of those references. However, taking into account [40], it is observed that the equation represents a bilinear fit to the data from the UK office survey [32]. This observation makes the equations valid only for loaded areas smaller than 200 m² (barring extrapolation outside the data range), explaining the rapid decrease in COV to unrealistically low values for larger floor areas.

The second family of APIT live load models in Table 2 consider a 5y Gumbel distribution. The listed references refer to Eurocode background documents and the 2010 review of stochastic models by Holický and Sýkora [30]. Considering [30], the 5y Gumbel distribution specified in Table 2 relates to office buildings designed in accordance with the Eurocode recommended nominal (characteristic) imposed load of 2 kN/m² to 3 kN/m². The factor of 0.2 listed in Table 2 then corresponds with a mean value of 0.4 kN/m² to 0.6 kN/m² for the sustained live load, which is in agreement both with the values specified above from [25–27] as well as with the mean value of 0.5 listed by Holický and Sýkora [30], referencing [31]. This is an important observation as it implies that—at least for office buildings—the APIT mean load is largely comparable in both live load model families. Nevertheless, differences between countries may exist, see e.g. the CIB report [39]. Holický and Sýkora further indicate that the COV can vary greatly, referring to [31, 39], and that it decreases with the area. This is in agreement with the observations in Tables 5 and 6. Although calibrated for typical office buildings, Holický and Sýkora indicate the model parameters can be used for other occupancies as a first approximation. The Gumbel distribution itself originates from considerations of maximum load for a given reference period, where a Gumbel distribution is fitted to the upper quantiles of the maximum load. Amongst others, Ellingwood and Culver also use the Gumbel distribution for this purpose [26].

Table 7
Sustained Live Load Data [27]

Occupancy	μ (psf)	μ (kN/m ²)	δ	Q_{nom}^* (kN/m ²)	μ/Q_{nom} (-)
Office	10.9	0.52	0.70	3	0.17
Residential	6.0	0.29	0.57	2	0.14
Hotel (room)	4.5	0.22	0.33	2	0.11
Retail (first floor)	17.9	0.86	0.31	5	0.17
Retail (upper floors)	12.0	0.57	0.88	5	0.11
Classroom	12.0	0.57	0.25	3	0.19
Warehouse	71.5	3.42	0.90	7.5	0.46

*Recommended values EN 1991-1-1 [29]

The PMC however specifies a Gamma distribution for the instantaneous sustained load [31], as noted also in [30]. As confirmed by the analysis of Honfi [44], the imposed load models in the current Eurocodes are in agreement with the PMC load model recommendations, except for classrooms and retail spaces. For a further in depth analysis of the relationship between the PMC recommendations and current standards, further reference is made to [44].

The PMC tabulates distribution parameters for different occupancies, reprinted in Table 8 below for the same occupancies as in Table 7. The ratio μ/Q_{nom} are comparable to those listed in Table 7. While a high ratio is found for warehouses, all other categories give values in the range 0.15 to 0.20. The PMC only lists a limited number of general references for the live load model: an earlier draft of the Model Code, an unpublished document, a draft of [29, 39].

The original draft of the 1989 CIB report [39] was prepared by Corotis and Sentler, indicating a (reasonably assumed) continuation of the work listed in Table 7. The report itself gives an overview of multiple surveys dating from 1893 to 1976, and amongst others refers to (earlier work by) the JCSS, [27], and a review by Sentler [45]. Of these references, only [27, 45] could be obtained as part of the current review study.

The standard deviation of this instantaneous imposed load model is calculated by Eq.(9), with σ_V the standard deviation of the overall load intensity, σ_U the standard deviation associated with the spatial variation of the load, A_0 an occupancy-specific reference area, A the loaded area, and κ an influence factor (commonly between 1 and 2.4; further taken as 2.2 for agreement with Ellingwood and Culver [26] based on their consideration of load effects on columns for their analysis).

$$\sigma^2 = \sigma_V^2 + \sigma_U^2 \kappa \cdot \min\left\{\frac{A_0}{A}; 1\right\} \quad (9)$$

The COV for very large loaded areas is listed in Table 8 as δ_{inf} , i.e. where the loaded area dependent term in Eq. (9) reduces to zero. With the exception of the first-floor retail space, these COV are smaller than those listed in Table 7. For

Table 8
Sustained Live Load Parameters [31]*

Occupancy	A_0	μ	σ_V	σ_u	δ_{inf}^{**}	μ/Q_{nom}
Office	20	0.5	0.3	0.6	0.60	0.17
Residential	20	0.3	0.15	0.3	0.50	0.15
Hotel (room)	20	0.3	0.05	0.1	0.17	0.15
Retail (first floor)	100	0.9	0.6	1.6	0.67	0.18
Retail (upper floors)	100	0.9	0.6	1.6	0.67	0.18
Classroom	100	0.6	0.15	0.4	0.25	0.20
Warehouse (storage)	100	3.5	2.5	6.9	0.71	0.47

*Dimensions (m²), (kN/m²), (-); Q_{nom} taken as recommended values EN 1991-1-1 [29], as listed in Table 7

**COV corresponding with large loaded area A , for which the area-dependent term in Eq. (9) reduces to zero

small loaded areas, however, the COV resulting from Eq. (9) exceed those in Table 7. The comparison is facilitated by Fig. 4, where the COV calculated from Eq. (9) and Table 8 is listed in function of the loaded area A . For comparison, also the COV data listed for office buildings in Table 5 is given by the scatter plot, highlighting a clear difference between both references.

The COV of 1.1 used in [14, 15] is found to correspond with JCSS recommendations for offices at approximately 75 m² loaded area (for $\kappa = 2.2$ as in [26]; Holický and Sýkora use $\kappa = 1.4$ and obtain this result for an office loaded area of 50 m²). Note that a different assumption with respect to the influence factor κ in Eq. (9) is equivalent to a change in loaded area A . A COV of 0.95 is obtained for offices and classrooms for a loaded area of approximately 120 m². The JCSS PMC further notes that one of the underlying assumptions for the equivalent uniformly distributed load model of Eq. (9) is a linear structural response [31]. The assumption of linearity can be omitted by considering the spatial variability of the load explicitly. The latter is however considered too demanding for practical feasibility. Nonlinear behaviour could be considered as part of the model uncertainty K_E .

4.3. Probabilistic Model

The background documents agree on the use of a Gamma distribution to describe the instantaneous sustained live load. Although the origin of this assumption was identified above as being a graphical fit for the data of the 1971 UK office load survey [32], it is adopted here as a recommendation based upon precedent and considering the impossibility of negative values. For $COV \geq 1$, the adoption of a Gamma distribution results in an exponential shape for the imposed load PDF.

The different background studies largely agree on the mean value μ for the sustained live load. When defining Q_{nom} through EN 1991-1-1 recommended values, the mean to nominal load ratio (μ/Q_{nom}) is largely found to be in the range 0.10-0.20 (similar when considering the ASCE 7-16 office recommendation of 65 psf). A value of 0.20 is considered reasonable for a first assessment of offices, residential areas, retail, hotels, and classrooms. The appropriateness of this ratio depends

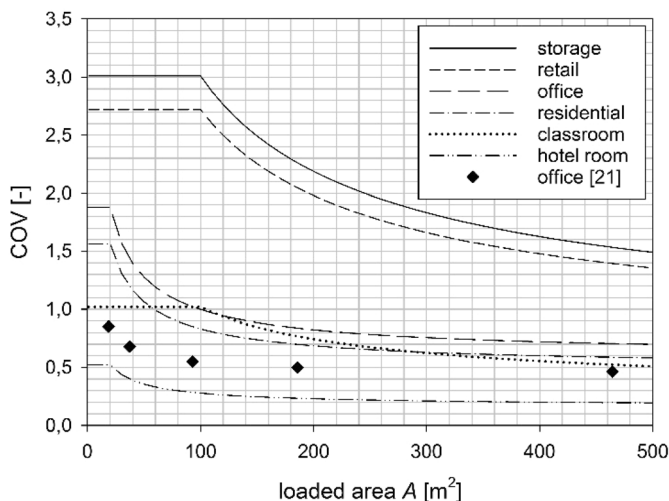


Figure 4. COV of the instantaneous sustained live load as a function of the load area A , in accordance with the Probabilistic Model Code [31], for $\kappa = 2.2$.

on the value of Q_{nom} . For specific projects, a direct definition of the mean APIT live load is recommended. Warehouses are excluded from the above recommendation and should be considered as a special case. For detailed analysis on warehouse load models, reference is made to the study by Lenner and Sýkora [46].

Considering the background study above, a large variation and load-area-dependency is found for the COV of the sustained live load. A value of 0.60, as commonly used, is found reasonable for large loaded areas (Table 8). For smaller loaded areas, this may be too low. In the absence of a direct evaluation, a COV of 0.95 is recommended for residential buildings and office buildings. This COV recommendation is based on Fig. 4 and supported by a calibration of the reliability index for normal design situations, as described further.

For structural elements in multi-story buildings, correlation between the live loads on different storeys must be discussed as vertical load bearing elements such as columns and transfer structures potentially carry live loads originating from multiple floors. In the case where all the floors are occupied by a different owner, or are used for different purposes, it can be assumed that the live loads on each floor are independent from each other. In that case unique live load realizations can be used for each floor. This independence of live load realizations on each floor reduces the occurrence probability of large total load realizations. However cases where two or multiple floors are used for similar purposes, e.g. by the same tenant, it is reasonable to assume a level of correlation between the live load realizations. In such cases, an assumption of perfect correlation is conservative as it results in a higher probability of larger total load realizations. Consequently, when the details of the occupancy per floor are not known, perfect correlation of

the live load for all floors (i.e. a single live load realization) is recommended for the analysis of a multi-story element.

5. Total Load Effect Model and Recommendation

5.1. Total Load Model

Two formulations have been identified above for combining the permanent load and live load, i.e. Eqs (4) and (5). These formulations also include stochastic factors accounting for uncertainties in the transformation of loads into load effects. This has been discussed to some extent in Sect. 2.2 above, indicating that limited background could be found for either of the models. Despite its regular use, the model applied by Ravindra and Galambos [25] is considered not to have the same authority as the general formulation applied by the JCSS, which is the common expert group on structural reliability of 6 international organizations (CIB, ECCS, *fib*, IABSE, RILEM and IASS; for completeness, the full organization names not mentioned earlier are: European Convention for Constructional Steelwork, International Association for Bridge and Structural Engineering, Réunion Internationale des Laboratoires et Experts des Matériaux, systèmes de construction et ouvrages, and International Association of Shell and Spatial Structures). Hence, the total load model of Eq. (2) is recommended for the total load effect, considering a lognormal model uncertainty K_E with mean of 1.0 and COV of 0.10.

This conclusion allows to present the calibration supporting the recommendation of a COV of 0.95 for the live load model in case of small load areas. The calibration is based on a comparison of the reliability index for normal design situations as obtained considering (i) the recommended Gamma distribution for the live load, with COV to be determined, and (ii) the traditional Gumbel distribution with COV of 1.1 in agreement with current Eurocode background documents for normal design conditions. A Monte Carlo procedure is conducted for a generalized test model, Eq. (10) where Z is the limit state. K_R represents the model uncertainty of the resistance effect and is considered here as a lognormal distribution with mean value of 1.2 and COV of 0.15. Note that this model uncertainty relates to normal design conditions. R is the resistance effect modelled as a lognormal distribution with COV of 0.15 and mean value calculated in accordance with the Annex C of EN 1990 [23], i.e. through Eq. (11) where R_d is the design resistance effect and β is set equal to 4.7 (the Eurocode target value of the reliability index for the ultimate limit state, considering a reference period of 1 year). Φ is the standard normal cumulative distribution function and $\mu_{\ln R}$ and $\sigma_{\ln R}$ are lognormal distribution parameters for the resistance effect.

$$Z = K_R R - K_E (G + Q) \geq 0 \quad (10)$$

$$P[R \leq R_d] = \Phi(-0.8\beta) = \Phi\left(\frac{\ln R_d - \mu_{\ln R}}{\sigma_{\ln R}}\right) \quad (11)$$

Figure 5 shows the obtained reliability index as a function of the load ratio χ , considering the traditional Gumbel distribution on the one hand and Gamma distributions with different COV on the other hand. From this visualization, a COV of 0.95 for the live load Gamma distribution results in a close agreement of the reliability index, as compared with the Gumbel model which has been applied in the background documents of the Eurocode. The preference for a COV of 0.95 is confirmed in Fig. 6 where the COV is evaluated for which the reliability index based on the Gamma distribution for the live load matches the reliability index obtained when using the Gumbel model with a COV of 1.1. It is thus concluded that the COV of 0.95 represents a good approximation for different ranges of load ratio χ as it provides a comparable reliability level. The COV of 0.95 results in a slightly higher reliability level than the reference Gumbel model, and is thus considered as a slightly conservative approximation. A COV of 0.90 would on the other hand be too low. For warehouses the COV may be significantly higher (see Sect. 4.2).

Taking into account the analyses above, an overview of the recommended model parameters for a non-project-specific evaluation of a normal occupancy building (e.g. office) with a small/large loaded area is listed in Table 9, to be applied with the total load model of Eq. (2), or equivalently (5), reprinted below as Eq. (12) for clarity. The obtained total load model is visualized in Fig. 7.

$$w = K_E(G + Q) = P_{nom}K_E((1 - \chi)g + \chi q) \quad (12)$$

In order to determine the COV value of the instantaneous sustained live load in a specific design, it is recommended to use the COV value of 0.95 for areas smaller than 100 m² and a COV value of 0.6 for areas bigger than 200 m². For intermediate floor areas a linear interpolation of COV value is recommended based on Fig. 4 and considering continuity of recommended COV value.

5.2. Approximation for the Total Load Effect

For simplicity, and to allow for the development of semi-probabilistic design approaches as in [47], it is convenient to approximate the total load distribution with a single theoretical distribution. To find out the best fit for the recommended total load model, Normal, Lognormal, Gamma and Gumbel distributions were compared using the AIC criterion [48] for the load ratios ranging from 0.1 to 0.9. Results are visualized in Fig. 8, where the vertical axis represents the negative AIC value. The lower the AIC (i.e. the higher the bar in the chart, since values are negative), the better the fit is. As indicated in Fig. 8, the lognormal distribution is the best approximation for most of the cases.

Figure 9 visualizes the CDF and cCDF for the total load model and its lognormal approximation for $\delta_Q = 0.95$, whereas Fig. 10 visualizes the corresponding PDF. As can be seen from this Fig. 9, the approximation is a much better fit for lower values of the load ratio, i.e. when the normally distributed permanent load constitutes a larger fraction of the total load. Especially for large quantiles of the CDF (low quantiles of the cCDF), a large difference between the total load model

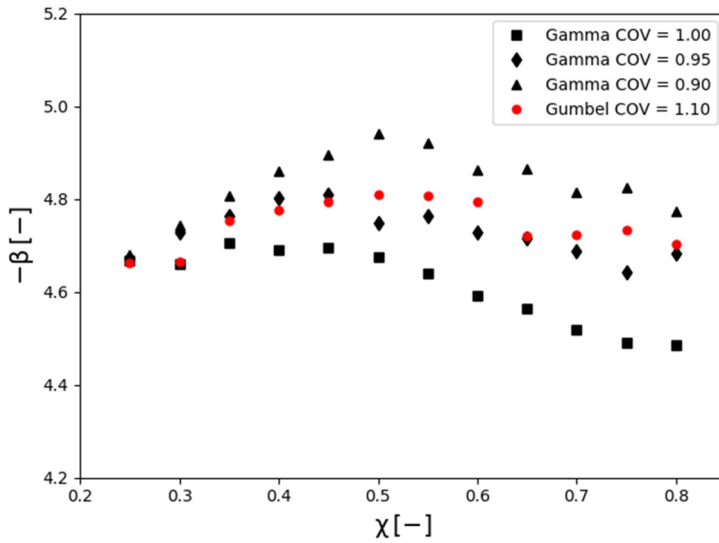


Figure 5. Normal design reliability index in function of the load ratio χ_r for different live load models: Gumbel distribution with COV = 1.1, and Gamma distribution with COV of 0.90/0.95/1.00.

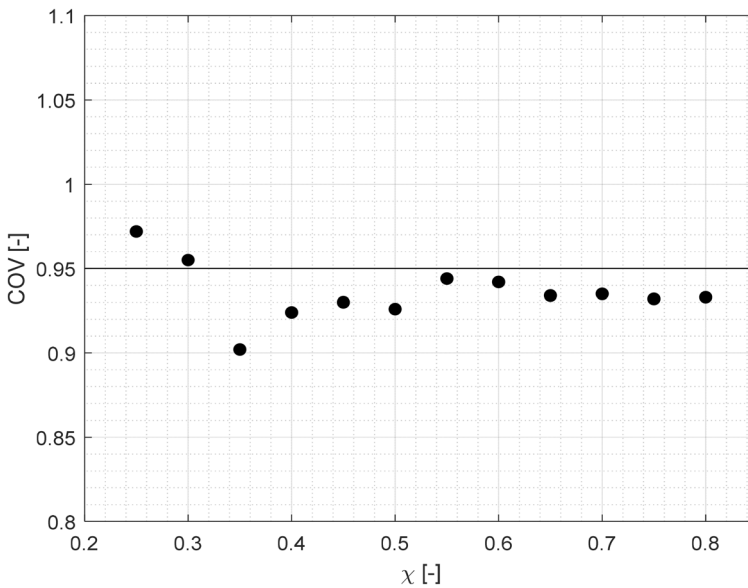
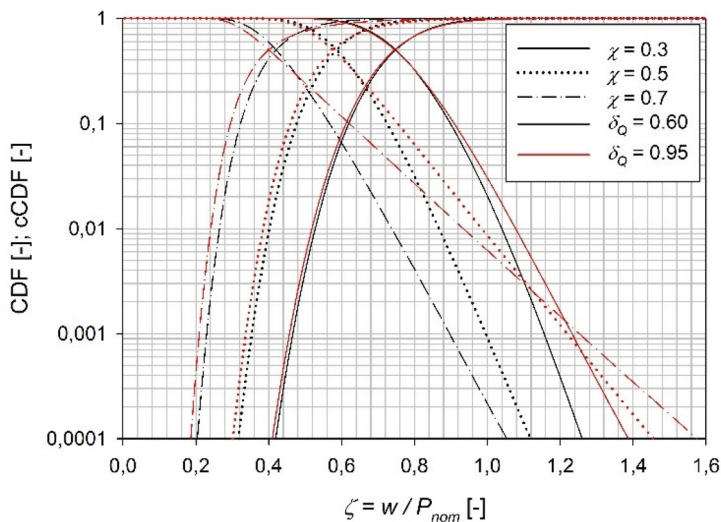


Figure 6. COV for the live load model, for which the normal design reliability corresponds to the same reliability index as for the reference Gumbel model with a COV of 1.1.

Table 9
Recommended Load Models for PFSE

Symbol	Load component	Distribution	μ/nom	δ
K_E	Load effect model uncertainty	Lognormal	1.0	0.1
G	Permanent load	Normal	1.0	0.1
Q	Instantaneous sustained live load (small/large loaded area)	Gamma	0.2	0.95/0.6

**Figure 7. CDF and cCDF for the total nominal load factor ζ according to the reviewed load models, with COV $Q = 0.60$ (black) and 0.95 (red) respectively (Color figure online).**

and its lognormal approximation is observed. From this graphical evaluation, it is tentatively concluded that a lognormal approximation should be applied with caution. This will especially be the case for situations with a low failure probability, where the difference for large CDF quantiles will have a clear effect. This is demonstrated further in the application cases.

6. Applications

To demonstrate the influence of the selected load model, failure probabilities are calculated for steel and concrete columns considering the three total load models listed in this paper. The first model is the one used by Guo and Jeffers [7], as defined by Eq. (1) (Model A). The second one is the model used by Gernay et al. [15], as defined by Eq. (2) (Model B). The final total load model is the model recommended in this paper (Model C), as summarized in Sect. 5.1 above.

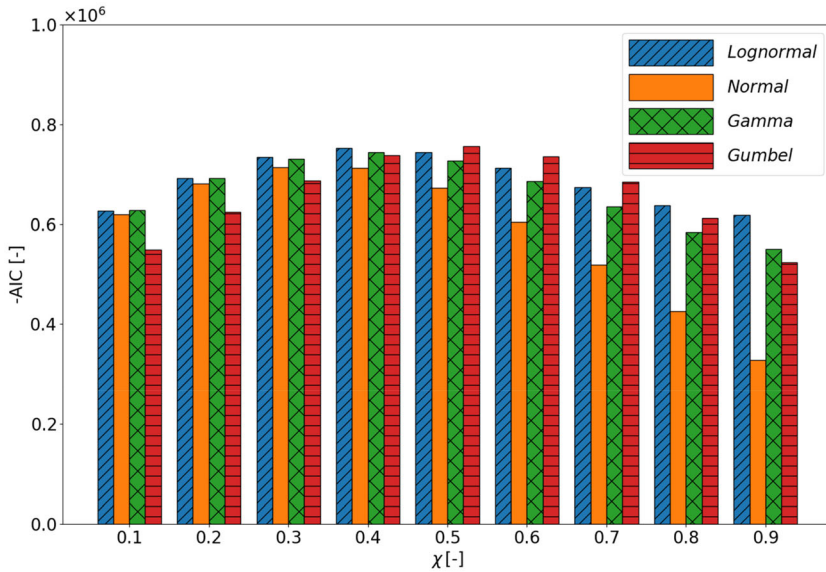


Figure 8. AIC for four different approximations of the recommended total load model.

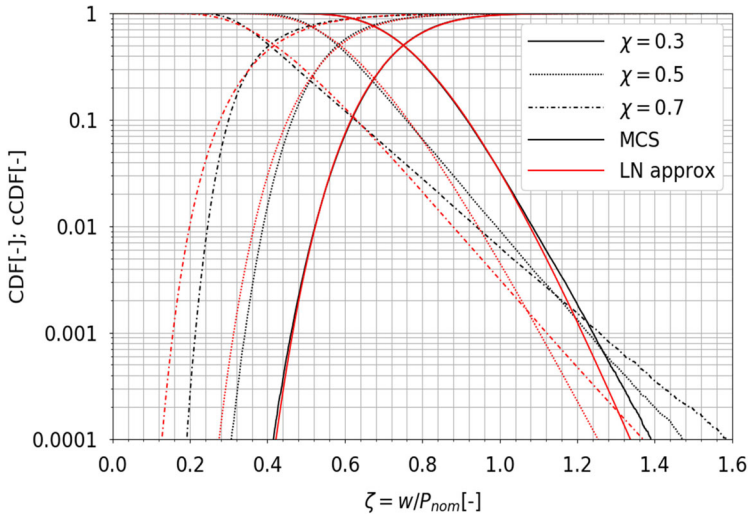


Figure 9. CDF comparison of the total load model (MCS) and its Lognormal approximation.

6.1. Steel Columns

Four steel columns are considered in this analysis. The selected columns are from the first, second, seventh, and eighth story of a 9-story office building designed

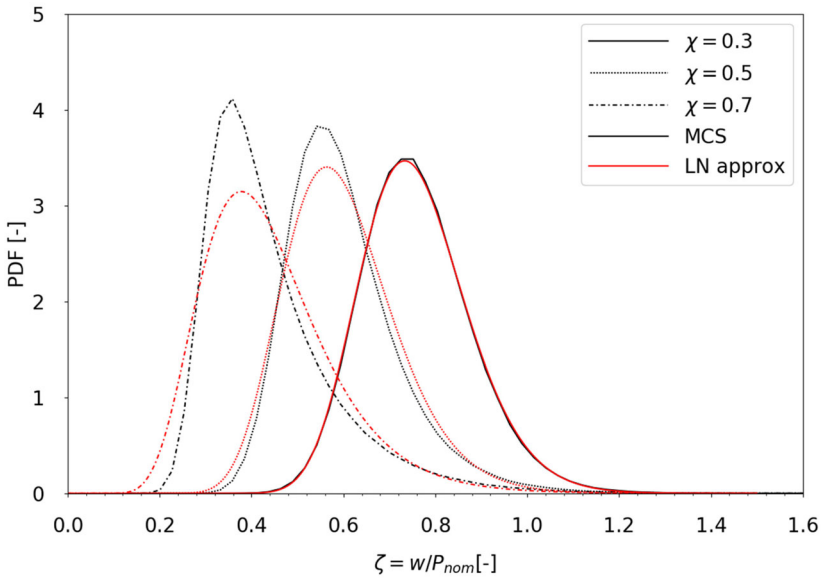


Figure 10. PDF comparison of the total load model (MCS) and its Lognormal approximation.

according to US guidelines. More details on the design and geometry of the building are presented in [49]. The columns have a spray applied fire resistance material (SFRM) with thickness determined by the 2-hour fire resistance rating requirement according to Section 722.5.1.3 of the ICC IBC-2018 [50], with details presented in Table 10.

The capacity of the columns after 2-hour standard fire exposure is assessed based on the simplified method from Appendix 4 of AISC [51]. The capacity is evaluated based on the assumption that support and restraint conditions remain unchanged during the fire exposure. The temperature of the steel member is calculated using the lumped heat capacity analysis. The nominal compression strength is calculated using the equation:

$$F_{cr} = 0.42 \sqrt{\frac{F_y(T)}{F_e(T)}} \times F_y(T) \quad (13)$$

where $F_y(T)$ is the yield stress at elevated temperature and $F_e(T)$ is the critical elastic buckling stress. The thickness, thermal conductivity, specific heat and density of the insulation material and the strength retention factor and modules of elasticity of steel are treated as stochastic variables. Their probabilistic description is presented in Table 11, taking into account the formulations proposed by [8, 52]. In these formulations, T represents the temperature in degrees Celsius, $k_{y,EN}$ is the yield strength retention factor listed in EN 1993-1-2 [53], and ε is a standard normally distributed variable (independent for each of the material parameters). The capacity obtained through Eq. (13) is multiplied with the random parameter K_R

Table 10
Column Sections, Insulations Thickness and Loads

Column	Insulation thickness (mm)	Gk (kN)	Qk (kN)
W14x109-1st floor	26.9	3692	689
W14x109-2nd floor	26.9	3285	608
W14x53-7th floor	31.8	1253	219
W14x53-8th floor	31.8	849	148

Table 11
Probabilistic Models Used for the Analysis of Steel Columns

	Distribution	Mean value	COV
SFRM— t_p (mm)	Lognormal	Nominal + 1.6 mm	0.2
ε	Normal	1	1
SFRM— k_p (w/m K)	$k_p = \exp(-2.72 + 1.89 \cdot 10^{-3}T - 0.195 \cdot 10^{-6}T^2 + 0.289 \cdot \varepsilon)$		
SFRM— ρ_p (kg/m ³)	$\rho_p = \exp(-2.028 + 7.83 \cdot 10^{-0.0065}T + 0.122 \cdot \varepsilon)$		
SFRM— c_p (J/kg K)	$c_p = 1700 - \exp(6.81 - 1.61 \cdot 10^{-3}T - 0.44 \cdot 10^{-6}T^2 + 0.213 \cdot \varepsilon)$		
Steel— $k_{y,\theta}$ (-)	$k_{y,\theta} = \frac{1.7 \times \exp[r_{\text{logit}} + 0.412 - 0.81 \times 10^{-3} \times T + 0.58 \times 10^{-6} \times T^{1.9} + 0.43 \times \varepsilon]}{\exp[r_{\text{logit}} + 0.412 - 0.81 \times 10^{-3} \times T + 0.58 \times 10^{-6} \times T^{1.9} + 0.43 \times \varepsilon]} + 1$ where $r_{\text{logit}} = \ln\left(\frac{(k_{y,EN} + 10^{-6})/1.7}{1 - (k_{y,EN} + 10^{-6})/1.7}\right)$		
Steel— $k_{E,\theta}$ (-)	$k_{E,\theta} = \frac{1.1 \times \exp[2.54 - 2.69 \times 10^{-3} \times T - 2.83 \times 10^{-6} \times T^{1.9} + 0.36 \times \varepsilon]}{\exp[2.54 - 2.69 \times 10^{-3} \times T - 2.83 \times 10^{-6} \times T^{1.9} + 0.36 \times \varepsilon]} + 1$		

which represents uncertainty of the model. This model uncertainty is defined as a Lognormal distribution with mean value of 1 and COV of 0.15, as applied in [47] based on ambient design model uncertainties and conceptual considerations of the effect of fire. This model for K_R is indicative as currently no comprehensive recommendations exist for model uncertainties of resistance models for fire-exposed structures. These values must thus be looked at with a sense of reservation, and not be considered as recommendations. One of the few studies exploring K_R for structural fire applications is the study by Achenbach et al. [54], but this remains an area in need of further research. Considering the above, failure is defined by the following equation, with A as the cross-section area and w the total load effect:

$$Z = K_R F_{cr} A - w < 0 \quad (14)$$

For each column, 10^4 Monte Carlo simulations (MCS) of the column capacity at 2-hour standard ASTM fire exposure are performed. On the other hand 10^8 MCS realizations are made for the total load effect w according to each load model. Probabilities of failure of all for all four cases are presented in the Table 12.

It can be observed that for these cases the failure probability does not differ between Model C (the proposed model) and Model B. On the other hand, Model

Table 12
Failure Probabilities Steel Columns Calculated Using Different Total Load Models

Column	Insulation thickness (mm)	$P_{f_{\text{Model A}}}$	$P_{f_{\text{Model B}}}$	$P_{f_{\text{Model C}}}$	$P_{f_{\text{Model C}} - \text{LN approx}}$
W14x109-1st floor	26.9	0.5861	0.5326	0.5326	0.5324
W14x109-2nd floor	26.9	0.2911	0.2624	0.2623	0.2623
W14x53-7th floor	31.8	0.4667	0.4313	0.4313	0.4312
W14x53-8th floor	31.8	0.2522	0.2292	0.2291	0.2291
Concrete column	n/a	0.0084	0.0076	0.0071	0.0067

A predicts failure probabilities which are approximately 10% higher. While the evaluations in Table 12 show that the choice of load model has an influence on the failure probability, it should be noted that the obtained failure probabilities are relatively large, resulting in limited practical difference. For these high failure probabilities, the lognormal approximation for the total load effect (see Sect 5.2), given in the last column of Table 11, provides a reasonable approximation for P_f .

6.2. Concrete Column

The three total load models were also compared with respect to the failure probability of a concrete column. The column in question is the part of the Eurocode reference concrete building per Biasoli et al. [55], as evaluated for performance-based structural fire engineering in ISO/TR 24679-6:20174 [56]. The column has dimensions of $500 \times 500 \text{ mm}^2$ and a height of 4 m, and is made of concrete C30/37 with siliceous aggregates, reinforced with 12 longitudinal rebars of diameter 20 mm with 42 mm of concrete cover. The full probabilistic analysis of the column capacity at 2 h of standard ISO-834 fire exposure was done using nonlinear finite element analysis and is discussed in detail in [47]. Compared to the evaluation in [47], two additional random parameters are considered in the following, more precisely (i) the temperature dependent concrete strength retention factor, modelled by the logistic model proposed in [20], and (ii) the concrete cover, modeled as a beta distribution with the mean value equal to nominal value increased by 5 mm and a standard deviation of 5 mm, and bounded by $[c_{\text{nom}} - 10 \text{ mm}, c_{\text{nom}} + 20 \text{ mm}]$ (i.e. a symmetrical Beta distribution bounded by ± 3 times the standard deviation), based on the recommendation in the JCSS PMC [57].

A total of 10^4 MCS simulations of the column capacity are compared with the 10^8 MCS realizations for each load model. The obtained failure probabilities are presented in Table 12. In this case, a larger difference is observed when comparing Models B and C with Model A. Similar to the application example for steel columns, Model A, provides a larger probability of failure. The lowest failure probability is obtained for the recommended total load model (Model C), suggesting conservativeness of the load models traditionally applied. Model B results in a failure probability assessment which is 7% higher, while Model A gives an 18%

higher evaluation. The lognormal approximation of Model C however underestimates the failure probability by approximately 6%. Overall, the order of magnitude for all total load models is found to be comparable for the investigated case.

The difference in the failure probabilities between the Models B and C as observed for the concrete column example and not for the steel column example can be attributed to two factors. Firstly, the difference between models B and C is smaller for lower values of the load ratio ($\chi = 0.4$ in the case of concrete column and $\chi \approx 0.15$ for the steel columns). Secondly the obtained failure probabilities for the steel columns are more than an order of magnitude larger than in the case of the concrete column. This reduces the effect of the differences between the two

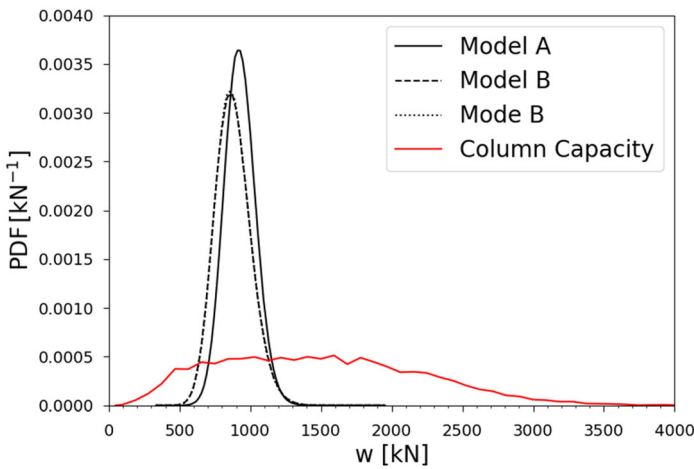


Figure 11. PDFs of the load models and the steel column capacity W14x53 on 7th floor.

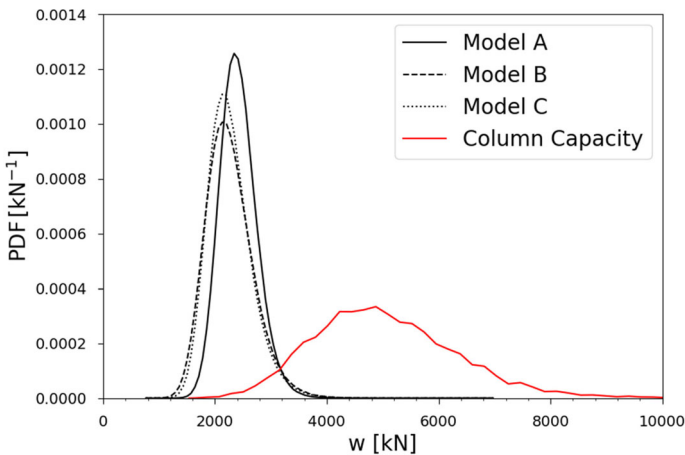


Figure 12. PDFs of the load models and concrete column capacity.

load models. This is illustrated by Figs. 11 and 12 where PDFs of all three load models and of the column capacity are presented, for the steel column W14x53 on 7th floor and the concrete column respectively.

7. Conclusions

Probabilistic models for describing the permanent load, live load, and total load have been reviewed, as applied in probabilistic structural fire engineering. It is concluded that the total load effect is ideally described by $K_E(G + Q)$, with K_E the model uncertainty for the load effect, G the permanent load, and Q the imposed load. The model uncertainty K_E can be described by a lognormal distribution with mean equal to unity and coefficient of variation (COV) of 0.10. The permanent load is preferably modelled by a normal distribution with mean equal to the nominal permanent load, and a COV which can either be assessed on a project specific basis, or can be set to 0.10 for a first assessment. For material with a larger uncertainty, such as natural wood, a larger COV may be required. The arbitrary-point-in-time imposed load effect Q can be modelled by a Gamma distribution. The mean value and COV of Q are however found to be highly influenced by the occupancy type. It is recommended to explicitly list the mean value and COV of the applied live load model. Traditional formulations describe parameters of live load distribution as a fraction of the nominal live load Q_{nom} , which can be confusing when considering different guidance documents' definition of the nominal live load. For common occupancies (office, residential), the mean live load can be taken as 0.2 times the nominal, with a COV of 0.60 for large load areas ($> 200 \text{ m}^2$) and 0.95 for smaller load areas ($< 100 \text{ m}^2$). Linear interpolation of the COV is suggested for intermediate floor areas. This recommendation does not apply to imposed loads in warehouses as these should be considered as a special case. The recommended load models are, however, based on historical data, and no recent load surveys were obtained. The recommended live load models have not been derived to apply for the special case of evacuation routes. Notably, overcrowding near emergency exits during the evacuation process is not taken into account. Furthermore, a comparison of load models indicates that it is not recommendable to approximate the total load model with a single theoretical distribution. The single distribution does not approximate the higher quantile values well, which can be of importance when considering small failure probabilities. It is envisaged that consolidated probabilistic load model proposed here will achieve a universal way of representing mechanical loads in PSFE studies, resulting in greater accuracy and comparability.

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