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ORIGINAL ARTICLE

Probabilistic models for live loads in buildings: critical review, comparison to Brazilian design standards and calibration of partial safety factors

Modelos probabilísticos para a ação de utilização em edifícios: análise crítica, comparação com normas brasileiras de projeto, e calibração dos fatores parciais de segurança

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Abstract: Nowadays, structural "limit state" design is made using characteristic or nominal values of actions, partial safety factors and load combination factors. The actual loading that a structure will be subjected to throughout its life is not known at the design phase. Yet, probabilistic models of such loadings are useful for the rational determination of partial safety factors and load combination factors. The probabilistic model leading to nominal live loads of NBR 6120:2019 (Design Loads for Structures) has never been openly discussed. Herein a simple probabilistic model describing spatial and temporal variabilities of live loads in buildings is presented and discussed. The model is built as a sum of two stochastic processes representing the sustained and intermittent parts of the live load. Model parameters are the ones recommended by the Joint Committee on Structural Safety (JCSS), based on extensive surveys done in several countries. By way of Monte Carlo simulations, sample values of live load actions are obtained for buildings of different occupancy types. These values are compared with those recommended by international standards, and those recommended in NBR 6120:2019 and NBR 8681:2003 (Actions and Safety of Structures). The corresponding statistics for the fifty-year extreme and arbitrary point-in-time distributions of live loads are presented; these statistics are very relevant for reliability analyses and for reliability-based code calibration. The stochastic live load model is also employed in a reliability-based calibration to obtain partial safety factors and load combination factors to be used in Brazilian design codes, for ultimate and serviceability limit state verifications.

Keywords: live load model, probabilistic model, structural reliability, partial safety factor, load combination factor, NBR 8681, NBR 6120.

Resumo: Hoje em dia, o projeto estrutural baseado em estados limites é feito utilizando valores característicos ou nominais das ações, fatores parciais de segurança e fatores de combinação de ações variáveis. As ações às quais uma estrutura estará sujeita durante sua vida não são conhecidas com exatidão na fase de projeto. Neste contexto, modelos estocásticos das ações são úteis para a determinação racional dos fatores parciais de segurança, e dos coeficientes de combinação de ações. O modelo probabilístico que levou aos valores nominais das ações de utilização da NBR 6120: 2019 (Ações para o Cálculo de Estruturas de Edificações) nunca foi discutido abertamente. Neste artigo, apresenta-se uma revisão crítica de um modelo estocástico simples que descreve as flutuações espaciais e temporais da ação variável de utilização em prédios. O modelo

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é construído como uma soma de dois processos estocásticos, representando as parcelas sustentada e intermitente da ação de utilização. Parâmetros para este modelo são recomendados pelo *Joint Committee on Structural Safety* (JCSS), com base em *surveys* realizados em diversos países. Utilizando simulações de Monte Carlo, amostras de ações de utilização são obtidas para edifícios com diferentes tipos de ocupação. Estes valores são comparados com aqueles recomendados em diferentes normas técnicas internacionais, bem como com valores preconizados nas normas brasileiras NBR 6120:2019 e NBR 8681:2003 (Ações e Segurança nas Estruturas). As estatísticas obtidas para as distribuições de probabilidade das ações "extrema de cinquenta anos" e de "ponto arbitrário no tempo" são apresentadas; estas distribuições são extremamente importantes em análises de confiabilidade. O modelo estocástico também é empregado em uma calibração baseada em confiabilidade dos coeficientes parciais de segurança e fatores de combinação de ações variáveis das principais normas de projeto brasileiras.

Palavras-chave: ação de utilização, modelo estocástico, confiabilidade estrutural, coeficiente parcial de segurança, fator de combinação de ações, NBR 8681, NBR 6120.

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1 INTRODUCTION

1.1 Background

In order to achieve consistent safety levels in the design of structures, while also meeting economical, functionality and robustness criteria, the engineer must have proper knowledge of the strength properties of materials and structural elements, but also of the loads to which a structure is expected to be subjected throughout its lifetime.

Among these loads, one of the most fundamental when designing buildings is the live load (sometimes also referred to as imposed load), generally specified in design codes as a uniformly distributed load depending on floor occupancy type. In Brazil, design live loads are prescribed by design code NBR 6120:2019 [1].

The nominal values of live loads given by most major foreign design codes are generally based on probabilistic models built from data measured in live load surveys. Extensive reviews of survey results are reported by Sentler [2] and Chalk and Corotis [3], covering investigations conducted between years 1893 and 1976 and covering many occupancy types in six different countries: Australia [4], United States [5]–[13], Finland [14], United Kingdom [15]– [18], Hungary [19] and Sweden [20], [21].

To the authors best knowledge, there are no records of any similar live load surveys carried out in Brazilian buildings, nor of the existence of stochastic models that may have been used to derive the nominal values for live loads presented in NBR 6120:2019. Instead, those values were established by consensus of the technical community, based upon comparisons with foreign design codes, such as the American ASCE/SEI 7-16 [22], the European EN 1991-1-1:2002 [23], and ISO 2103:1986 [24].

1.2 Representative values of variable actions

When designing a structure using a semi-probabilistic approach, such as the limit states format employed by most design codes, representative values of the loads are considered. These can be characteristic or nominal values, design values, or combination values used in ultimate or serviceability limit states. The definitions of these values are given in the Brazilian design code for Actions and Safety of Structures, NBR 8681:2003 [25].

Particularly for the live load, the recently superseded version of the design loads code, NBR 6120:1980 [26], did not mention the return period corresponding to the nominal values proposed. The current version, which came into effect late 2019, repeats the definition found in NBR 8681:2003: "the characteristic values of variable actions, established by consensus, correspond to values that have between 25 to 35% probability of being exceeded, in the unfavorable sense, in a period of 50 years". Furthermore, NBR 6120:2019 adds to this definition, stating that these probabilities correspond to an average return period between 174 and 117 years, respectively.

The definition used in this study is that the characteristic value L_k of the live load corresponds to the 70th percentile (i.e., the value that has 30% exceedance probability) of the fifty-year extreme live load, denoted L_{50} in this paper. This would be equivalent to saying that the mean return period of L_k is around 140 years. Consequently, L_k is equal to the mode of the 140-year extreme distribution (L_{140}) and can also be obtained as the $1 - 1/140 \approx 99{,}3\%$ fractile of the 1-year extreme distribution (L_1), provided that the annual maxima are independent. This hypothesis of independence in challenged in the sequence.

In the limit state design format, employed in Brazilian structural design codes, the required safety margin is achieved by introducing partial safety factors γ_m that reduce the characteristic value of material strength and γ_f that increase the nominal values of actions (or their effects), resulting in design values.

The safety factor for actions is expressed as the product of three other partial factors, $\gamma_f = \gamma_{f1}\gamma_{f2}\gamma_{f3}$. The first of these, γ_{f1}, takes into consideration possible unfavorable deviations from the representative values due to the inherently variable nature of loads. The second, γ_{f2}, is a load combination factor that takes into account the reduced probability that all actions happen simultaneously with their representative values. Lastly, γ_{f3} accounts for the inaccuracies in the assessment of action effects, whether due to constructive deviations or to shortcomings arising from simplifications assumed in modelling.

Particularly for live loads, the safety factor given by $\gamma_{f1}\gamma_{f3}$, denoted γ_L in this paper, is usually equal to 1.4 when considered grouped with other variable actions, or 1.5 when considered separately, as indicated in NBR 8681:2003. The γ_{f2} factor can be equal to ψ_0, ψ_1 or ψ_2, depending on what limit state is being verified.

The combination factor $\psi_0 \leq 1$, used in verification of ultimate limit states (ULS), takes into account that it is highly unlikely that two (or more) independent variable actions simultaneously present their maximum values of over a reference period. It is calculated so that the probability of the combined effect – due to multiple variable actions – being exceeded during the reference period is somewhat equivalent to the exceedance probability when only a single variable action is considered with its characteristic value.

The frequent and quasi-permanent values, used in verification of serviceability limit states (SLS), are obtained by multiplying the characteristic values by reduction factors $\psi_1 \leq 1$ and $\psi_2 \leq 1$. The frequent value ψ_1 can be defined in two different ways: based on the frequency with which the variable action exceeds this value or based on a small fraction of the total lifetime of the structure in which it is surpassed. NBR 8681:2003 states that the frequent value is that which is exceeded about 10^5 times in a period of 50 years, or during 5% of the structure lifetime. In the present study, the second definition was considered, since it is easier to use. Similarly, the quasi-permanent value ψ_2 is defined so that its total time of application is a considerable portion (around half) of the structure´s lifetime.

2 METHODOLOGY

2.1 Probabilistic model for live loads

Live loads in buildings depends on its corresponding occupancy type, and are intrinsically stochastic in nature, varying in space and time. In general, live loads can be decomposed in two parts with different behavior regarding its temporal variability: a sustained load and an extraordinary load (sometimes also referred to as intermittent or transient load).

The sustained load includes weight of all furniture, equipment, stored objects and personnel that are regularly present in the analyzed area. This load is the one effectively measured in load surveys.

The extraordinary load is associated with exceptional events that may lead to short duration high-intensity loading, such as temporary crowding due to a party or special event; or caused by a large number of people trying to evacuate the building in an emergency situation; or even the relocation and concentration of furniture in a room while the adjacent premises are undergoing renovations. Due to the exceptional and transient nature of extraordinary loads, it is very unlikely that these events can be reliably measured in load surveys.

The model analyzed in this study is the hierarchical model presented in Part 2 of the JCSS Probabilistic Model Code [27], which is based on a formulation initially proposed by Peir [28] and Peir and Cornell [29]. This model has been used with great success by several authors since then, among which McGuire and Cornell [30]; Ellingwood and Culver [31]; and Chalk and Corotis [3]. These were the studies that served as a basis for obtaining the live load statistics used in the calibration of the partial safety factors of North-American design codes in the eighties [32].

The central idea of the model is to represent the sustained load $Q(t)$ by a rectangular-wave process with random intensities and durations (Figure 1a), and the extraordinary load $P(t)$ by a spike process with random intensities and time between pulses (Figure 1b). The total live load is given by the sum of these processes, $L(t) = Q(t) + P(t)$ (Figure 1c). As shown in Figure 1, finding the maximum of the combined process is not a trivial task, since this value may not coincide with any of the individual maxima for each process.

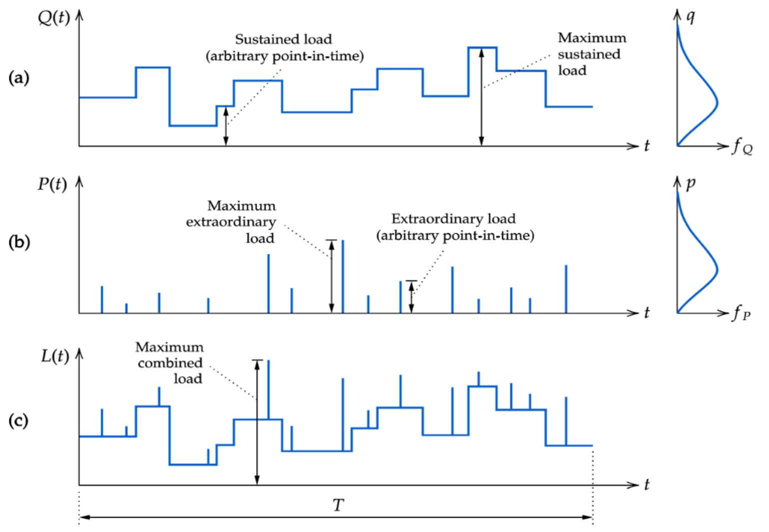

Figure 1. Time histories of live loads: (a) sustained load; (b) extraordinary load; (c) total live load.

2.1.1 Sustained load

As presented in JCSS [27], the sustained load intensity acting on an infinitesimal area δA at a location (x, y) of a given floor of a given building at an arbitrary point-in-time can be represented as a stochastic field $W(x, y)$ expressed by:

$$W(x, y) = m + V + U(x, y) \tag{1}$$

where m is a "grand mean" of the load intensity over all buildings under the same occupancy type; V is a zero-mean normally distributed random variable; and $U(x, y)$ is a zero-mean random field.

The random variable V can be thought as the sum of two other zero-mean independent and normally distributed random variables B and F, where B describes the deviation of the average for the whole building from the grand mean m; and F describes the deviation of the floor average with respect to $m + B$. The random field $U(x, y)$ represents the spatial variability of the load intensity within that particular floor and shows a characteristic skewness to the right [27].

This model, while very simple, allows for the calculation of load effects caused by the real loading up to a sufficient degree of accuracy for all practical purposes. Assuming linear elastic behavior, where the superposition principle is valid, the resulting effect S can be obtained by:

$$S = \iint_A W(x, y) I(x, y)\, dx\, dy \tag{2}$$

where $W(x, y)$ is the load intensity; $I(x, y)$ is the surface influence for the desired effect; and A is the influence area. For non-linear structural response, the load effect can be approximated by an incremental analysis assuming stepwise linearity, replacing W and S in Equation 2 for steps ΔW and ΔS of load magnitude and effect, and the surface influence $I(x, y)$ for some equivalent function that takes into account the total load history.

In design codes, live load values are generally specified as uniform loads. Thus, it is of practical interest to define an equivalent uniformly distributed load (EUDL), i.e., the uniform load that produces the same effect S as the actual load field $W(x, y)$. Denoting the sustained load EUDL by Q, it follows that:

$$Q = \frac{\iint_A W(x,y) I(x,y)\, dx\, dy}{\iint_A I(x,y)\, dx\, dy} \tag{3}$$

Its mean and variance are given by:

$$\mathrm{E}[Q] = \frac{\iint_A E[W(x,y)] I(x,y)\, dx\, dy}{\iint_A I(x,y)\, dx\, dy} = m \tag{4}$$

$$\mathrm{Var}[Q] = \frac{\iint_A \iint_A I(x_1,y_1) I(x_2,y_2)\, \mathrm{Cov}[W(x_1,y_1) W(x_2,y_2)]\, dx_1\, dy_1\, dx_2\, dy_2}{\left[\iint_A I(x,y)\, dx\, dy\right]^2} \tag{5}$$

In general, if the load intensity at a particular location (x_1, y_1) is greater than the floor average, it is likely that the load intensity at a nearby point (x_2, y_2) is also high. In other words, there is a generally positive correlation to the field $U(x, y)$ that tends to decay as the distance separating the points increases.

Hauser [33] proposed three different empirical expressions for the correlation of the random field U. The following one is widely used, due to its convenience for integration:

$$\mathrm{Cov}[U(x_1, y_1), U(x_2, y_2)] = \sigma_U^2 \exp\left(-\frac{r^2}{d^2}\right) \tag{6}$$

In the above expression, σ_U^2 is the variance of U; $r = \sqrt{(x_1 - x_2)^2 + (y_1 - y_2)^2}$ is the horizontal distance separating the points (x_1, y_1) and (x_2, y_2); and d is a constant to be determined that dictates how fast the correlation decays over distance (usually between 1 to 2 m).

Mitchell and Woodgate [16] studied what they called the “stacking effect”, i.e., the tendency for tenants to load different floors in a similar way vertically. This effect can be accounted for by introducing a vertical correlation parameter ρ_c in Equation 6, as shown in [29]. This parameter was estimated by Peir and Cornell [29] to be approximately equal $\rho_c = 0.7$ for office buildings.

In this study, the random field $U(x, y)$ was regarded as a “white-noise” process, which means that load intensities in two points are uncorrelated if there is any separation between them. This frequently employed assumption is quite reasonable, as long as the area A is not too small [31]. Choi [34] investigated both hypotheses and concluded that, in general, the assumption that load intensity is spatially correlated is marginally better than the white-noise approximation. However, this uncorrelated assumption is sufficiently accurate for practical applications, and has the advantage of considerably simplifying the quadruple integral in Equation 5, allowing for a conservative upper bound to be established for $\mathrm{Var}[Q]$:

$$\mathrm{Var}[Q] \leq \sigma_V^2 + \sigma_U^2 \frac{A_0}{A} \kappa \tag{7}$$

where σ_V^2 is the variance of the random variable V; σ_U^2 is the variance of the random field U; A_0 is the smallest area for which a distributed load is of interest; and κ is a shape factor (sometimes also referred to as a peak factor) depending on the influence surface, given by:

$$\kappa = A \frac{\iint_A [I(x,y)]^2\, dx\, dy}{\left[\iint_A I(x,y)\, dx\, dy\right]^2} \tag{8}$$

Equation 7 is valid only for $A \geq A_0$. For $A < A_0$, one should take $A_0/A = 1$.

Usually, it is more convenient to normalize the double integrals in Equation 8. For example, for a rectangular area with sides a and b, one can define normalized coordinates (ξ, η) ranging from 0 to 1 so that $x = \xi a$ and $y = \eta b$. Then, the expression for κ becomes:

$$\kappa = \frac{\int_0^1 \int_0^1 [I(\xi,\eta)]^2\, d\xi\, d\eta}{\left[\int_0^1 \int_0^1 I(\xi,\eta)\, d\xi\, d\eta\right]^2} \tag{9}$$

Naturally, κ depends on the shape of the influence surface, which in turn depends on the considered load effect (Figure 1 of the Data Availability Material), usually assuming values between 2 and 3 [54]. JCSS [27] presents some examples of influence surfaces with $\kappa = 2.0$ and $\kappa = 2.4$, but does not clarify to which effect each of these corresponds. McGuire and Cornell [30] report the following values for κ: 2.76 for midspan moment in beams, 2.04 for end moment in beams, and 2.20 for column loads. Tran et al. [35] present κ values for flat slabs, ranging from 1.2 to 1.9.

The calculated EUDL is generally observed to be relatively insensitive to the action being considered, provided that the influence area and the shapes of their influence surfaces are reasonably similar. The exception to this rule would be midspan shear in beams, which becomes comparable to other effects when considering only half the influence area, since the influence surface for this effect has regions with negative values [30]. In this study, $\kappa = 2.0$ was adopted as a general value representing no particular effect for the sake of simplicity.

At this point, it is important to emphasize the distinction between influence area and tributary area, made very clear in the ASCE/SEI 7-16 [22]. The area A in Equations 7 and 8 is the influence area, i.e, the area over which the influence surface for a structural effect is significantly different from zero. This definition does not correspond to the usual notion of tributary area – often mistakenly called influence area – which is thought of as the area that contributes to the loading on a particular element, delimited by the panel centerlines in a slab. The influence area is usually equal to twice the tributary area for beams and four times the tributary area for columns (Figure 2).

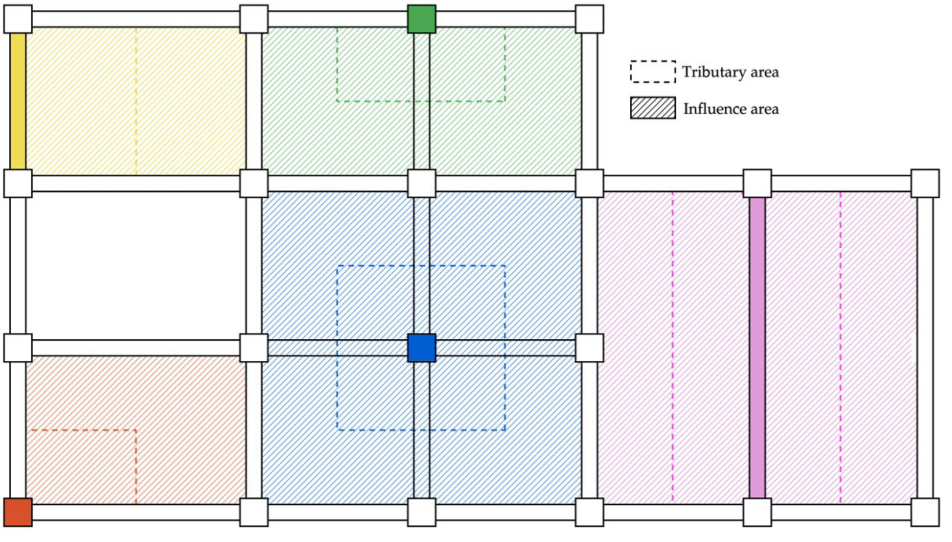

Figure 2. Tributary and influence areas for typical structural members.

Data from load surveys show a distinct skewness where most of the observed values sit left of the mean and exhibit very good agreement with a gamma distribution [29], [36]. Since the arbitrary point-in-time load intensity and the EUDL differ from each other only by the weighting function $I(x, y)$, it is reasonable to extend this hypothesis to Q and assume that it will also be gamma distributed, with mean and variance given by Equations 4 and 7, respectively.

In addition to spatial variability, live load is also variable over time. The EUDL is, therefore, a function of time. In general, the sustained load remains relatively constant for long periods of time, showing only insignificant fluctuations around a mean value that changes from time to time due to a tenancy or occupancy change.

Typically, it is assumed that the EUDL for sustained load is constant between occupancy changes[1], and that the number of occupancy changes follow a Poisson distribution with mean rate λ_q (Figure 1a). Consequently, the time between occupancy changes (or the duration of a tenancy) follows an exponential distribution, and the mean number of occupancy changes in a reference period T is equal to $\lambda_q T$.

Under these assumptions, the extreme value distribution for the maximum sustained load $Q_{\max}$ over a reference period T can be obtained from the arbitrary point-in-time distribution using the following expression [29]:

[1]This might not be the case for storage areas, where it may be necessary to take into account a gradual increase of the sustained load over time between occupancy changes.

$$F_{Q_{\max}}(x) = F_Q(x) \exp\left[-\lambda_q T F_Q(x)\left(1 - F_Q(x)\right)\right] \tag{10}$$

For values of x in the upper tail region, which are the loads of practical interest, $F_Q(x)$ tends to 1, and Equation 10 can be simplified to:

$$F_{Q_{\max}}(x) \approx \exp\left[-\lambda_q T\left(1 - F_Q(x)\right)\right] \tag{11}$$

2.1.2 Extraordinary load

The extraordinary load is associated with unusual gatherings of people, furniture or equipment in an area for a short period of time. Due to its extraordinary and transient nature, it is quite difficult to accurately measure data related to this type of loading during load surveys. Most of the available data on extraordinary load has been gathered through questionnaires submitted to the surveyed building occupants and is, therefore, liable to a considerable amount of uncertainty and subjectivity.

A model for extraordinary loads was initially proposed by Peir [28], which divides the area of interest into a number of randomly distributed load cells and represents the extraordinary event as a cluster of concentrated loads (such as the weight of people) acting on these cells, both the number and intensities of these loads being random variables. A similar model was used by McGuire and Cornell [30] and Ellingwood and Culver [31]. Harris et al. [37] proposed a more general extension of this model, where three different extraordinary load processes are considered, each modeled by a group of loads with their own parameters. The combination of these loads is accomplished using an expression proposed by Wen [38].

The JCSS Probabilistic Model Code [27] states that, for design purposes, the same approach as for the sustained load can be used. Thus, the EUDL for extraordinary load (denoted P in this paper) has mean and variance given by:

$$\mathrm{E}[P] = m_p \tag{12}$$

$$\mathrm{Var}[P] = \sigma_{U,p}^2 \frac{A_0}{A} \kappa \tag{13}$$

where the subscript p is used to differentiate extraordinary load parameters from sustained load ones (which are denoted by subscript q).

Similarly to the sustained load, it is assumed that the arbitrary point-in-time extraordinary load is adequately described by a gamma distribution, although there is not enough data to substantiate this assumption.

While it gives different values for m_p and $\sigma_{U,p}$, JCSS [27] also states that the standard deviation usually results in the same magnitude as the mean value, and that the extraordinary load is, therefore, assumed to be exponentially distributed. This assumption, however, is in clear contradiction with the previous statement: adopting the same formulation employed for sustained load would necessarily imply in a constant mean value for a given occupancy type and a standard deviation that decays with the increase in area, whereas an exponential distribution should always have equal mean and standard deviation regardless of the area A.

This inconsistency seems to have been clarified in a current draft for the "Technical Report for Reliability Background of Eurocodes"[2], that prescribes a single parameter m_p and states that $P \sim \mathrm{Exponential}(\mu_p = \sigma_p = m_p)$. However, the authors personally believe that the choice for an exponential distribution to represent extraordinary load is inadequate. This is due to the fact that, since its variance is area-independent, it tends to quickly dominate the behavior of the total load $L = Q + P$ as the area increases and $\mathrm{Var}[Q]$ gets smaller, leading to excessively conservative results for large areas when compared to the live load reduction allowed for in major design codes around the world.

Based on data from 1989 extraordinary events recorded in a load survey carried out in Sydney [39] in the seventies, Choi [40] found out that in reality both the mean and the standard deviation of the extraordinary load are area dependent. However, the variation for the mean value is much smaller, and it seems reasonable that it could be disregarded.

In this study, the intermittent part of the live load is represented by a Gamma distribution, following many other studies. Moments of the distribution are obtained from Equations 12 and 13.

[2] Unpublished, still being worked on at the time this paper was written.

As for temporal variability, the extraordinary load is represented by a Poisson-arriving spike process with mean rate λ_p (Figure 1b). Accordingly, time between pulse arrivals follows an exponential distribution. The duration d_p of each pulse is considered deterministic.

The extreme value distribution $P_{\max}$ of a Poisson-distributed number of extraordinary events happening over a reference period T is obtained from the arbitrary point-in-time distribution P using the same approach as for the sustained load (Equation 11).

2.1.3 Model parameters

Ideally, model parameters should be estimated from statistical analysis and fitting to the results of load survey data. Since there are no specific survey data on Brazilian live loads, the values for the model parameters used in this study were taken from JCSS [27] and Honfi [41], as shown in Table 1.

Table 1. Live load parameters for some major occupancies.

Occupancy type	A_0 (m^2)	Sustained load				Extraordinary load				Reference
		m_q *(kPa)*	$\sigma_{V,q}$ *(kPa)*	$\sigma_{U,q}$ *(kPa)*	$1/\lambda_q$ *(years)*	m_p *(kPa)*	$\sigma_{U,p}$ *(kPa)*	$1/\lambda_p$ *(years)*	d_p *(days)*	
Office	20	0.50	0.30	0.60	5	0.20	0.40	0.3	1–3	JCSS [27]
Residence	20	0.30	0.15	0.30	7	0.20	0.30	1.0	1–3	JCSS [27]
Hotel room	20	0.30	0.05	0.10	10	0.20	0.40	0.1	1–3	JCSS [27]
Patient room	20	0.40	0.30	0.60	5–10	0.20	0.40	1.0	1–3	JCSS [27]
Classroom	100	0.60	0.15	0.40	10	0.20	0.40	0.3	1–5	Honfi [41]
Retail	100	0.90	0.60	0.60	1–5	0.40	0.60	1.0	1–14	Costa [42]

The JCSS [27] also proposes parameters for classroom and retail areas. However, investigations by Costa [42] and Honfi [41] show that these parameters are too conservative, when compared to actual values employed in major international codes. For that reason, the values suggested by Honfi [41] and Costa [42] for these occupancies are adopted in this study.

2.1.4 Total live load

In order to obtain the total live load, one must consider the combined effects of the stochastic processes for the sustained and extraordinary loads over time, i.e., $L(t) = Q(t) + P(t)$ (Figure 1c). The statistical combination of extreme loads is not a trivial task.

An approximate theoretical model for the maximum total load $L_{\max}$ over a given reference period T is presented in Chalk and Corotis [3]. This simplified model, however, is limited as it assumes several simplifications that, while reasonable for values in the upper tail region of the distribution, makes this formulation more suitable for obtaining estimates for nominal values of loads (corresponding to the upper fractiles) rather than describing the complete distribution of $L_{\max}$.

In the present study, the total live load statistics for the extreme value and arbitrary point-in-time distributions are derived through Monte Carlo simulations. Daily realizations of both sustained and extraordinary loads are generated according to the known distributions of the model previously described, over reference periods T equal to 1, 50 and 140 years. An example of a realization of the sustained and extraordinary loads in an office floor with $A = 500$ m^2 over 50 years is shown in Figure 3.

This process is repeated 10^4 times for each considered influence area (ranging from $A = 10$ to 500 m^2), reference period and occupancy type. The obtained data is then plotted and fitted to candidate distributions. The quality of the distribution fit to the histogram is assessed through goodness-of-fit tests such as the Pearson's chi-squared, Kolmogorov-Smirnov or Anderson-Darling tests. A pseudocode detailing the simulation procedure employed herein is presented in Figure 4.

Figure 5 shows the obtained PDF and CDF histograms for 10^4 samples of the fifty-year extreme live load (L_{50}) in an office floor with influence area $A = 100$ m^2. Superimposed to the histograms, the fitted distribution is also shown in blue, which in this case is a Type I Extreme Value distribution, also known as Gumbel distribution. The quality of this fit is also graphically visualized through P-P (probability-probability) and Q-Q (quantile-quantile) probability plots, presented in Figure 6. The obtained p-values for the goodness-of-fit tests are shown in Table 2. At a significance level of $\alpha = 0.05$, the null hypothesis that L_{50} follows a Gumbel distribution is accepted considering all performed tests.

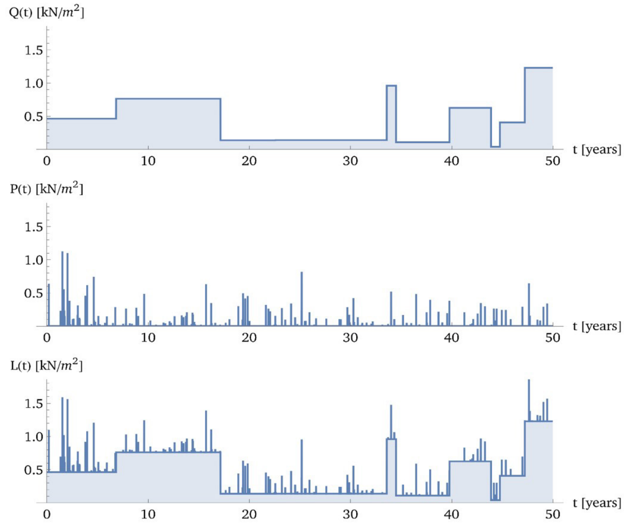

Figure 3. Time histories of one sample of sustained, extraordinary and total live load for an office floor with $A = 500\ \mathrm{m}^2$.

Algorithm: Monte Carlo simulation of live load in buildings

```
for each occupancy type do // office, residential, hotel, patient room, classroom, retail
    define A0 from Table 1 // from Table 1
    define sustained load parameters (m_q, σ_V,q, σ_U,q and λ_q) // from Table 1
    define extraordinary load parameters (m_p, σ_U,p, λ_p and d_p) // from Table 1
    for each influence area A do // A = 10 m² to 500 m² in steps of 10 m²
        calculate mean and variance of sustained load EUDL // Equations 6 and 7
        calculate mean and variance of extraordinary load EUDL // Equations 12 and 13
        calculate distribution parameters from moments // assuming Gamma distribution
        for each reference period T do // T = 1 year, 50 years or 140 years
            k = 0
            while k < nsamples do // nsamples = 10000 samples
                ∑ t_i = 0
                while ∑ t_i ≤ T do
                    generate samples of time intervals t_i between tenancy changes following an
                      exponential distribution with parameter λ_q
                    generate samples of sustained load intensity corresponding to each time interval t_i
                      following a Gamma distribution
                end while
                ∑ t_j = 0
                while ∑ t_i ≤ T do
                    generate samples of arrival times t_j of each extraordinary load following an
                      exponential distribution with parameter λ_p
                    generate samples of extraordinary load intensity corresponding to each arrival time
                      t_j following a Gamma distribution
                end while
                for each time discretization t in T do // discretization step = 1 day
                    total live load at time t = (sust. live load at time t) + (extr. live load at time t)
                end for
                evaluate maximum live load L_T in reference period T for sample k
                k++
            end while
            calculate mean and standard deviation of L_T from all samples
            calculate distribution parameters of L_T // assuming Gumbel distribution
            perform goodness-of-fit tests // Pearson χ², Kolmogorov-Smirnov, Anderson-Darling
            calculate characteristic value from fitted distribution
        end for
    end for
end for
```

Figure 4. Pseudocode of Monte Carlo simulations of live loads in buildings

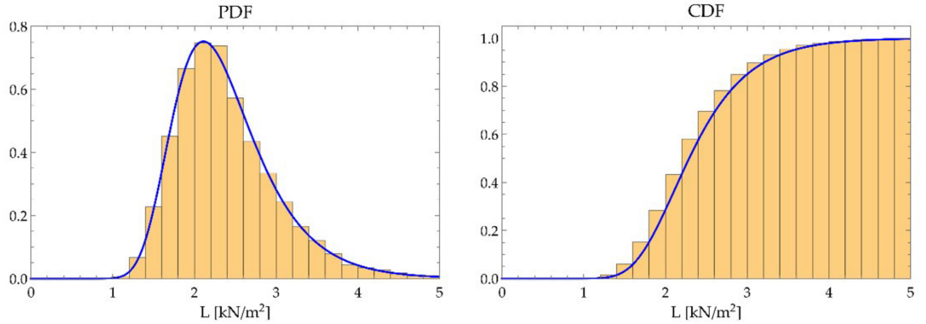

Figure 5. Probability density and cumulative distribution histograms vs. fitted Gumbel distribution for L_{50} in an office floor with $A = 100\ \mathrm{m}^2$.

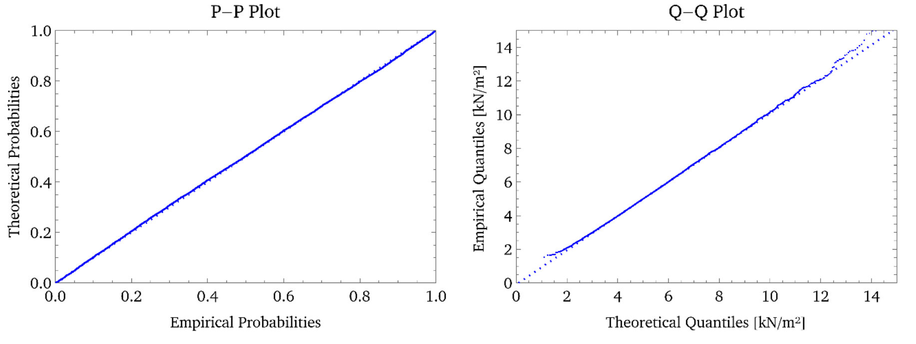

Figure 6. P-P and Q-Q probability plots showing the deviation of L_{50} from the fitted Gumbel distribution in an office floor with $A = 100\ \mathrm{m}^2$.

Table 2. Goodness-of-fit tests for L_{50} in an office floor with $A = 100\ \mathrm{m}^2$.

Test	Statistic	P-value
Anderson-Darling	0.2482592	0.9713050
Kolgomorov-Smirnov	0.0056237	0.9098539
Pearson χ^2	70.272	0.7479799

2.2 Partial safety factor for ULS verification

In this study, the partial safety factor $\gamma_F = \gamma_{f1}\gamma_{f3}$ for live loads is estimated using the Design Value Method, as presented in the Annex C of EN 1990:2002 [43], [44]. The partial safety factor γ_S for the effect of a generic variable action can be determined from its design value S_d and characteristic value S_k by:

$$\gamma_S = \frac{S_d}{S_k} \tag{14}$$

Characteristic values obtained with the stochastic model presented herein are shown in Section 3 (Results). The design value S_d, in turn, can be calculated as a function of the known probability distribution of S. For a Gumbel distributed variable, S_d is given by:

$$S_d = u - \frac{1}{a}\ln\left(-\ln\left(\Phi(-\alpha_S\beta_T)\right)\right) \tag{15}$$

where $\Phi(\cdot)$ is the cumulative distribution function of the standard normal distribution; α_S is the FORM sensitivity factor for the action effects, and β_T is the target reliability index. In Section 3.6, the reliability-based calibration of partial safety factors employed in Brazilian design codes for steel [45] and concrete [46] structures – originally performed by Santiago et al. [47] – is re-processed using the live load statistics developed herein, and the mean reliability index using current NBRs 8681, 8800 and 6118 factors is found to be equal to 3.17 for a period of 50 years. Hence, a target reliability index of $\beta_T = 3.17$ is considered herein. Also, the sensitivity factor was taken as $\alpha_S = -0.66$, which is the average value found in the re-calibration.

In Equation 15, u and a are the location and scale parameters of the Gumbel distribution, respectively, calculated from its mean μ and standard deviation σ as:

$$u = \mu - \frac{\gamma}{a} \tag{16}$$

$$a = \frac{\pi}{\sigma\sqrt{6}} \tag{17}$$

where $\gamma = 0.577216$ is the Euler-Mascheroni constant. Alternatively, Equation 15 can be reasonably approximated by [44]:

$$S_d \approx \mu - \sigma\left(0.45 + 0.78\ln\left(-\ln\left(\Phi(-\alpha_S\beta_T)\right)\right)\right) \tag{18}$$

2.3 Combination value

Similarly, the combination factor ψ_0 was also estimated using the same approach described in Annex C of EN 1990:2002 [43], [44]. This method is based on representing the effects of two independent generic variable actions to be combined, S_1 and S_2, by a Ferry-Borges-Castanheta model, that is, by a rectangular-wave process with fixed durations T_1 and T_2 (with $T_1 > T_2$) smaller than the reference period T. The magnitude of the effect in each basic interval is assumed constant, uncorrelated, and equal to the maximum value within this period (Figure 2 of the Data Availability Material). It is also assumed that S_1 and S_2 are stationary and ergodic, so that a particular realization over a sufficiently long interval may be used, instead of an envelope of samples.

The basic period for live loads is taken as the mean time between tenancy changes, $T = 1/\lambda_Q$, which usually ranges from 5 to 10 years for the major occupancy types. Live load effects are usually to be combined with environmental loads such as wind, whose basic period is generally taken as $T_2 = 1$ year.

Under these assumptions, the combination factor can be calculated as:

$$\psi_0 = \frac{F_S^{-1}(\Phi(0.4\beta_c)^r)}{F_S^{-1}(\Phi(\beta_c)^r)} \tag{19}$$

where $F_S^{-1}(\cdot)$ is the inverse cumulative distribution function of the extreme value of the accompanying action in the reference period T; $\Phi(\cdot)$ is the standard normal cumulative distribution function; r is the ratio T/T_1 rounded to the nearest integer; and β_c is the equivalent reliability index for the interval T_1, given by:

$$\beta_c = -\Phi^{-1}(\Phi(\alpha_S\beta_T)/r) \tag{20}$$

In the above expression, α_S and β_T are the same as described in Equation 15.

Alternatively, the combination factor can be derived according to Turkstra's Rule, leading to the following expression for a Gumbel distributed variable:

$$\psi_0 = \frac{1-0.78V\left[0.577+\ln\left(-\ln\left(\Phi(-0.4\alpha_S\beta_T)\right)\right)+\ln(r)\right]}{1-0.78V\left[0.577+\ln\left(-\ln\left(\Phi(-\alpha_S\beta_T)\right)\right)\right]} \tag{21}$$

where $V = \sigma/\mu$ is the coefficient of variation of the accompanying action for the reference period T. A more detailed derivation of these formulas can be found in ISO 2394:1998 [48].

2.4 Frequent and quasi-permanent values

Figure 3 of the Data Availability Material shows the temporal variability of a certain effect of a generic variable action S over a reference period T. For a given level S^*, the relative duration η that the process $S(t)$ spends above that level S^* given by the sum of the time periods $t_1, t_2, \ldots, t_n$ divided by T.

For an ergodic process, the relative duration η can be computed as:

$$\eta = pq = q\left(1 - F_{S_{\mathrm{apt}}}(S^*)\right) \tag{22}$$

where $F_{S_{\mathrm{apt}}}$ is the cumulative distribution function of the average point-in-time value of action S; and q is the probability of S having a non-zero value. It is important to note that the distribution S_{apt} refers only to the cases where S has a non-zero value. Thus, the level S^* corresponding to a given relative duration η can be obtained by:

$$S^*(\eta) = F_{S_{\mathrm{apt}}}^{-1}\left(1 - \frac{\eta}{q}\right) \tag{23}$$

The distinction for the case where S can assume values equal to zero may be relevant for a generic stochastic process $S(t)$. For live loads, however, the sustained load $Q(t)$ – and therefore the total load $L(t)$ – is always "on", i.e., $q = 1$ in Equation 23.

Following the definitions stated in Section 1.2, the frequent and quasi-permanent factors ψ_1 and ψ_2 can be calculated as:

$$\psi_1 = \frac{L_1}{L_k} = \frac{F_{L_{\mathrm{apt}}}^{-1}(1-0.05)}{L_k} \text{ and } \psi_2 = \frac{L_2}{L_k} = \frac{F_{L_{\mathrm{apt}}}^{-1}(1-0.50)}{L_k} \tag{24}$$

where L_k is the characteristic value calculated according to the definition given in NBR 8681:2003 [25] and NBR 6120:2019 [1]

It should be noted that, in order to determine ψ_1 and ψ_2 using Equation 24, one must know the arbitrary point-in-time distribution of the total live load (L_{apt}), which is obtained through Monte Carlo simulation. These simulations, however, can be very time and memory consuming. Alternatively, an approximate theoretical model can be employed that allows one to calculate the relative duration η that $L(t)$ spends above a given load level from the arbitrary point-in-time distributions for the sustained and extraordinary load. Both follow a gamma distribution whose moments are easily determined from the model parameters in Table 1. A more detailed derivation of this analytical model is provided in Corotis and Tsay [49]. Herein, the simulation approach is adopted, since many realizations of the load processes were already carried out in order to derive the 1, 50 and 140-year extreme distributions.

3 RESULTS AND DISCUSSIONS

3.1 Characteristic values of live loads

The characteristic values of live loads for the occupancy types indicated in Table 1 were obtained through Monte Carlo simulation for increasing values of influence area, up to $A = 500\ \mathrm{m}^2$. The obtained results for office and residential buildings are shown in Figures 7 and 8, respectively.

Figure 7a shows the characteristic values as calculated by three approaches: a) as the 70th fractile (30% exceedance probability) of L_{50}; b) as the mode of L_{140}; and c) from the annual maxima L_1 as the value corresponding to the 140-year return period. Those values are compared to the nominal values from different international design codes [22], [23], [24], including the live load reduction prescribed by these codes. The nominal values from NBR 6120:2019 [1] are not indicated, since the Brazilian code does not allow area-based live load reduction; instead, it allows only story-based reduction for columns and foundations. A comparison with NBR 6120:2019 live-load reduction factor is presented in Section 3.2.

In general, the results obtained using the JCSS model seem to be slightly higher than those indicated in the considered design codes. However, a direct comparison is inappropriate, given that the definitions of characteristic value adopted by these codes differ from that of NBR 8681:2003. Furthermore, the curves for the 70th percentile of

L_{50} and the mode of L_{140} are practically coincident, but the results calculated from L_1 are somewhat higher. This occurs because the annual maxima are not fully independent, since the tenancy duration for the sustained load is usually longer than 1 year.

Figure 7b represents, in red, the nominal value ($L_n = 2.5\ \mathrm{kN/m^2}$) given in NBR 6120:2019 for office buildings, and the simulation results for exceedance probabilities of 25% and 35% in 50 years. The blue region between the curves correspond to the values that are in agreement with the definition of characteristic value from NBR 8681:2003. For influence areas around 100 to 120 m^2, the results obtained from the stochastic model are consistent within the 25% to 35% range definition. For the design of an internal beam, an influence area between 100 and 120 m^2 corresponds to a floor plan with a regular span between 7.1 and 7.7 m. For an internal column and considering a single-story load, this interval represents spans between 5.0 and 5.5 m.

Figure 7c shows the frequency with which the maximum total load L_{50} is caused by the combinations denoted by the authors as Cases I to IV, as explained in Table 3. For office buildings, the relative importance of the sustained load in the combination tends to increase while, on the other hand, the extraordinary load becomes less relevant for larger areas.

Table 3. Combinations of sustained and extraordinary loads leading to the maximum total load L_{50}.

Case	Description
I	Lifetime maximum sustained load + maximum extraordinary load during that tenancy
II	Lifetime maximum extraordinary load + corresponding instantaneous sustained load
III	Simultaneous occurrence of lifetime maxima for both sustained and extraordinary loads
IV	Combination where the neither the sustained nor the extraordinary loads are at their maximum values

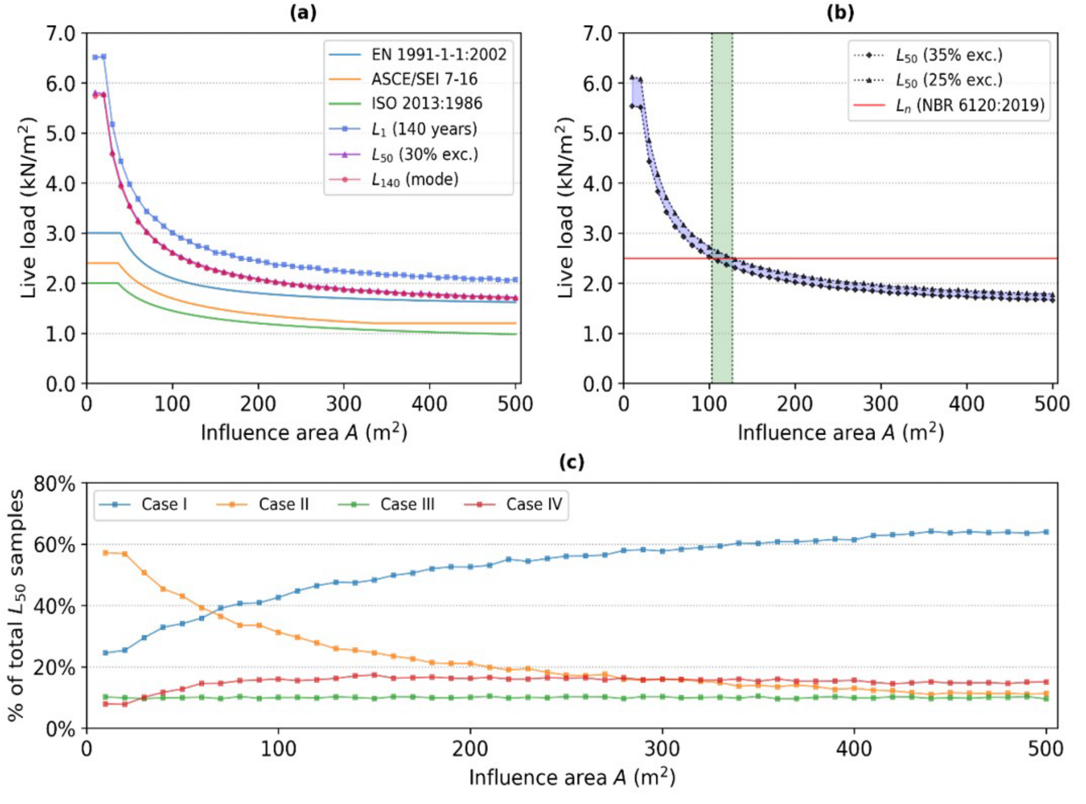

Figure 7. Simulated total live load for office buildings

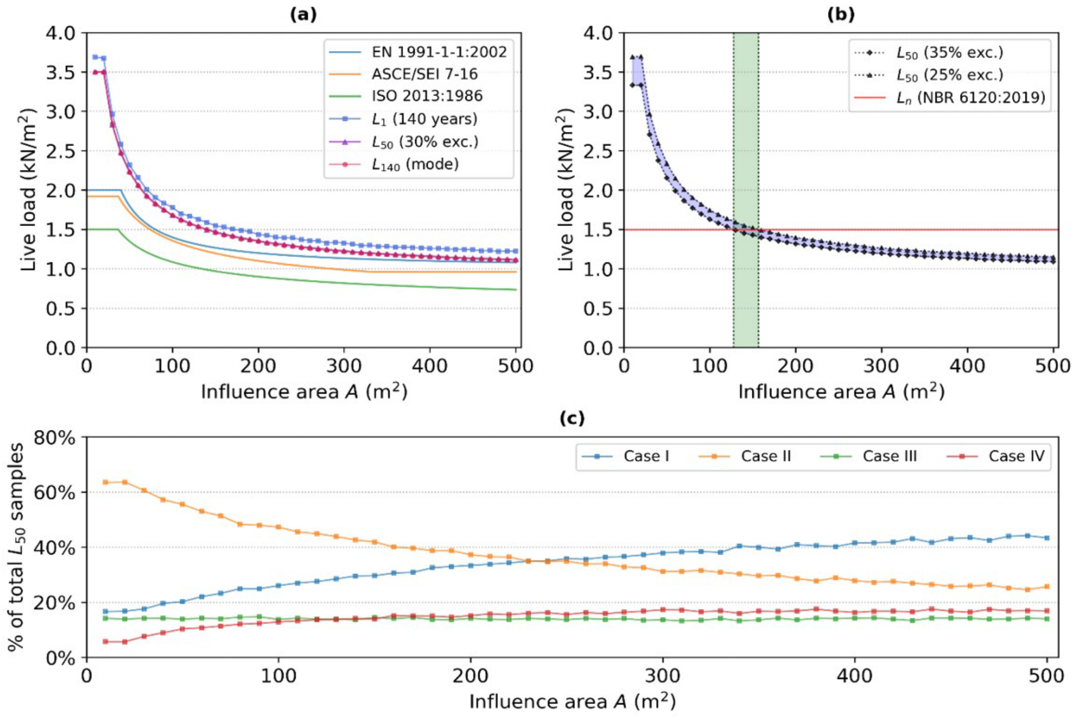

Figure 8. Simulated total live load for residential buildings.

Similar to office buildings, the simulated loads for residential buildings (Figure 8) seem to reasonably agree with the design codes – especially for higher influence areas –, resulting in marginally higher values. This is probably because office and residential buildings are by far the occupancy types with the most amount of available survey data, and therefore have more reliable model parameters. However, for influence areas smaller than 100 m^2, the stochastic model produces loads greater than the normative nominal values; this should be considered with caution when designing elements with small influence area. Figure 8 also shows the same tradeoff between sustained and extraordinary load as the influence area increases.

Similar results for the other occupancy types in Table 1 are shown and discussed in [42] and in the dataset related to this paper (Data Availability Material). The simulation results for classrooms and retail areas using JCSS [27] parameters, not shown here, led to results much higher than the representative values given in design codes [42]. A comparison with parameters used in similar studies [3], [37] shows that the values recommended by the JCSS are unreasonably high and should be revised, especially for the extraordinary load.

The same shortcoming of the JCSS [27] model was also observed by Honfi [41]. In an attempt to bring the results more in line with those of other occupancy types, the author proposed a set of modified parameters for these occupancy types (presented in Table 1), which are used in this study. The suggested parameters are more consistent with the JCSS [27] statement that the standard deviation and mean value of the extraordinary load are usually of the same magnitude. It should be mentioned that, for patient rooms and retail areas, the largest mean duration of the sustained load was adopted ($1/\lambda_Q = 10$ years and $1/\lambda_Q = 5$ years, respectively).

3.2 Live load reduction factor

As shown in the previous results, the characteristic value for live loads is primarily dependent on the influence area A. For larger areas, the equivalent uniformly distributed load tends to decrease, as it becomes more and more unlikely that the load magnitude would be very high over the entire loaded area. To account for this behavior, design codes usually allow some form of live load reduction to be applied.

The ASCE code [22] presents an expression for live load reduction based on the square root of the influence area, allowing for a reduction of up to 50%. Similar expressions can be found in EN 1991-1-1:2002 [23] and ISO 2103:1986 [24]. While the latter unambiguously states that the area to be considered is the tributary area, Eurocode 1 refers only to a "loaded area", not making clear whether or not the area intended to be used in the formula corresponds to the customary definition of tributary area.

The Brazilian NBR 6120:2019 allows the design loads to be reduced only for columns and foundations. The reduction factor is specified as a function of the number of floors for which live load reduction is permitted. In addition to area-based reduction, Eurocode 1 [23] also allows story-based reduction for columns.

A column typically will have an influence area spanning over multiple floors, each floor owned by a different tenant. However, tenancy changes are not likely to occur over all floors simultaneously. Hence, there is some correlation between two successive values of the sustained load when designing a column, since a tenancy change in one floor only affects part of the area contributing to that effect. McGuire and Cornell [30] studied the influence of tenant arrangement and independence and floor-to-floor-correlation and concluded that the organization of tenants in a building does not significatively affect the upper fractiles of the maximum total load. Therefore, it is reasonable to conservatively assume that one tenant occupies the entirety of the influence area for a column. In this study, the same white-noise model employed in the previous section is also employed for multi-story column design.

In order to compare the stochastic model results with provisions of the Brazilian code, it is necessary to first assume a regular column spacing so that the total influence area of a column can be computed from the number of floors. Two situations were considered: an interior column and an edge column of a multi-story building with regular column spacing of 5 m, which is a usual span for concrete beams. The influence area contributing to the column load is, therefore, $A = 4A_{\text{trib}} = 4 \cdot 5 \cdot 5 = 100\ \text{m}^2$ per supported floor for the interior column, and half that area for the edge column. The adopted peak factor was $\kappa = 2.2$, as indicated by McGuire and Cornell [30] for column loads.

Simulations were performed only for office and residential buildings, since those are the most common reducible occupancy types and the model results have been shown to agree well with the nominal loads specified in NBR 6120:2019. The results are shown in Figure 9. Since each floor would have its own reduction factor, results from the simulations were compared to the average reduction factor over all floors. The story-based live load reduction formula given in Eurocode 1 [23] is also presented, for comparison purposes.

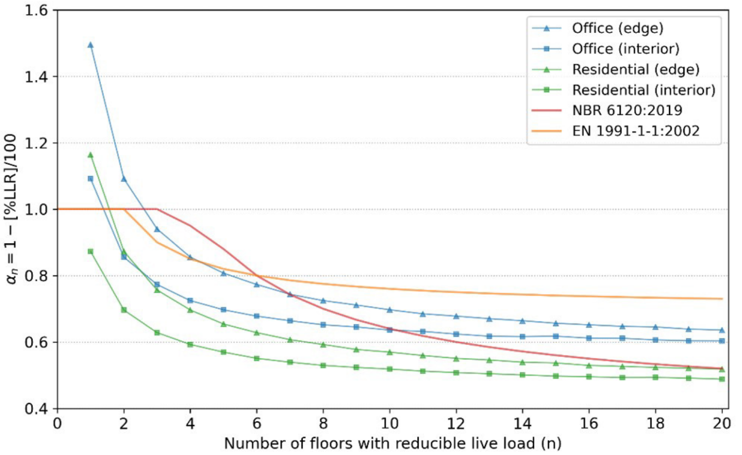

Figure 9. Comparison of the stochastic model results with the live load reduction factor allowed in the Brazilian design code for office and residential buildings.

As can be seen in Figure 9, the live load reduction allowed in NBR 6120:2019 is conservative until around 6 to 10 floors, but then becomes non-conservative, since α_n tends to 0.4 (i.e., an allowed reduction of 60%) when the number of floors increases, but the simulation results caps around 60% of the nominal load for offices and 50% for residential buildings when the area goes to infinity. The story-based reduction formula, on the other hand, seems to be overly conservative, allowing for a maximum reduction of 30%.

Because it is more consistent with the stochastic model, the influence-area based approach employed in ASCE/SEI 7-16 [22] seems to be better suited for determining live load reduction. Similar expressions are proposed for office and residential buildings by fitting simulation data to a power law of the form $\alpha_A = a + bA^{-0.5}$, where A is the influence area:

$$\text{Office} \Longrightarrow \alpha_A = 0.4 + \frac{6.25}{\sqrt{A}} \leq 1.0 \tag{25}$$

$$\text{Residential} \Longrightarrow \alpha_A = 0.3 + \frac{5.45}{\sqrt{A}} \leq 1.0 \tag{26}$$

Figure 10 compares these two formulas to the simulation values obtained herein for office and residential occupancies: a very good match can be observed. Naturally, for practical applications, it is desirable to have a single formula that is valid for all occupancy types for which reduction is allowed. The objective of this example is only to show that the model presented herein can be used to derive a reasonably simple formula that is both easy to use and provides a good and consistent fit with the mathematical formulation.

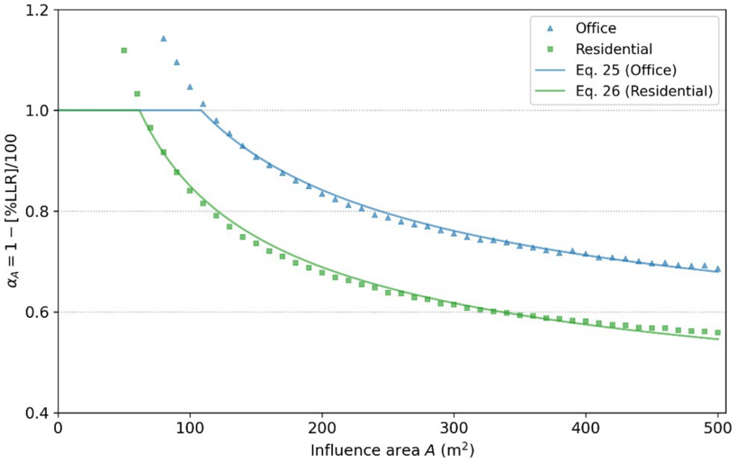

Figure 10. Example of proposed expressions for live load reduction based on influence area.

3.3 Partial safety factor for ULS verification

Figure 11 shows the variation of the partial safety factor γ_L for live loads, estimated using Equation 18, valid for a Gumbel distributed variable. Results for classrooms and retail premises are based upon the modified set of model parameters proposed by Honfi [41] and Costa [42].

It is clear that γ_L varies over a wide range of values for the different occupancy types considered, but tends to decay as the area increases, as a result of the decrease in the coefficient of variation of L_{50}. Table 4 shows the values of γ_L calculated for specific reference areas chosen so that the characteristic value from the simulations is equal to the representative value in NBR 6120:2019. For these areas, the coefficient γ_L seems to lie between 1.50 and 1.60 for most occupancy types. This result is more in line with the values $\gamma_L = 1.60$ prescribed by ASCE/SEI 7-16 [22] and $\gamma_L = 1.50$ found in EN 1991-1-1:2002 [23], This also indicates that the value $\gamma_L = 1.40$ adopted in Brazilian codes is too low and doesn't properly reflect the variability of live loads.

For comparison, Santiago et al. [47] found $\gamma_L = 1.68$ (rounded to 1.70) as result of a reliability-based calibration exercise. Yet, the authors of [47] acknowledged that the live load statistics they employed were leading to unusually low reliability indexes for many of the considered structural configurations, when compared to similar studies. This was the main motivations to develop the study presented herein. Section 3.6 presents the fresh results obtained in a re-evaluation of the reliability-based calibration, which reflect the new live load statistics in Table 4.

As for the 50-year extreme live load (L_{50}) statistics themselves, the results found in this study seem to indicate that the coefficient of variation adopted by Santiago et al. [47] was too high. The statistics indicated in Table 4 are more in line with those presented by Ellingwood et al. [32] and Szerszen and Nowak [50]. The reason why the L_{50} statistics reported by Holický and Sýkora [51] are so unlike the others is because the authors relate the characteristic value to a 5% exceedance probability in a reference period of 50 years, according to the definition found in background documents to the Eurocode 0 pre-standard ENV 1991-1:1994 [52]. In addition to that, Holický and Sýkora [51] only considered the sustained load part.

The average point-in time live load (L_{apt}) statistics obtained in this study have a coefficient of variation somewhat higher than those reported by Ellingwod et al. [32], but a smaller bias factor.

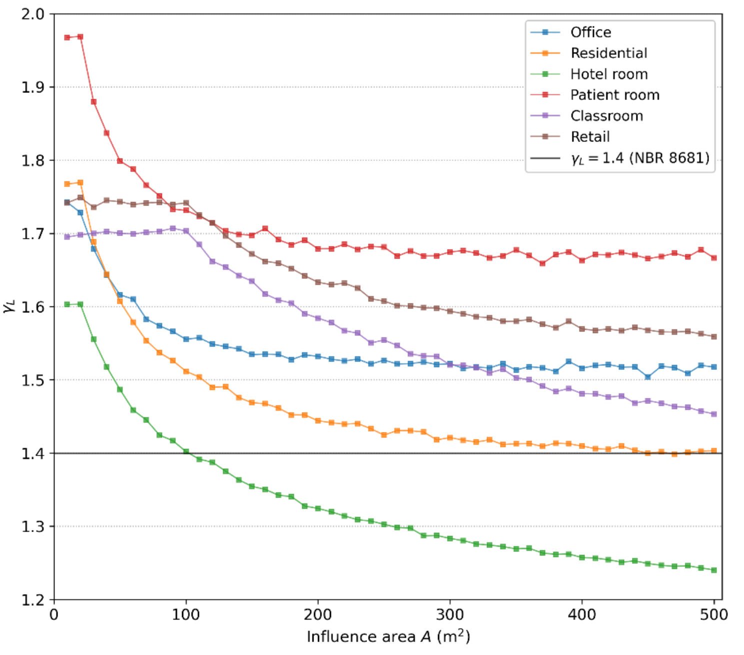

Figure 11. Partial safety factor γ_L with increasing influence area A.

Table 4. Live load statistics and corresponding estimated partial safety factor γ_L for specific reference areas A_{ref} and comparison with statistics from the literature.

Occupancy type	L_n* (kN/m²)	A_{ref} (m²)	L_{apt} (Gamma)		L_{50} (Gumbel)		L_{140} (Gumbel)		γ_L	ψ_0
			μ	c.o.v.	μ	c.o.v.	μ	c.o.v.		
Office	2.5	110	0.20 L_n	0.94	0.93 L_n	0.26	1.11 L_n	0.21	1.56	0.42
Residence	1.5	140	0.20 L_n	0.75	0.93 L_n	0.22	1.09 L_n	0.18	1.48	0.52
Hotel room	1.5	220	0.20 L_n	0.24	0.95 L_n	0.14	1.05 L_n	0.13	1.31	0.67
Patient room	2.0	110	0.20 L_n	1.16	0.89 L_n	0.35	1.13 L_n	0.28	1.72	0.42
Classroom	3.0	300	0.20 L_n	0.61	0.92 L_n	0.24	1.09 L_n	0.20	1.52	0.53
Retail	4.0	310	0.22 L_n	0.86	0.92 L_n	0.28	1.11 L_n	0.22	1.59	0.40
Average			**0.21 L_n**	**0.76**	**0.92 L_n**	**0.25**	–	–	**1.53**	**0.49**
Santiago et al. [47]			0.25 L_n	0.55	1.00 L_n	0.40	–	–		
Ellingwood et al. [32]			0.25 L_n	0.55	1.00 L_n	0.25	–	–		
Szerszen and Nowak [50]			–	–	0.93 L_n	0.18	–	–		
Holický and Sýkora [51]			–	–	0.60 L_n	0.35	–	–		

* The reference value L_n for each occupancy type is the nominal value given in NBR 6120:2019 [1].

3.4 Combination factor ψ_0, frequent and quasi-permanent values ψ_1 and ψ_2

Due to space constraints, combination values, frequent and quasi-permanent values are presented and discussed in the dataset related to this manuscript (Data Availability Material).

3.5 Updated reliability-based calibration of NBRs 8681, 6118 and 8800

The live load statistics L_{50} and L_{apt} presented in Table 4, direct result of this study, were used to reprocess the reliability-based calibration of partial load factors and load combination factors of NBRs 8681, 6118 and 8800. Due to space constraints, only the main results are presented here. For more information on the implementation of the calibration procedure, the reader is referred to Santiago et al. [47], where it is described in detail.

Using the new live load statistics, the mean reliability index obtained using current NBR 6118:2014 partial safety factors is equal to 3.17 and using NBR 8800:2008 factors is 3.28. The target reliability index $\beta_T = 3.17$ was considered herein in the calibration.

Table 5. Updated partial safety factors, following the procedure of [47] with new live load statistics.

Coefficient	Before calibration NBR 8800:2008 [45]	Before calibration NBR 6118:2014 [46]	Original calibration [47] $\beta_T = 3.0$	New calibration† $\beta_T = 3.17$
γ_C	–	1.40	1.40	1.40
γ_S	–	1.15	1.15	1.15
γ_{a1}	1.10	–	1.10	1.10
γ_{a2}	1.35	–	1.30	1.40
γ_D	1.25	1.40	1.25	1.20♑
γ_L	1.50	1.40	1.70	1.50
γ_W	1.40	1.40	1.65	1.50
ψ_L	0.50 / 0.70 / 0.80	0.50 / 0.70 / 0.80	0.35	0.45
ψ_W	0.60	0.60	0.30	0.35
$\gamma_L \cdot \psi_L$*	0.75 / 1.05 / 1.20	0.70 / 0.98 / 1.12	0.60	0.68
$\gamma_W \cdot \psi_W$*	0.84	0.84	0.50	0.53

* Effective combination value for secondary action. † Rounded values, to make NBR 6118 compatible with NBR 8800. ♑ This coefficient is suggested to remain 1.4 when live and wind loads are zero.

Results for the re-calibration are presented in Table 5. The main effect observed in [47] is also observed here: more uniform reliability indexes, for the different structures designed using the codes, are obtained by increasing the main variable load and reducing the secondary load in the combinations. The recommended values for live load ($\gamma_L = 1.5$) and for wind load ($\gamma_W = 1.5$) are very close to the values recommended in Eurocodes. The values $\gamma_L = 1.5$ and $\psi_L = 0.45$ obtained in the reliability-based calibration are very close to the $\gamma_L = 1.52$ and $\psi_0 = \psi_L = 0.51$ obtained herein as the mean for different occupancy types (see Table 4). The partial safety factors in Table 5 are recommended to be adopted in future revisions of NBRs 8681, 6118 and 8800.

4 CONCLUDING REMARKS

In this paper, the temporal and spatial variability of the live load in buildings is addressed, using a stochastic model that is well documented in the literature. Due to the lack of survey data for Brazilian buildings, the model parameters suggested by JCSS [27] and Honfi [41] were adopted. Monte Carlo simulations were performed for office buildings, residential buildings, hotel rooms, patient rooms, classrooms, and retail areas. From the results, the following conclusions can be drawn:

a) The parameters μ_Q and $\sigma_{U,Q}$ for sustained load suggested by JCSS seem to be largely based on the summary of survey data presented by Chalk and Corotis [3]. The $\sigma_{V,Q}$ parameter seems to be slightly larger than the findings of those authors, but not unreasonably so.

b) There is some contradiction in the JCSS Probabilistic Model Code over whether the extraordinary load should be modeled as a gamma or an exponential distribution. It is the authors' personal belief that the gamma distribution is more adequate, which is backed by most of the studies employing similar models found in the literature.

c) The parameters for the extraordinary load are mostly empirical, since there are very few survey data regarding this kind of load. For classrooms and retail premises, the JCSS suggested parameters are unreasonably high when compared to similar studies.

d) Brazilian code NBR 6120:2019 presents two definitions for the characteristic value of live loads: exceedance probabilities between 25 to 35%, and mean return periods between 174 and 117 years. The second definition would only be true if the annual maxima for live loads were independent, which is not the case, given that the mean time between occupancy changes is greater than one year for most uses. Hence, NBR 6120:2019 should follow NBR 8681:2003 and limit itself to the first definition.

e) Employing the model described herein, live load statistics that are consistent with the definitions given by Brazilian design codes were derived to be used in reliability analyses. The obtained fifty-year extreme live load (L_{50}) has a bias factor of 0.92 and coefficient of variation of 25%. For the arbitrary point-in-time distribution (L_{apt}), those values are equal to 0.21 and 76%, respectively. The obtained statistic for L_{50} has a smaller coefficient of variation than the one employed by Santiago [53] and is more in line with most of the statistics reported by other authors in the literature. The arbitrary point-in-time distribution (L_{apt}) obtained herein is also significantly different than that of [53].

f) The reference areas shown in Table 4 for which the nominal load values given in NBR 6120:2019 are reproduced by the stochastic model depend on occupancy type. These areas are somewhat greater than those considered in similar studies, such as Chalk and Corotis [3] – which employs a different model for the extraordinary load with both the mean and standard deviation decaying with the increase in area – further corroborating that the JCSS parameters might be overly conservative. Investigations by Costa [42] show that, while the reference areas that lead to Brazilian nominal loads obtained using other models for intermittent loads are smaller, their corresponding bias factors and coefficients of variation do not change appreciably with respect to the values reported in Table 4 for the JCSS model. Hence, it is the authors understanding that the L_{50} and L_{apt} statistics presented herein are adequate for use in reliability problems.

g) Currently, NBR 6120:2019 allows live load reductions only for columns and foundations, and the reduction factor is given as a function of the number of supported floors. However, an approach similar to ASCE/SEI 7-16 [22] is more consistent with the stochastic model, i.e., allowing live loads to be reduced for floor beams and slabs as well (although to a smaller extent), and based on the influence area.

h) The obtained results show that the partial safety factor for live loads currently employed in Brazilian codes ($\gamma_L = 1.40$ for grouped variable actions) is too low and should be revised. Using the live load statistics obtained herein, the reliability-based calibration of partial safety factors of NBRs 8681, 6118 and 8800 was re-processed, following Santiago et al. [47]. Using the target reliability index $\beta_T = 3.17, \gamma_L = \gamma_W = 1.5$ were found, which are recommended for adoption in future revision of the above codes. Suggested combination values are $\psi_L = 0.45$ and $\psi_W = 0.35$.

i) The results presented in the complementary dataset (Data Availability Material) related to this study also showed that, in general, the combination factor ψ_0 and the frequent value reduction factor ψ_1 should probably be higher for Category A buildings (residential and other private access buildings). On the other hand, a quasi-permanent reduction factor of $\psi_2 = 0.3$ seems to be sufficient for all occupancy types considered in this study, whereas a value of $\psi_2 = 0.4$ is currently prescribed for Category B buildings (office and other public access buildings). It should be noted that these results are very sensitive to the model parameters, which – as previously stated – need further investigation, and should therefore be considered with caution.

The probabilistic model employed in this study was shown to be appropriate to represent live load variability. Most of the obtained results show good agreement with the nominal loads found in Brazilian and foreign design codes, especially for office and residential buildings, since those are the occupancies most extensively surveyed. However, the majority of load survey data that backs up the model parameters was gathered decades ago. Ideally, new surveys should be carried out using modern technologies in order to validate and further support the stochastic model parameters.

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