the original event is sufficiently small (Carper and Smilowitz 2006 and NIST 2007). The philosophy of designing to limit the spread of damage rather than to prevent damage entirely is different from the traditional approach to designing to withstand dead, live, snow, and wind loads, but is similar to the philosophy adopted in modern earthquake-resistant design.

In general, structural systems should be designed with sufficient continuity and ductility that alternate load paths can develop following individual member failure so that failure of the structure as a whole does not ensue. At a simple level, continuity can be achieved by requiring development of a minimum tie force, say 20 kN/m (1.37 kip/ft), between structural elements (NIST 2007). Member failures may be controlled by protective measures that ensure that no essential load-bearing member is made ineffective as a result of an accident, although this approach may be more difficult to implement. Where member failure would inevitably result in a disproportionate collapse, the member should be designed for a higher degree of reliability (NIST 2007).

Design limit states include loss of equilibrium as a rigid body, large deformations leading to significant second-order effects, yielding or rupture of members or connections, formation of a mechanism, and instability of members or the structure as a whole. These limit states are the same as those considered for other load events, but the load-resisting mechanisms in a damaged structure may be different and sources of load-carrying capacity that normally would not be considered in ordinary ultimate limit states design, such as arch, membrane, or catenary action, may be included. The use of elastic analysis underestimates the load-carrying capacity of the structure (Marjanishvili and Agnew 2006). Materially or geometrically nonlinear or plastic analyses may be used, depending on the response of the structure to the actions.

Specific design provisions to control the effect of extraordinary loads and risk of progressive failure are developed with a probabilistic basis (Ellingwood and Leyendecker 1978, Ellingwood and Corotis 1991, and Ellingwood and Dusenberry 2005). One can either reduce the likelihood of the extraordinary event or design the structure to withstand or absorb damage from the event if it occurs. Let F be the event of failure (damage or collapse) and A be the event that a structurally damaging event occurs. The probability of failure due to event A is

$$P_f = P(F|A)\, P(A) \qquad \text{(C2.5-1)}$$

in which $P(F|A)$ is the conditional probability of failure of a damaged structure and $P(A)$ is the probability of occurrence of event A. The separation of $P(F|A)$ and $P(A)$ allows one to focus on strategies for reducing risk. $P(A)$ depends on siting, controlling the use of hazardous substances, limiting access, and other actions that are essentially independent of structural design. In contrast, $P(F|A)$ depends on structural design measures ranging from minimum provisions for continuity to a complete post-damage structural evaluation.

The probability, $P(A)$, depends on the specific hazard. Limited data for severe fires, gas explosions, bomb explosions, and vehicular collisions indicate that the event probability depends on building size, measured in dwelling units or square footage, and ranges from about 0.2×10^{-6}/dwelling unit/year to about 8.0×10^{-6}/dwelling unit/year (NIST 2007). Thus, the probability that a building structure is affected may depend on the number of dwelling units (or square footage) in the building. If one were to set the conditional limit state probability, $P(F|A) = 0.05 - 0.10$, however, the annual probability of structural failure from Eq. C2.5-1 would be less than 10^{-6}, placing the risk in the low-magnitude background along with risks from rare accidents (Pate-Cornell 1994).

Design requirements corresponding to this desired $P(F|A)$ can be developed using first-order reliability analysis if the limit state function describing structural behavior is available (Ellingwood and Dusenberry 2005). The structural action (force or constrained deformation) resulting from extraordinary event A used in design is denoted A_k. Only limited data are available to define the frequency distribution of the load (NIST 2007 and Ellingwood and Dusenberry 2005). The uncertainty in the load due to the extraordinary event is encompassed in the selection of a conservative A_k, and thus the load factor on A_k is set equal to 1.0, as is done in the earthquake load combinations in Section 2.3. The dead load is multiplied by the factor 0.9 if it has a stabilizing effect; otherwise, the load factor is 1.2, as it is with the ordinary combinations in Section 2.3.2. Load factors less than 1.0 on the companion actions reflect the small probability of a joint occurrence of the extraordinary load and the design live, snow, or wind load. The companion actions $0.5L$ and $0.2S$ correspond, approximately, to the mean of the yearly maximum live and snow load (Chalk and Corotis 1980 and Ellingwood 1981). The companion action in Eq. 2.5-1 includes only snow load because the probability of a coincidence of A_k with L_r or R, which have short durations in comparison to S, is negligible. A similar set of load combinations for extraordinary events appears in *Eurocode 1* (2006).