Load Combination Requirements for Fire-resistant Structural Design

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ABSTRACT: Fire protection of building structural systems traditionally has relied on component qualification testing, with acceptance criteria based on component survival during a "standard" fire for a prescribed rating period. These test procedures do not address the impact of the fire on a structural system. With advances in fire science and the advent of advanced structural analysis, the routine use of the computer as a design tool and limit states design, it is becoming possible to consider realistic fire scenarios and effects explicitly as part of the structural design process. In this modern engineering design approach, load requirements for considering structural actions due to fire in combination with other loads are essential, but have yet to be implemented in standards and codes in the United States. This paper provides a probabilistic basis for appropriate combinations of loads to facilitate fire-resistant structural design and recommends specific load combinations for this purpose. The probabilistic basis is essential for measuring compliance with performance objectives, for comparing alternatives, and for making the role of uncertainty in the decision process transparent.

KEY WORDS: buildings (codes), design (buildings), fire, limit states design, performance-based engineering, probability, reliability, statistics, structural engineering.

INTRODUCTION

BUILDING STRUCTURES ARE designed to withstand the effects of dead loads due to the weight of the structure and permanent attachments, live loads due to use and occupancy, and environmental loads and effects arising from snow, wind, and earthquakes. Appropriate loads and load combination requirements for structural design are stipulated in documents

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such as ASCE Standard 7-02 [1], the International Building Code [2], and NFPA 5000 [3].

These structural loads, in reality, vary randomly in space and time. The determination of appropriate values of loads and load combinations for limit states (strength, or load and resistance factor design (LRFD)) design has been facilitated by modern structural reliability principles and probability-based design procedures. The fundamental principle is that each design load combination scenario should have an acceptably small probability of being exceeded during a service period $(0,T)$, often assumed to be about 50 years. A combination of time-varying loads, $Q_i(t)$, due to man-made or environmental events for checking structural safety or serviceability is expressed as,

$$U(t) = Q_1(t) + Q_2(t) + \cdots + Q_m(t) \tag{1}$$

as illustrated in Figure 1. The maximum of the combined loads in time interval $(0,T)$ is denoted as:

$$U_{\max} = \max_{0 \le t \le T} U(t) \tag{2}$$

The design load combination, U_d, is selected so that random variable, $U_{\max}$, has a small but acceptable probability of being exceeded during the period of interest.

Since the extremes of individual load (effects) rarely occur simultaneously,

$$U_{\max} < \max\, Q_1(t) + \max\, Q_2(t) + \cdots + \max\, Q_m(t) \tag{3}$$

in which $\max Q_i(t) =$ maximum of load Q_i in $(0,T)$. Conceptual difficulties in evaluating $U_{\max}$ arise from the fact that $Q_i(t)$ are random processes, and their maxima generally are not possible to model in closed-form; the maximum of the sum is even more difficult to obtain. An important contribution of probability-based load combination analysis was to recognize (from theoretical considerations and, later, from Monte Carlo simulation) that Equation (3) could be converted to a problem involving random variables through the so-called principal action–companion action format,

$$U_{\max} = \overset{m}{\underset{i}{\max}} [\max Q_i(t) + \underset{k \neq i}{\Sigma Q_k(t)}] \tag{4}$$

In words, Equation (4) states that (provided that the individual loads are nearly statistically independent) the maximum combined effect occurs when one of the loads achieves its maximum (or "principal") value during $(0,T)$,

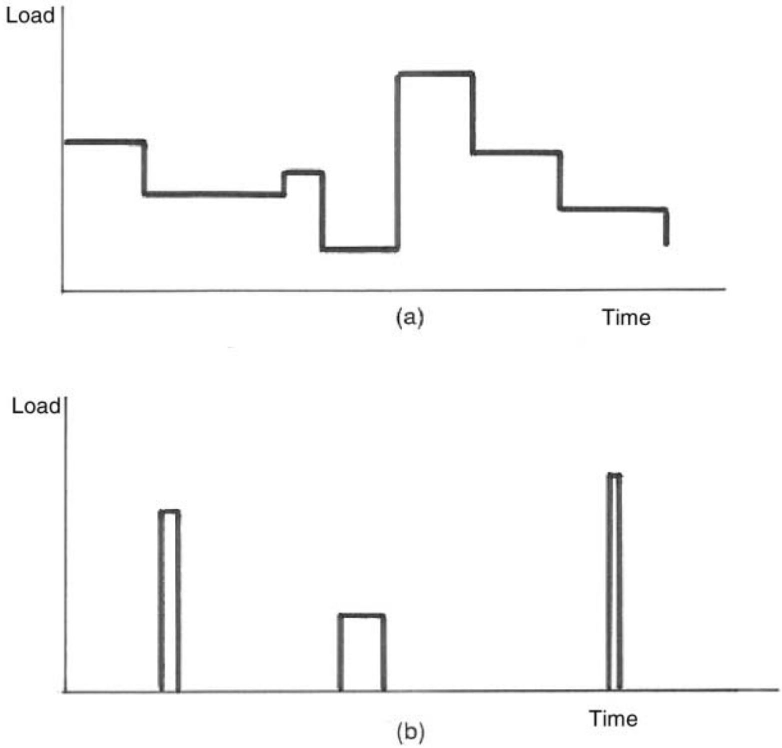

Figure 1. *Stochastic load models: (a) Sustained loads (Type I) and (b) Intermittent loads (Type II).*

with other loads at their "companion action" values [4]. This simple scheme is the basis for all probability-based load combination procedures in limit states design codes worldwide [e.g., 1,5,6].

In Chapter 2 of ASCE Standard 7-02 [1], Equation (4) becomes (in abbreviated form),

$$U_d = 1.2D + 1.6L + 0.5S \tag{5a}$$

$$U_d = 1.2D + 1.6S + (0.5L \text{ or } 0.8W) \tag{5b}$$

$$U_d = 1.2D + 1.6W + 0.5L + 0.5S \tag{5c}$$

$$U_d = 1.2D + 1.0E + 0.5L + 0.2S \tag{5d}$$

$$U_d = 0.9D + (1.6W \text{ or } 1.0E) \tag{5e}$$

in which D, L, W, S, and E are dead, occupancy live, wind, snow, and earthquake loads specified in ASCE Standard 7-02, Sections 3–9, respectively. [Equation (5e) is used to check overall stability of the structure or portions of it when the lateral forces due to wind or earthquake effects tend to destabilize (overturn) the structure while the gravity load stabilizes it.] In limit states design, the design load is the product of a nominal load, Q_n, and a load factor, γ_Q. The factored design loads $1.6L$, $1.6S$, $1.6W$, and $1.0E$ in Equations (5) are the "principal" actions in Equation (4). The probability that the actual load acting on the structure exceeds these design loads is of the order of 0.001–0.002/yr in most parts of the United States. (Probabilities of structural failure are at least two orders of magnitude less because design strengths are selected conservatively [7].) The companion action factors in Equations (5a)–(5d) are less than 1.0, which reflects the negligible probability of a coincidence of 50-year peak loads [see Equation (3)]. In all cases, there is a small but finite probability that the design load will be exceeded in any given year; the risk to the building structural system accordingly is small but finite. This fact is seldom stated explicitly in codes, standards, and commentaries.

A fire with the potential to damage a building structure severely is a low-probability event in comparison with the events giving rise to significant occupancy live, snow, wind, or earthquake loads in Equations (5). The probability of a coincidence of a fire with live, snow, wind, or earthquake loads at or near the magnitudes of L, S, W, or E in ASCE Standard 7-02 is very small, and a structure is likely to be loaded to only a fraction of the design load when the fire occurs. In the case of live load on floors, previous research has shown that when structural actions due to fire are considered, a suitable companion action floor live load is $0.5L$ [8]. On the other hand, severe fires can lead to ultimate structural limit states such as gross inelastic deformation, instability, or partial collapse, and the loads present in a building structure at the time of a fire may have a significant influence on its fire resistance.

The shift from prescriptive fire-resistant design requirements to performance-based engineering [9], in which building structural systems are designed and evaluated for realistic fire scenarios, subject to the approval of the authority having jurisdiction, mandates a thorough examination of structural load requirements for combining fire with other loads. On the international scene, load combinations for accidental events appear in Eurocode No. 1 [6] and BS 5950 [10], among others. Load combinations for dealing with rare accidental actions appear in the (nonmandatory) Commentary C2.5 of ASCE Standard 7-02, but have yet to be implemented in building codes in the United States. This paper supports the development of structural design for fire conditions in providing a reliability basis for appropriate combinations of loads, for checking the stability of the overall frame under fire conditions and, when necessary, for providing a basis for intelligent design and interpretation of fire test results.

PERFORMANCE-BASED ENGINEERING

Current professional interest in the new paradigm of performance-based engineering (PBE) stems from the desire to minimize arbitrariness and to consider a broad spectrum of design alternatives, to achieve economical solutions to building safety problems through engineering analysis rather than prescriptive measures, to take advantage of new building technologies, to match structural design criteria to performance expectations of building stakeholders, and to add value to the building process [11,12]. External to the engineering profession, its implementation in building regulations also is being driven by the World Trade Organisation Agreement on Technical Barriers to Trade [13], which stipulates that "Whenever appropriate, Members shall specify technical regulations based on product requirements in terms of performance rather than design or prescriptive characteristics" (Clause 2.8).

Fire protection traditionally has been based on component qualification testing (e.g., ASTM Standard E119 [14] or ISO Standard 834 [15]) and prescriptive design (ASCE Standard 29-99 [16]), with acceptance criteria relying on component survival during a "standard" fire for a prescribed rating period. The shortcomings of such procedures are well-known [e.g., 17]. Many structural components and systems that are known to perform acceptably under realistic fire exposures are penalized or proscribed by prescriptive design [18–20]. With advances in fire science and advanced structural analysis coupled with the routine use of the computer as a design tool, it is becoming possible to consider alternate fire scenarios and fire effects explicitly as part of the structural design process [21–25]. The Society of Fire Protection Engineers is moving its standards program for fire-resistant design toward PBE [26]. The American Institute of Steel Construction (AISC) has recently charged a task committee to develop improved fire-resistant structural design concepts for steel building structural systems; these are likely to be included in an Appendix to the 2005 AISC Specification [27]. Comparable work is ongoing on the international front [28,29].

Performance-based engineering, in contrast to traditional prescriptive approaches that essentially follow a design recipe, involves goal-setting and documentation on the part of the design team. Performance objectives for fire-resistant design might include, but are not necessarily limited to [e.g, 2,3,9]:

- Life safety and rapid egress of building occupants from the building
- Life safety for firefighters entering the building; survival of burn-out of building contents
- Protection of property or minimal disruption of business operation
- Environmental protection

Risk measures are important in PBE, as they become the basis for measuring compliance with performance objectives, for comparing alternatives, and for making the role of uncertainty in the decision process transparent. Within this context of PBE defined as above, the remainder of this paper will address issues related to risk and the derivative load requirements necessary to mitigate that risk through advanced principles of structural analysis.

ACCEPTABLE RISK BASES FOR FIRE-RESISTANT STRUCTURAL DESIGN

Fire risk mitigation and fire safety measures can be aimed at three levels: (1) preventing the outbreak of fires through elimination of ignition sources or hazardous practices; (2) preventing uncontrolled fire development through early detection and suppression; and (3) preventing loss of life or structural collapse through provision of general structural integrity, compartmentation, fire protection systems, and other measures. With these three levels in mind, the probability of structural failure due to fire can be expressed as,

$$P[F] = P[F|DI]\, P[D|I]\, P[I] \tag{6}$$

in which $P[I]$ is the probability of ignition, $P[D|I]$ is the probability of development of a structurally significant fire, given that ignition occurs, and $P[F|DI]$ is the probability of failure, given the occurrence of I and D. Measures taken by the structural engineer of record to design fire resistance into the building structural system affect $P[F|DI]$. The other two probabilities are controlled by nonstructural measures that are outside the scope of this paper.

Analysis of reliability of structures subjected to gravity loads suggests that acceptable limit state probabilities of individual structural elements and connections is of the order of 10^{-5} to 10^{-4}/yr, and that the probabilities of failure of redundant structural systems is approximately one order of magnitude lower [7]. The de minimis risk, the level below which the risk is not of regulatory concern, is of the order of 10^{-7} to 10^{-6}/yr [30,31]. Accordingly, PBE for fire resistance should strive to limit $P[F]$ in Equation (6) to the order 10^{-6}/yr or less. This criterion will be used to identify those load combinations that must be included in fire-resistant structural design as well as to screen out those that can be neglected, as described in the sequel.

PROBABILISTIC MODELS OF COMMON STRUCTURAL LOADS

Man-made and environmental events and the structural loads arising from them are random in occurrence, duration, and intensity. Such events giving rise to significant structural actions can be modeled as Poisson pulse processes [32,33]. Two sample functions of Poisson pulse processes are illustrated in Figure 1. In Figure 1(a), the load is always present (Type I), although it varies randomly in time. In Figure 1(b), the load is intermittent (Type II). Static loads (in which temporal fluctuations within each event can be ignored), such as occupancy live loads and snow loads, can be modeled directly by Type I processes. Fluctuating or dynamic loads due to wind or earthquake can be modeled conservatively for load event combination analysis purposes by Type II processes by assuming that the intensity of the pulse corresponds to the maximum fluctuating structural action occurring during an individual event.

The occurrence of load events (Type II) or the changes in load intensity (Type I) are described by the Poisson probability law:

$$P[N(t) = r] = (\nu t)^r \exp(-\nu t)/r!; \quad r = 0, 1, 2, 3 \ldots \tag{7}$$

in which $N(t)$ is the number of events to occur in time interval $(0, t)$ and ν is the mean rate of their occurrence per year. For a Type I load, the mean duration of the load event is simply $1/\nu$. For an intermittent Type II event, the mean duration is related to the probability, p, that the event is nonzero at any point in time through the relation $p = \nu\tau$, in which τ is the mean duration of the pulse. Assuming that the successive pulse intensities are identically distributed and statistically independent random variables, a knowledge of the mean rate of occurrence, ν, the mean duration, τ, and the cumulative distribution function (CDF), $F(x)$, of the pulse intensity are sufficient to characterize the stochastic load process completely.

With the occurrence and intensity statistics defined as above, one may determine the intensity of the maximum load to occur in time period $(0, T)$. For example, the CDF of the maximum intermittent (Type II) load is,

$$F_{\max}(x) = \exp[-\nu T(1 - F(x))] \tag{8}$$

Conversely, if $F_{\max}(x)$ is known or can be estimated, then the CDF of the individual load pulses (in the upper range of x, which is significant for load analysis) can be recovered from,

$$F(x) = 1 + \nu^{-1} \ln F_{\max}(x) \tag{9}$$

Equations (8) and (9) will be used in the analysis of individual loads in the following sections.

MODELING COMBINATIONS OF EVENTS AND LOADS

The basis for the load combination analysis is the probability distribution of the maximum of a sum of time-dependent structural loads modeled as stochastic processes, as summarized in Equations (1) and (2). Conservative analytical approximations are available for the CDF of $U_{\max}$ [32,33]; these approximate solutions have been validated by Monte Carlo simulation [34]. The closed-form approximations offer insights that are not immediately apparent from the Monte Carlo simulation, and are the basis for the load combination screening in later sections.

An upper bound for the probability that $U_{\max}$ exceeds x during the period of time $(0,T)$ is given by [33],

$$P[U_{\max} > x] < [\sum_i \nu_i \, G_{Qi}(x) + \sum_{ij}\sum \nu_{ij} G_{Qi+Qj} + \ldots]T \tag{10}$$

in which $G_{Qi}(x)$ and ν_i is the conditional probability of exceeding x and mean occurrence rate of Q_i alone; G_{Qi+Qj} and ν_{ij} is the conditional probability and mean occurrence rate of combination $Q_i + Q_j$, etc. For events that are modeled as intermittent (Type II) Poisson pulse processes, the mean rate of a coincidence of two events is,

$$\nu_{ij} = \nu_i \nu_j(\tau_i + \tau_j) \tag{11}$$

Note that events (or combinations of events) with very small mean rates of occurrence, ν_i, or mean rates of coincidence, ν_{ij}, contribute little to $P[U_{\max} > x]$ in Equation (10). It follows that such events contribute little to risk and need not be included in codified design. Accordingly, the focus in the probabilistic load modeling in the following sections is on the mean rate of occurrence of events and their combinations; the CDF of load intensity is considered only where that load, in combination with structural action due to fire, might be a significant design consideration.

STATISTICAL MODELS OF INDIVIDUAL LOADS

The load statistics required to perform the load combination analysis is summarized below. It should be noted that environmental load statistics are site-dependent; the values presented are typical. Additional published data are available in [4,8,34–37]. The load data are organized to correspond to the presentation in ASCE Standard 7-02.

Dead Load (D)

Dead loads are permanent gravity loads that are fixed in position and usually invariant in time if remodeling is neglected. Dead load includes the weight of the structure and permanent equipment and attachments. The intensity of dead load can be modeled by a normal distribution with a mean value of 1.05 times the nominal, D, and a coefficient of variation (COV) equal to 0.10. A major source of this variability arises from the treatment of point loads as area-averaged or distributed loads for design purposes.

Occupancy Live Load (L)

Live loads on floors are produced by the use and occupancy of the building – personnel, their possessions, and moveable items such as partitions and equipment. The total live load is modeled as the sum of two distinct components: a sustained component, L_s, and a short-duration transient component, L_e. The sustained component is the live load ordinarily on the floor at any point in time during the service life of the building, and is modeled by a Type I process (Figure 1(a)). Live load surveys in buildings develop statistical data on L_s, which can be processed using probabilistic load modeling techniques [35,36]. The intensity of L_s can be described by a Gamma CDF, with a mean value ranging from $0.24L$ to $0.5L$ (depending on the building occupancy and tributary loaded area) and a COV $\approx$ 0.60. The transient component, L_e, arises from infrequent events (Type II in Figure 1(b)), which are not measured by a survey and are determined from postulated event scenarios (remodeling, emergency crowding, etc.). The intensity of L_e can be modeled by a Gamma CDF, with typical occurrence statistics $\nu_{Le} = 1/\text{yr}$ and duration $\tau_{Le} < 1$ day, depending on scenario. It should be noted that the presence of both L_s and L_e is necessary for the (random) live load to approach the nominal live load, L (e.g., 50 psf (2.4 kPa) in general offices), stipulated in ASCE Standard 7-02 and other building codes. The live load that must be combined with fire (or other accidental load) for design or component qualification testing must take this fact into account [8].

Miscellaneous Roof Live Load (L_r)

Roof live load is mainly due to maintenance and repair operations. Such operations are intermittent in nature (Type II), occur at 5–10-yr intervals during the service life of a building ($\nu_{Lr} = 0.1$–0.2/yr), and have a duration that typically is 1 month or less. Consistent with the assumption that the mean and COV of the 50-yr maximum roof live load equals L_r and 0.30, respectively, the mean intensity of roof live load [Equation (9)] is $0.5L_r$ and the COV is 0.60. The load pulse intensity can be described by a Gamma CDF.

Roof Snow Load (S)

Snow loads are seasonal and intermittent in nature (Type II in Figure 1). They are significant for design of roofs in the northern tier of states, and are dependent on building site, exposure, and roof geometry. Although snow loads would be negligible if the supporting roof structure were directly exposed to fire, it is possible that in a compartmentalized multistory building, forces from snow might be transmitted from the roof through the structure to lower-story columns. The annual extreme roof snow load can be described by a lognormal distribution, with a mean and COV that typically are $0.2S$ and 0.9, respectively [34,37]. The mean rate of (structurally significant) snows during the winter season depends on local climatology. In temperate climates, ν_S is of the order of 2/season, with mean duration of 1 week or less. In northern climates or at high elevations, snow accumulates from storm to storm, and ν_S is of the order of 2/snow season, with duration of 2 months. The CDF of the point-in-time load is related to that of the annual maximum through Equation (9).

Wind Load (W)

Wind loads are intermittent in nature (Type II in Figure 1). The annual extreme wind load can be modeled by a Type I distribution, with mean and COV that typically are 0.35W and 0.60 [37]. The mean rate of occurrence of significant winds in nontropical climates is roughly 4/yr, with typical storm duration of 4 h or less. The CDF of the wind load pulse intensity can be obtained from Equation (9).

Earthquake Effects (*E*)

Earthquakes are intermittent in nature (Type II in Figure 1). Research during the past three decades has placed earthquake ground motions on a firm probabilistic basis (available on site-dependent basis as www.geohazards.usgs.gov). The current ground motions are based on a 2% probability of being exceeded in 50 years (2500-yr return period). The CDF of individual earthquake pulses can be determined from the CDF of the 50-yr maximum using Equation (9). The CDF is Pareto/Type II, and the COV may exceed 100%. The mean rate of occurrence typically is 0.1/yr for sites in the Western United States (WUS) and 0.02/yr (or less) for sites in the Eastern United States (EUS). The duration of strong ground motion typically would be 30 s in the WUS and 10 s in the EUS.

STRUCTURALLY SIGNIFICANT FIRE

Fire occurrence can be modeled as a Poisson event, with a probability of ignition that depends on occupancy. This ignition probability appears to be related to the floor area in an approximately linear fashion at small to moderate areas, and is of the order of $10^{-6}/\text{m}^2/\text{yr}$ for residential and light commercial occupancies [28]. The (conditional) probability of a subsequent fully developed fire (post-flashover) depends on the presence of active fire detection and mitigation systems. For example, if sprinklers are present, this conditional probability is of the order of 0.01. Therefore, the mean rate of occurrence of structurally significant fires (ignition, followed by full development), ν_T, would be of the order of $10^{-8}/\text{m}^2/\text{yr}$. (This rate may not apply to large public assembly areas, such as sports arenas and convention centers, where both the ignition and flashover probabilities may be different). Each fire scenario (ignition, fuel load, ventilation, compartment surface, compartmentation) produces a temperature–time curve which can be used to determine the structural actions (imposed deformations and resulting forces) [19,20,38]. For event combination analysis purposes, the structural action resulting from a fire can be enveloped (conservatively) by a rectangular pulse with a duration equal to the estimated fire duration (or rating period, if ASTM E119/ISO 834 are used as the basis). This duration would depend on the fuel load and ventilation, but seldom would exceed 4h.

COMBINING STRUCTURAL ACTIONS DUE TO FIRE WITH OTHER LOADS

Structural design requirements affect the term $P[F|DI]$ in Equation (6). Requiring that load combinations involving loads with essentially zero probability of coincidence be considered in design does not enhance structural performance or reliability or contribute to risk mitigation.

For a given time period (say, one year), the probability of occurrence of a rare event in that year is indistinguishable in a numerical sense from the mean rate of occurrence per year of the same event. For practical purposes, then, if the coincidence of a structurally significant fire (ignition, followed by full development) and another load event has a mean rate of occurrence of less than 10^{-6}/yr, or

$$P[D|I]\, P[I] \approx \nu_T < 10^{-6}/\mathrm{yr} \tag{12}$$

then the contribution of that load combination to $P[F]$ will be less than the de minimis level identified previously and there is no need to include it in load combination criteria for design. For a compartment with floor area of $100\,\mathrm{m}^2$, ν_T is approximately 10^{-6}/yr. For a floor area of $1000\,\mathrm{m}^2$, ν_T increases by one order of magnitude. The mean duration of the fire is of the order of 4 h (4.6×10^{-4} yr) or less.

It should be noted that, consistent with the requirements of PBE, the design load combinations are to be used as part of an analysis of the performance of the building structural system as a whole during postulated fire scenarios, rather than a building component such as a floor or column. Thus, the scope of the load combination development is broader than the loads to be imposed on components during traditional fire qualification testing procedures.

Fire and Live Load

This is probably the most important load combination for designing building structural systems to withstand the effects of severe fires. Recall that $L = L_s + L_e$, and that L reaches significant intensities only when L_e is present. With the mean occurrence rate and duration of L_e being 1/yr and 1 day (2.7×10^{-3} yr), respectively, the mean coincidence rate of $L_e + T$ from Equation (11) is,

$$\nu_{Le+T} = (1/\mathrm{yr})(10^{-6}/\mathrm{yr})\,(4.6 \times 10^{-4}\mathrm{yr} + 2.7 \times 10^{-3}\mathrm{yr}) = 3.2 \times 10^{-9}/\mathrm{yr} \tag{13}$$

Because this mean coincidence rate is three orders of magnitude less than the threshold of 10^{-6}/yr, there is no need to consider combinations of L_e and fire, and the live load combined with fire should consider only the sustained component, L_s.

Fire and Roof Live Load

Making plausible assumptions regarding the occurrence of repair events giving rise to miscellaneous roof live loads, the mean coincidence rate of $L_r + T$ from Equation (11) would be,

$$\begin{aligned}\nu_{Lr+T} &= (0.2/\text{yr})(10^{-6}/\text{yr})\,(4.6 \times 10^{-4}\text{yr} + 0.083\,\text{yr}) \\ &= 1.7 \times 10^{-8}/\text{yr}\end{aligned} \tag{14}$$

This mean coincidence rate would increase by two orders of magnitude if the roof covered an enclosed area of 10,000 m^2 rather than 100 m^2, placing the mean rate close to the screening threshold of 10^{-6}/yr. However, the mean fire incidence rate of $10^{-8}/m^2/yr$ assumed above may not apply to large public assembly occupancies, as noted previously; additional data should be sought to address this issue. While it is possible that the combination of fire and miscellaneous roof live load may have to be considered for certain structures supporting large roofs for assembly occupancies, this combination need not be required for ordinary building structures.

Fire and Snow Load

The mean rate of occurrence of a coincidence of fire and snow (in a temperate and severe winter climates) is estimated (assuming, conservatively, a snow season of 6 months) as,

$$\begin{aligned}\nu_{S+T} &= (10^{-6}/\text{yr})\,(2/\text{yr})(4.6 \times 10^{-4} + 0.02)/2 \\ &= 2.0 \times 10^{-8}/\text{yr (temperate)}\end{aligned} \tag{15a}$$

$$\begin{aligned}\nu_{S+T} &= (10^{-6}/\text{yr})\,(2/\text{yr})(4.6 \times 10^{-4} + 0.17)/2 \\ &= 1.7 \times 10^{-7}/\text{yr (severe)}\end{aligned} \tag{15b}$$

It may be concluded that the combination of fire and snow load need not be considered for occupancies in which compartment areas are 1000 m^2 or less. For very large public assembly occupancies, if the value of ν_{S+T} were found to approach the threshold of 10^{-6}/yr, as remarked in the discussion following Equation (14), the combination of snow load and fire effects might have to be considered in the design of columns that are some distance removed from the direct effect of the fire.

Fire and Wind Load

Using the statistics presented earlier, the mean rate of a coincidence of nontropical windstorms and severe fire is,

$$\nu_{W+T} = (10^{-6}/\text{yr})\,(4/\text{yr})(4.6 \times 10^{-4}\text{yr} + 4.6 \times 10^{-4}\text{yr}) = 3.7 \times 10^{-9}/\text{yr} \tag{16}$$

Even for very large fire-exposed areas, the mean rate of occurrence of the coincident event is less than 10^{-6}/yr. Furthermore, in hurricane-prone coastal areas, the mean rate of occurrence of hurricane windstorms is much less than 4/yr, making the coincidence rate even less than in Equation (16). Thus, combinations of fire and wind load need not be considered.

Fire and Earthquake Load

The mean rate of a coincidence of fire and earthquake effect is estimated as,

$$\begin{aligned}\nu_{E+T} &= (10^{-6}/\text{yr})\,(0.10/\text{yr})(4.6 \times 10^{-4}\text{yr} + 9.5 \times 10^{-7}\,\text{yr}) \\ &= 4.6 \times 10^{-11}/\text{yr (WUS)}\end{aligned} \tag{17a}$$

$$\begin{aligned}\nu_{E+T} &= (10^{-6}/\text{yr})\,(0.02/\text{yr})(4.6 \times 10^{-4}\text{yr} + 3.2 \times 10^{-7}\,\text{yr}) \\ &= 9.2 \times 10^{-12}/\text{yr (EUS)}\end{aligned} \tag{17b}$$

in which Equations (17a) and (17b) apply to buildings sited in the Western and Eastern US. If the earthquake induces the fire, the two events are not statistically independent, as assumed in Equations (17). The model of load occurrences and load coincidences can be modified to take this dependence into account [39]. If some fires occur completely at random with mean rate ν_T while others are initiated by earthquakes, with probability, p, and random delay time, T_d, the overall occurrence of fires can be modeled by a Poisson process, with mean rate $\nu_T{}^* = \nu_T + p\nu_E$. Collectively, the earthquake and fire events tend to cluster because of the common term ν_E. If the event durations and T_d are modeled by exponential distributions, the mean rate of coincidence is approximately,

$$\nu_{E+T} = \nu_T{}^*\,\nu_E(\tau_T + \tau_E)[1 + p\nu_E/(\nu_T{}^*\,\nu_E\,\mu_{Td})] \tag{18}$$

in which μ_{Td} is the mean delay time. However, the delay from fire ignition to flashover or development of a structurally significant fire (several minutes) is substantially greater than the duration of significant ground motion (30 s or less). Thus, the check of general structural integrity under fire conditions would not need to consider the lateral forces due to the earthquake; gravity effects would be sufficient. However, the effective (increased) incidence rate of fire, $\nu_T{}^*$, should be considered in Equations (13)–(15) to determine whether to consider combinations involving live loads or snow loads.

STRUCTURAL DESIGN FOR FIRE CONDITIONS

Combinations of structural actions due to occupant activities, environmental, and accidental events must be provided in a conventional load factor format for practical implementation in structural design.

Load Combinations

Load combinations involving a combination of fire with lateral loads due to wind or earthquake have been screened out due to their low mean coincidence rate. The remaining load combinations involve fire and gravity load effects, and are required for design based on advanced structural analysis or for designing a suitable fire test of a structural component or assembly. Noting that $P[F]$ in Equation (6) is limited to less than 10^{-6}/yr, and having screened out load combinations with mean coincidence rates less than 10^{-6}/yr, the probability $P[U_{\max} > U_d]$ is set at approximately 0.1 and the load factors in the principal action/companion action load combination are adjusted accordingly. This results in,

$$U_d = 1.2D + T + 0.5L + (0.5L_r \text{ or } 0.2S) \tag{19}$$

in which T denotes the structural action resulting from the postulated fire scenario. It should be noted that the maximum structural action may occur when one (or more) of the loads equals zero. As might be noted from the brief review of probabilistic load models presented above, the companion actions $0.5L$, $0.5L_r$, and $0.2S$ represent the most probable values of load on the structure at the time of the fire.

It is assumed that uncertainty in the structural action due to the fire is subsumed in the determination of T, as is the practice in modern earthquake-resistant design, and thus the load factor on T is set equal to 1.0. Future research efforts should be directed to specifying T for different fire scenarios, occupancies, compartment ventilation, and bounding surfaces [20,24].

Lateral Stability Under Gravity and Fire Loads

The lateral stability of building frames normally is ensured by designing the frames for lateral forces from wind and/or earthquake effects. The above event combination analyses show that neither wind nor earthquake effects need be considered in checking the overall behavior of a building frame during a severe fire. However, few building frames are perfectly symmetric or symmetrically loaded by gravity loads. Moreover, columns and beams are not perfectly straight; nor are fabrication and erection procedures perfect. Consequently, even a "perfect" frame is subject to sway. If this sway is not accounted for, or if the imposed deformations from the fire give rise to significant frame deformations, large secondary (P-delta) forces will develop in the frame and lead to overall instability of the frame under gravity loads.

This event can be mitigated by stipulating that the lateral stability of the building frame be checked by imposing a small notional lateral force equal in magnitude to $0.002\Sigma P$ at each floor level, in which the term ΣP is the cumulative gravity force due to the summation of dead and live loads acting on the story above that level. This approach to ensuring lateral stability under gravity loads has been recommended by the Structural Stability Research Council [40], and is being implemented in several modern standards [e.g., 5,27].

Structural Resistance

The above load combinations determine the required strength of the building frame (strength that must be provided in design) from structural analysis. The design strength is determined by [27,41],

$$\text{Required strength} < \text{Design strength}\,(\phi\, R_n) \tag{20}$$

in which R_n is the nominal strength stipulated in the material specification or code (e.g., strength in tension, flexure, shear or compression) and ϕ is the resistance factor that takes into account uncertainties in the determination of R_n and mode and consequences of failure. The design strength and deformations should be calculated taking the elevated temperature properties of the structural materials into account [29,42]. The stability check of the frame should include second-order forces arising from differential heating of the structural system. However, the selection of specific resistance factors for governing structural limit states is the responsibility of standard-writing groups for the individual construction materials [e.g., 27,41] and is outside the scope of this paper.

CONCLUSIONS

Practical design requirements and load combinations for demonstrating compliance with PBE requirements for fire resistance, by either advanced structural analysis or qualification testing, can be developed to be consistent with a level of performance expressed in probabilistic terms.

The probability of a coincidence of fire with maximum live load, miscellaneous roof live load, significant wind storms, or earthquakes is negligible. Load combination requirements involving these loads in combination with structural actions due to fire are unnecessary and their use would waste resources and result in uneconomical design requirements and solutions.

Research should be undertaken to develop realistic fire exposures for different occupancies and compartment characteristics (e.g., ventilation, materials, fuel load) to make it possible to obtain improved appraisals of performance of building systems during severe fires. The load combination and design requirements in this paper provide a framework for measuring performance of building structural systems and determining and evaluating alternative cost-effective solutions in the public interest.

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