

# Load and Resistance Factor Criteria for Progressive Collapse Design

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### ABSTRACT

A progressive collapse initiates from a local structural failure and propagates, by a chain reaction mechanism, into a failure that involves a major portion of the structural system. The aftermath of the Ronan Point collapse in 1969 saw numerous attempts in the 1970's to develop criteria for progressive collapse resistance. Improved building practices and design procedures to control the likelihood of progressive collapse are receiving renewed interest by standards organizations in the United States and elsewhere in the aftermath of the tragedy of September 11, 2001. Procedures for assessing the capability of a damaged structure to withstand damage without the development of a general structural collapse can be developed using concepts of structural reliability analysis and probability-based limit states design. This paper describes design strategies to minimize the likelihood of progressive collapse, and prospects for the implementation of general provisions in national standards such as ASCE Standard 7, *Minimum design loads for buildings and other structures*.

### INTRODUCTION

A progressive collapse of a building is a catastrophic partial or total failure that ensues from an initiating event that causes local damage that cannot be absorbed by the inherent continuity and ductility of the building structural system. Following this local damage or failure, a chain reaction of failures propagates vertically or horizontally and develops into an extensive partial or total collapse, where the resulting damage is disproportionate to the local damage caused by the initiating event. Such collapses can be initiated by many causes, including design and construction errors and events that are beyond the design basis or are not considered explicitly in design. Such events would include abnormal loads not normally considered in design (e.g., gas explosions, vehicular collisions, and sabotage), severe fires, extreme values of environmental loads that stress the building system well beyond the design envelope, and misuse. Continuous highly redundant framed structures tend to absorb local damage well; other systems, such as large-panel or bearing wall systems or precast concrete slabs or steel joist floors supported on masonry walls, are inherently more vulnerable because of the difficulties in providing continuity and ductility in such systems. However, all buildings are susceptible to progressive collapse in varying degrees (Ellingwood and Leyendecker 1978). It has been estimated that approximately 15 to 20 percent of building collapses develop in this manner (Leyendecker and Burnett 1976).

The normal structural design process usually provides a certain amount of strength and ductility that is also available to withstand abnormal loads and progressive collapse. However, the evolution in building systems made possible by the widespread use of the computer in design has enabled engineers to take much of the inherent robustness out of the structural system. This, along with the use of high-performance materials, has led to building systems that are light and flexible, may be vulnerable to load conditions outside the design envelope, and may have little inherent energy-absorbing capacity or built-in protections against progressive collapse. Construction technologies aimed at minimizing erection costs also may lead to structures with little inherent resistance to progressive collapse. Social and political factors also have led to an increase in incidents that may initiate such failures. Finally, public awareness of building safety issues has increased markedly during the past thirty years as a result of building performance during well-publicized natural and man-made disasters. Structural failures, whatever their cause, are thoroughly examined in the public arena. In this environment, progressive collapse is likely to become an increasing regulatory issue.

No building system can be engineered and constructed to be absolutely risk-free because of uncertainties in demands on the system, in engineering properties of construction materials and in predictions of building system performance from the current generation of design software. Building codes are among the tools used by structural engineers for managing risk in the interest of public safety. The provisions for structural design in codes and standards for load combinations and design strength address the risks in building performance as the code and standard-writers have historically understood them. Progressive collapses, while uncommon, inevitably elicit an emotional public response because of the suddenness with which they occur and their catastrophic human and economic consequences. Demands for regulatory action are understandable. In the circumstances, building code officials and other regulatory authorities may be pressured into making decisions that may have a negative impact on building design and construction with little real risk reduction. Decisions or rapid code changes that take place in such a charged atmosphere are unlikely to be good decisions that lead to long-term improvements in building practices. For such lasting improvements, the development and implementation of improved building practices for progressive collapse must be removed from the social and regulatory climate that invariably exists following a major building failure by progressive collapse.

This paper considers the development and implementation of design requirements for progressive collapse mitigation in the current building regulatory climate, and the prospects of developing load and resistance factor criteria for progressive collapse design. Its scope is limited to completed buildings rather than those under construction. Despite the fact that a number of progressive collapses of buildings have occurred during their construction phases, the attitudes of the public toward such collapses, and safety during construction in general, is different from that of completed buildings and other structures (Allen and Schriever 1973). The responsibility of the regulatory community for devising methods for dealing with such collapses lies in a different direction.

# CURRENT BUILDING CODE APPROACHES TO PROGRESSIVE COLLAPSE

## Historical Perspective

The concern regarding progressive collapse as a structural engineering issue was focused by the collapse of the Ronan Point Tower, a residential apartment building in Canning Town, London, UK, in May, 1968, two months following initial occupancy of the building. Ronan Point was a 22-story building, with pre-cast concrete panel bearing wall construction. An explosion of natural gas from the building service system on the 18<sup>th</sup> floor failed an exterior bearing wall panel, which led to loss of support of floors above and subsequent collapse of floors below due to impact of debris. Subsequent to the report of the commission of inquiry, a number of codes and standards in the United States, Canada and Western Europe implemented provisions aimed at minimizing the future likelihood of such occurrences. How several of these codes have dealt with this issue is summarized below.

## Specific Approaches to Progressive Collapse-resistant Design

Most structural design standards in North America and in Western Europe have acknowledged the existence and potential consequences of abnormal loads and progressive collapse for some time. Most standards contain a statement of required structural performance, to the effect that local damage to the structure shall not have catastrophic consequences. In some codes, accidental loads are acknowledged explicitly (e.g., United Kingdom, Sweden, The Netherlands). Most recognize the desirability for continuity between structural elements, and several specify minimum tie forces to achieve continuity. Some specify the “notional removal” of an external load-bearing element; others, a floor area or volume of damage that the remaining structure is required to bridge. The applicability of provisions varies from country to country – in some, they apply to practically all buildings, while in others only to certain forms of construction or buildings over a certain minimum height (typically 5 to 6 stories).

*United States* - ASCE 7/ANSI A58 first introduced a requirement for progressive collapse due to “local failure caused by severe overloads” in Section 1.3.1 of ANSI Standard A58.1-1972, the first edition following the 1968 Ronan Point collapse. No commentary or other guidance was provided. ANSI Standard A58.1-1982, Section 1.3, retitled General Structural Integrity, contained a more comprehensive performance statement, and referred to a greatly expanded commentary section and references for guidance. The 1988 and 1993 editions (now titled ASCE Standard 7) illustrated several structural system layouts that would lead to development of alternate load paths. Section 1.4 of ASCE 7-95 retained the performance requirement that a building be designed to sustain local damage, with the structural system as a whole remaining stable. However, the commentary was shortened, keeping the discussion of general design approaches to general structural integrity but eliminating the figures and other specific guidance. At the same time, a new Section 2.5 was added that required a check of strength and stability of structural systems under low-probability events, where required by the authority having jurisdiction (AHJ). The provisions in ASCE 7-98 and ASCE 7-02 are essentially the same as in the 1995 edition. The (non-mandatory) Commentary C2.5 recommends the following load combination for checking the ability of a damaged structure to maintain its overall stability for a short time following an abnormal load event:

$$(0.9 \text{ or } 1.2) D + (0.5 L \text{ or } 0.2 S) + 0.2 W \quad (1a)$$

in which D, L, W and S are specified dead, live, snow and wind loads determined according to Sections 3, 4, 6 and 7 of ASCE 7-02. This check suggests the notional removal of selected (presumably damaged) load-bearing elements at the discretion of the engineer without stipulating tolerable damage. If certain key elements in the structural system must be designed to withstand the effects of the accident (perhaps to allow the development of alternate load paths), they should be designed using the combination,

$$(0.9 \text{ or } 1.2) D + A_k + (0.5 L \text{ or } 0.2 S) \quad (1 b)$$

in which  $A_k$  is the postulated action due to the abnormal load. Normally, only the main load-bearing structure would be checked using these equations.

Building code officials in the United States have not been enthusiastic about provisions related to general structural integrity because they are difficult to cast in prescriptive code language and to enforce. Most building codes in the United States have not contained such provisions. Whether the new paradigm of performance-based engineering and the related new initiatives will impact this resistance remains to be seen.

*Canada* - Section 4.1.1.3(1) of the 1995 edition of the *National Building Code of Canada* (NBCC) requires structures to be designed for sufficient structural integrity to withstand all effects that may reasonably be expected to occur during the service life. Commentary C on Part 4 defines structural integrity as "the ability of the structure to absorb local failure without widespread collapse," and advises designers to consider and take measures against severe accidents with probabilities of occurrence of approximately  $10^{-4}$ /yr or more. Several general approaches - local resistance, minimum tie forces, provision of alternate paths of support - are suggested and a list of references is provided. The structural integrity provisions in the NBCC date back to the 1970 edition, which was issued not long after the Ronan Point collapse. The commentary guidelines were quite detailed through the 1977 edition, but since the 1980 edition they have stated in a more general way. Specific load combinations or other prescriptive measures currently are not presented.

*Eurocode 1* - The general design requirements in Section 2 of *Eurocode 1 –Actions on Structures, Part 1 – Basis of Design* (CEN 250 1994) state that a structures shall be “designed in such a way that it will not be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.” The engineer is permitted to choose a design method that eliminates or reduces the hazard, uses a structural system that is insensitive to the hazard, ties the system together, or to design so that the system can tolerate accidental removal of an element. The design load combination used to demonstrate compliance using a specific “accidental” action,  $A_k$ , is specified as,

$$D + A_k + \psi_1 Q_1 + \sum \psi_{2i} Q_i \quad (2a)$$

in which  $\psi_1$  and  $\psi_{2i}$  are companion action factors for "frequent" and "quasi-permanent" values of load, which depend on the load and are presented in a table. As an illustration for combinations of dead, live and snow load for light occupancies, we would have,

$$D + A_k + 0.5 L \quad (2b)$$

$$D + A_k + 0.2 S + 0.3 L \quad (2c)$$

$$D + A_k + 0.5W + 0.3 L \quad (2d)$$

### **Critique**

Structural design provisions for progressive collapse fall into two general categories: (1) Provide specific local resistance for the abnormal load, and (2) Develop alternate load paths. Within the second category are two sub-categories: (i) provide general continuity requirements through minimum connection/tie forces, and (ii) Identify a tolerable area or volume of building damage, and design the structural system to bridge over this damage volume. Both categories encounter difficulties in practical implementation:

*Specific local resistance* – In this approach, “hard spots” are designed in the structure, at areas that are believed to be prone to accidental loads (e.g., exterior columns at risk from vehicular collision or sabotage) or that may be required to develop alternate load paths. One such requirement that was proposed subsequent to Ronan Point was that key load-bearing elements surrounding residential compartments served with natural gas be designed to withstand a pressure of 34 kPa (720 psf). This enormous pressure was based on a series of tests that measured explosion pressures in residential compartments. One unattractive feature of such an approach is that it provides resistance to only one hazard. A second is that specifying such a load provides exactly the sort of information that one might require to defeat the design. Specific abnormal loads seldom can be designed against economically; it is better to eliminate the hazard or control the consequence of local damage (Breen and Siess 1979).

*Alternate load paths* - Designing the structural system to develop alternate load paths in the event of local damage is intuitively more attractive because it focuses the attention of the designer on the behavior of the structural system following the disaster. On the other hand, notional element removal may not be realistic, and the idea of tolerating a significant amount of local damage, even in the interest of preserving general structural integrity, is unattractive from a human point of view, particularly if such damage is accompanied by significant morbidity and mortality. Stipulating the amount of damage to be tolerated is a difficult social and regulatory decision. Moreover, the analysis of the structure in the damaged state is difficult and beyond the capabilities of some structural engineers.

## **PROBABILISTIC FORMULATION OF DESIGN REQUIREMENTS**

Consideration of competing hazards in risk management is an essential tool for maximizing return on investment of resources on technology and regulatory provisions that will enhance and improve building practices. It is not feasible technically or economically to consider in detail in design all hazards that might impact building performance. Moreover, some hazards would have little impact on building risk. A competing hazard model allows the technical community and decision-makers to screen out trivial hazards, to focus on those hazards that lead to unacceptable increases in building failure rates, and to devise appropriate risk mitigation strategies for those hazards (Stewart and Melchers 1997).

### **Competing hazard models**

Building failures can result from a number of hazards – occupancy loads and other demands, misuse, extreme environmental effects, fires, and other abnormal loads. If each of these distinct hazards is represented by an event,  $H_i$ , then the total probability of structural collapse can be written as,

$$P(F) = \sum P[F|DH_i] P[D|H_i] P[H_i] \quad (3)$$

in which  $F$  = structural collapse event,  $P[H_i]$  = probability of hazard  $H_i$ ,  $P[D|H_i]$  = probability of local damage, given that  $H_i$  occurs, and  $P[F|DH_i]$  = probability of collapse, given that hazard and local damage both occur. The term  $P[F]$  represents the overall probability of building failure, and must be limited to some socially acceptable value through a combination of professional practice and appropriate building regulation. While current building codes and standards and code enforcement keep failure rates at a (fortunately) very low level, no one knows exactly what a socially acceptable failure rate might be. However, there is evidence (Pate-Cornell 1994) that the *de minimis* risk, that risk below which society normally does not impose any regulatory guidance, is on the order of  $10^{-7}$ /year. For the sake of the following discussion, we may take  $10^{-7}$ /year as a target value, with the understanding that this requires a sociopolitical decision that is outside the scope of the current technical paper.

Proper management of risk in the built environment involves examining each of the terms in the above sum. If the probability  $P[H_i]$  is substantially less than the *de minimis* threshold, then the probability of damage or failure due that hazard is likely to contribute little to  $P[F]$ , and can be safely ignored. One of the difficulties with the current generation of codes and standards is that they are focused by tradition or for historical reasons on a relatively small subset of the hazards that might impact building performance – wind, earthquake, and so on - and on mitigating the risks associated with these hazards. Modern building practices, as well as sociopolitical changes, have highlighted a group of hazards that historically either have not been recognized as significant (explosions or detonations) or have been dealt with through deemed-to-satisfy clauses rather than through formal structural calculations (severe fires).

### **Incidence, Magnitude and Characteristics of Abnormal Loads**

Abnormal loads are, by definition, low-probability events and few buildings are ever exposed to them. Accordingly, such events are either not normally considered in structural design for economic reasons or addressed indirectly through passive protective measures rather than by explicit structural calculations. Abnormal loads may be grouped as pressure loads (e.g., explosions, tornado wind pressures), impact (e.g., vehicular collision, missile impact, debris, swinging objects during construction or demolition), or as deformation-related (fire, foundation subsidence). Characteristically, the loads usually act over a short period of time in comparison with ordinary design loads. They may be static or dynamic in their structural action, depending on the frequency content of the load and the dynamic response characteristics of the structural system affected (Ellingwood and Leyendecker 1978). It generally is believed that strategies to manage the risk of progressive collapse focus on methods that enable a damaged structural system to maintain its overall integrity following an abnormal load event (Breen and Siess 1979). Nevertheless, it seems desirable to understand the characteristics of some of these load events, for such knowledge is necessary in order to predict the extent of damage that might have to be tolerated by the building structural system.

There are some statistical data to estimate the incidence of certain abnormal load events, expressed in terms of mean rate of occurrence (Leyendecker and Burnett 1976; CIB W14 1983; Ellingwood and Corotis 1991). These are presented simply for illustrative purposes to provide a context for the subsequent discussion of feasible code and standard provisions. Mean rates of occurrence for gas explosions, bomb explosions and vehicular collisions are approximately:

Gas explosions (per dwelling):  $(2 \times 10^{-5}/\text{yr})$   
Bomb explosions (per dwelling):  $(2 \times 10^{-6}/\text{yr})$   
Vehicular collisions (per building):  $(6 \times 10^{-4}/\text{yr})$   
Fully developed fires (per building):  $(5 \times 10^{-8}/\text{m}^2/\text{yr})$

These mean rates of occurrence appear to be reasonably independent of building construction but are dependent on occupancy. Some incidences depend on building size (e.g., buildings with a large number of independent occupancies or living compartments are at higher risk for gas explosions or fire effects), accessibility or configuration (e.g., vehicular collisions or sabotage affect mainly ground story or basement areas). There have been no attempts to examine such incidences with respect to special building or occupancy characteristics [e.g., Is the primary occupancy governmental? Is the building monumental or otherwise a community symbol?]. Of course, not all incidents lead to local damage or failure.

Following the Ronan Point collapse, a number of studies were conducted to determine the residential compartment pressures developed during an explosion of natural (building service) gas. The pressures developed in tests conducted at the optimum air/fuel mixture seldom exceeded 17 kPa (2.5 psi or 360 psf). This is a very large load when compared to ordinary design loads due to live, wind and snow load, but is substantially less than the 34 kPa load that was being mentioned frequently as the “normative” abnormal load in several regulatory documents at the time. Bomb detonations, in contrast, create shock wave pressures that expand at a velocity on the order of  $10^3$  m/s and create (positive) pressures with duration on the order of 10 ms, depending on weight of charge and distance from the detonation. Most of the

information on the structural impact of such detonations has been gathered by the military and is not widely available in the general structural engineering community. Moreover, making this information generally available might also enable a terrorist to defeat any preventive design strategy taken, and thus is not a viable option. Forces due to vehicular collision (involving trains and barges, as well as trucks) may be obtained from an analysis that relates the energy absorbed by the vehicle and the structure itself during the collision.

In contrast to the preceding hazards, advances appear possible in the short term in fire-resistant structural design. The ignition of a fire can be modeled as a Poisson event, with a mean rate of occurrence that is related to floor area, as in the example above [an office occupancy with alarm/sprinkler system was assumed]. Given that a fire ignites and flashover occurs, a temperature-time curve (fire exposure curve) can be produced [that depends on fuel load and compartment ventilation] that can be used in advanced structural analysis to assess the structural response during the heating and cooling phases following exhaustion of combustibles. Recent full-scale building fire tests (Bennetts and Thomas 2002) indicate suggest that advanced structural analysis can reproduce the actual behavior of the structure reasonably well.

Designing against any of the above hazards requires that (set of) a specific abnormal event scenarios be postulated. Determining the magnitudes of abnormal loads generated in these scenarios is problematic. Moreover, in all cases, there are significant uncertainties in the determination of the forces to be absorbed by the building structural system and their consequences.

### **Reliability Bases for Design for Conditional Limit States**

Basic progressive collapse prevention strategies must be aimed at three basic levels: (1) to prevent the occurrence of accidental loads through social or political means; (2) to prevent the occurrence of local significant structural damage that is likely to initiate a progressive collapse; and (3) to prevent structural system collapse and loss of life through structural design, compartmentalization, development of alternate load paths, alternate exitways, and other active and passive measures. Accidental loads occur randomly in space and in time, and have uncertain magnitudes. Similarly, the variables describing the capacity of structural members and systems and other loads that act at the time the accidental event occurs are also random. Accordingly, structural reliability principles are required to devise appropriate strategies for managing this risk.

Let us consider the term in the summation in eq (3) that is related to progressive collapse due to a postulated abnormal load. Let  $H_i$  be the event that a potentially damaging load occurs,  $D$  be the event that structurally significant local damage occurs, and  $F$  be the event that structural collapse ensues from this local damage. The probability of structural collapse due to this postulated event then is,

$$P(F) = P(F|DH_i) P(D|H_i) P(H_i) \quad (4)$$

This breakdown of the collapse probability is instructive to focus attention on appropriate strategies for hazard prevention, withstanding local damage, and absorbing local damage without progressive collapse. Reductions in  $P[F]$  can be accomplished by reducing any one, or all three, of the probabilities in this equation, and the most cost-effective strategy for most buildings is likely to involve some combination of the three. Note, for example, that  $P(H_i)$  is essentially independent of any structural design strategy taken to increase safety. It may be controlled by changes in siting the building or access to it, by controlling hazard substances within the building, and by educating the building occupants on the need for caution with dangerous substances or unauthorized access. Certain hazards may be dealt with most effectively in this manner (e.g., terrorist attack); other hazards involve the other two terms.

In a “specific local resistance” design strategy, the focus is on minimizing  $P(D|H_i)$ , that is, to minimize the likelihood of initiation of damage that may lead to progressive collapse. As noted previously, this strategy may be difficult or uneconomical, and may leave some significant hazards unaddressed. Accordingly, it is likely that  $P(D|H_i)$  will be very close to 1.0 in many practical code implementations, meaning that the collapse probability becomes, approximately,

$$P(F) = P(F|DH_i) P(H_i) \quad (5)$$

It is in minimizing the conditional probability,  $P(F|DH_i)$ , that the science and art of the structural engineer becomes paramount; such strategies might range from minimum provisions for continuity to a complete post-damage structural analysis, in which load-carrying mechanisms not normally considered in ordinary building design (e.g., membrane action of floors, large-deformations, significant nonlinear action) are mobilized.

*Models of accidental loads* - It may be assumed that the occurrence of an abnormal event,  $H_i$ , can be modeled as a Poisson process, with yearly mean rate of occurrence,  $\lambda_i$ . The probability of occurrence of this event during some reference period,  $T$ , is thus approximately  $P(H_i) = \lambda_i T$  for small  $\lambda_i$ . In the case of fire, residential gas explosions, and some other accidental loads, parameter  $\lambda_i$  may be related to building floor area affected or dwelling unit rather than the building. In that case,  $\lambda_i = p A_f$ , in which  $A_f$  = floor area and  $p = p_1 p_2$ ; term  $p_1$  = probability of occurrence of the hazard per unit area and  $p_2 < 1.0$  represents the effect of warning or control systems, if any, that would mitigate the likelihood of structurally damaging events.

*Structural reliability* - To evaluate  $P[F|DH_i]$ , one must first postulate a mathematical model,  $G(\mathbf{X}) = 0$ , of the structural system based on principles of mechanics and supplemented, where possible, with experimental data. The load and resistance variables are expressed by vector  $\mathbf{X}$ . We must then determine the probability distributions of each variable and integrate the joint density function of  $\mathbf{X}$  over that region of the probability space where  $G(\mathbf{X}) < 0$  to compute the conditional limit state probability. Alternatively, first-order (FO) reliability analysis may be used to compute a conditional reliability index,  $\beta$ , defined as,

$$\beta = \mu_G / \sigma_G \quad (6)$$

in which  $\mu_G$  and  $\sigma_G$  = mean and standard deviation of  $G(\mathbf{X})$ . This reliability index is related to  $P(F|DH_i)$  through (Ellingwood 1994; 2001),

$$\beta = \Phi^{-1} [P(F|DH_i)] \quad (7)$$

in which  $\Phi^{-1} ( )$  is the percent-point function of the standard normal probability distribution. With  $P(H_i) = \lambda_i T$ , eq (7) becomes,

$$\beta = \Phi^{-1} [P(F)/\lambda_i T] \quad (8)$$

The first-generation probability-based limit states design criteria (such as AISC LRFD, the Canadian CSA S16.1, and the Eurocodes) all are based, to varying degrees, on reliability of individual structural members and components. The technology to evaluate member limit state probabilities is mature (Ellingwood 1994), and evaluations performed independently by different authorities in different contexts lead to results that are of the same order of magnitude in most of the industrialized world. However, to implement reliability-based design criteria against progressive collapse in a practical sense, the limit state probability (or reliability index) must be evaluated for a structural system. In contrast to member reliability, this evaluation is difficult, even at the present state of the art and with computational resources available.

The identification of suitable conditional limit state functions also presents a significant research challenge. This identification should focus on the necessary conditions for overall stability of equilibrium following local damage. Thus, sources of load-carrying capacity that normally would not be considered in LRFD should be included in  $G(\mathbf{X})$ ; these might include membrane or catenary action in floor systems, substantial inelastic behavior of members and connections, and other load-resisting mechanisms accompanied by large inelastic deformations. Structural analysis should allow for geometric and material nonlinearities and should model connection behavior at extreme conditions accurately.

Assuming that an analysis of a damaged structure can be performed, an acceptable value of  $\beta$  upon which to base design for the conditional limit states is suggested by eq (8). Analysis of limit state probabilities for structural members currently designed for gravity loads, in which failures occur in a ductile fashion, suggests that these member failure probabilities (again, under gravity loads) are on the order of  $10^{-5}/\text{yr}$ . The probability of structural system failure is an order of magnitude less, depending on the redundancy in the system and the degree continuity between members (Ellingwood 2001). If  $\lambda_i = 10^{-6}$  to  $10^{-5}$  (cf hazards listed above), then the conditional probability should be on the order of  $10^{-2}$  to  $10^{-1}$ , and the target value of  $\beta$  should be on the order of 1.5. Load and resistance criteria can be developed to be consistent with this reliability.

### **Probability-Based Load Combinations and Resistance Criteria**

Modern probabilistic analysis of load combinations has shown that the maximum combined structural action due to several events the intensities of which vary in time occurs when one of the loads achieves its maximum value while the other loads are at their frequent values. In other words, the probability that two (or more) loads attain their maximum values

simultaneously is negligible, provided that the events are statistically independent. Thus, a structure may be designed for a combined action that is less than the sum of the maxima of the individual actions (Ellingwood, et al 1982). This observation, which has been verified both analytically and by simulation, has given rise to the “principal action-companion action” load combination format found in one form or another, in all modern probability-based limit states design codes, including ASCE Standard 7-02, *Minimum Design Loads for Buildings and Other Structures* (ASCE 2002).

As an example, the total live load on a floor system at any time is the sum of a sustained and an extraordinary component. The sustained component is the load normally present (including normal personnel load), and is typically measured during a live load survey. The mean of this sustained live load is on the order of 25 to 30% (depending on tributary loaded area) of the nominal ASCE 7-98 live load. The extraordinary component arises from remodeling or emergency crowding and has a short duration (on the order of hours to days). The extraordinary component is not measured during a load survey and equals zero most of the time. The nominal live load in ASCE Standard 7-02 is approximately equal to the mean of the maximum (sustained plus extraordinary) live load to occur in a period of 50 years (Chalk and Corotis 1980). The mean rate of occurrence of a coincidence of the extraordinary live load and a structurally significant fire has been found to be on the order of  $10^{-9}$ /yr or less (Ellingwood and Corotis 1991). How long the building must remain standing following the occurrence of local damage is an issue in determining the appropriate load combination. Certainly the building must maintain its overall stability long enough for the building occupants to evacuate safely, for rescue workers to perform needed tasks, and for a post-disaster condition assessment to be completed.

Two scenarios requiring load and resistance criteria can be envisioned for progressive collapse-resistant design. In the first, we consider criteria for evaluating the capability of a damaged structure to bridge over or around the damaged volume or area without a progressive collapse developing from the local damage. In the second, criteria are devised to check the (local) resistance to withstand a specific postulated accidental load.

In the first scenario, an appropriate load combination to base the reliability analysis of a building structural system (with local damage) would be,

$$\text{Dead} + \text{sustained live} + \text{daily snow} + \text{hourly maximum wind} \quad (9)$$

Using load statistics reported elsewhere (e.g., Galambos et al 1982), and principles of reliability analysis described earlier, we find that the required strength given by the load combination

$$(0.9 \text{ or } 1.2 D) + (0.5 L \text{ or } 0.2 S) + 0.2 W \quad (10a)$$

has an annual probability of approximately 0.05 of being exceeded. Note that the load factors on L and W are less than unity because the means of the live and wind loads in eq (9) are substantially less than the nominal loads specified in ASCE 7-02. While it has been suggested that the load factor on D might be taken as 1.0, engineers tend to underestimate the dead load

(Galambos, et al 1982); indeed, the mean dead load in modern construction is on the order of 5 to 10 percent higher than the nominally specified value. Accordingly, the dead load factor is kept as 1.2 (or 0.9 when dead load stabilizes the structural system), as with other load combinations.

In the second scenario, the objective is to calculate the resistance required to resist a postulated accidental load. Using a similar procedure the required strength becomes,

$$1.2 D + A_k + (0.5 L \text{ or } 0.2 S) \quad (10b)$$

$$(0.9 \text{ or } 1.2) D + A_k + 0.5 L + 0.2 W \quad (10c)$$

in which  $A_k$  = structural action due to the postulated abnormal load. This structural action can be a force, as in the case of explosion or impact, or deformation-related, as in the case of fire or ground subsidence.

The required strength from eqs (10) should not exceed the factored resistance,  $\phi R_n$ , in which  $R_n$  is determined by the standard or code. Since most code criteria for resistance are conservative, it is appropriate to set  $\phi = 0.95$  or  $1.0$ , depending on the uncertainty associated with the load-resisting mechanism considered. The assessment of building performance should be aimed at obtaining as realistic an estimate of structural performance as possible; arbitrary factors of safety should not be attached and propagated inconsistently throughout the analysis.

## IMPLEMENTATION IN CODES AND STANDARDS

The existence and potential consequence of abnormal loads and the possibility of progressive collapse should be addressed explicitly in codes and standards and become an integral part of the design process. This formal recognition draws the attention of the profession to the problem. Codes and standards of practice should stress that considering the possibility of progressive collapse is a necessary design requirement, regardless of whether the initiating event is an “accidental” or “normal” load, and that such effects must be accommodated from an overall structural safety viewpoint.

At the most general level, the need to consider accidental loads and progressive collapse in all types of building structures should be addressed through a performance requirement that appears in a document on General Design Principles and Load Requirements. There are a number of examples of such performance requirements, including those in ASCE Standard 7-02, Section 1.4 - General Structural Integrity, and the National Building Code of Canada, Section 4.1.1.3(1) and Commentary C. There should be some statement regarding the general scope of the provisions (e.g., all buildings in excess of four stories, etc.). Finally, this section should contain loading criteria for checking the ability of the structure to resist accidental loads. Generally, only the principal load-bearing system would need to be considered in these safety checks.

Specific design requirements for providing general structural integrity have proven very difficult to quantify for regulatory purposes because of the diversity of structural systems, construction technologies and methods, and possibilities for initial damage. Perhaps the easiest type of provision to implement is one in which minimum levels of continuity and ductility are provided through tie force requirements (Fintel and Schultz 1979) or through procedures that might be similar to basic earthquake-resistant design. Some European codes specify minimum tie forces to achieve continuity – figures ranging from 20 kN/m to 40 kN/m (1,371 to 2,741 lb/ft), depending on span, appear common. Such requirements avoid the need to specify either a tolerable damage area or volume or a specific accidental load. However, it is difficult to recommend minimum tie forces without having in mind some damage that must be bridged. This approach, with its emphasis on continuity and ductility, is similar to the philosophy toward earthquake-resistant design in low-to-moderate seismic areas.

Requirements pertaining to the ability of a damaged structure to carry loads are more difficult to implement in a code and to deal with in a design office. A major difficulty is in identifying the amount of damage that should be tolerated. There have been a number of proposals for doing this, including “notional” removal of one major load-bearing element at the perimeter of the building, or assuming a certain extent of damage by floor area or volume. However, such definitions may not be germane for some construction technologies or common practices. Moreover, while powerful computerized structural analysis utilizing large-deformation, nonlinear behavioral models have become available in recent years for performing the supporting structural analysis, experimental data to support such analyses may not be available. Connection performance during and after extreme events must be thoroughly understood, as it is essential to develop alternate load paths. This should be a research priority. Debris loading is significant, and its impact from floors above should be included in any post-damage assessment of the structural system.

The alternative of designing structures to withstand specific abnormal loads has some significant drawbacks. There are little data to define accidental load magnitudes; there is no assurance that the appropriate accidental load will be considered, nor that the design for one load will mitigate the consequences of another; and the member and connection design requirements may be excessive. On the other hand, it may be necessary to design certain members or regions in a structural system to withstand loads far in excess of what otherwise would be required under normal design conditions. Such local regions of strength may be required to provide support for floors or walls acting as story-height beams or tied arches, to anchor floors transmitting load through catenary action, and to ensure that alternative load paths can develop or provide zones of refuge within the building.

Common sense strategies can enhance the overall stability of a structural system and its ability to bridge over damaged zones. Good design practice requires that structural systems be robust and that their performance not be sensitive to uncertainties in occupancy or environmental loads or other influences not considered explicitly in design. These include selecting the layout of walls and columns to provide stability to walls and to limit the amount of wall that can be damaged. Short returns on walls increase their stability. Compartmentalized construction involving concrete walls that are reinforced to provide structural integrity (Corley et al 1998) or special moment frames of the type used in high seismic zones are inherently

stable forms of construction. Slender compression elements and brittle details should be avoided in critical points of potential alternate load paths, as should details that cause yielding in confined zones. Some of these building practices are summarized in the Commentary to ASCE Standard 7-02, Section 1.4.

Evaluation of building performance in a damaged state is clouded with uncertainty and imprecision. The calculations of overall building stability and equilibrium should be relatively simple and easily scrutinized. For buildings beyond a certain threshold, it would be advisable for a peer committee to review the proposed building design and the prospects of developing alternate load paths.

## **CONCLUSIONS**

Professionalism requires an acknowledgement of the fact that good design involves looking beyond the minimum design requirements in a code or standard. It is not possible to eliminate entirely the risk of progressive collapse of building construction in the current socio-economic climate. The building design team must take responsibility for demonstrating, as part of the design documentation, that steps have been taken to reduce the risk of progressive collapse to an acceptably small level. It is essential for structural engineers to understand the issues involved and to think the unthinkable at the conceptual design stage. It is at that stage where general structural integrity for specific structural systems can be addressed most effectively. Furthermore, structural engineers must strive to communicate these concerns regarding consequences of extreme events on building performance to building developers, architects, owners and occupants. Risk communication must become a significant part of arriving at acceptable strategies for progressive collapse prevention, and its importance in arriving at both socially acceptable and technically feasible solutions cannot be overemphasized. Finally, structural engineers must be educated to think in terms of system behavior and design rather than member behavior and design. This will require a paradigm shift in structural engineering education, but this shift is becoming feasible with the next generation of structural analysis courseware.

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