Chapter C2
COMBINATIONS OF LOADS

Loads in this standard are intended for use with design specifications for conventional structural materials, including steel, concrete, masonry, and timber. Some of these specifications are based on allowable stress design, while others employ strength (or limit states) design. In the case of allowable stress design, design specifications define allowable stresses that may not be exceeded by load effects due to unfactored loads, that is, allowable stresses contain a factor of safety. In strength design, design specifications provide load factors and, in some instances, resistance factors. Structural design specifications based on strength design have been adopted by a number of specification-writing groups. Therefore, it is desirable to include herein common load factors that are applicable to these new specifications. It is intended that these load factors be used by all material-based design specifications that adopt a strength design philosophy in conjunction with nominal resistances and resistance factors developed by individual material-specification-writing groups. Load factors given herein were developed using a first-order probabilistic analysis and a broad survey of the reliabilities inherent in contemporary design practice. References [C2-1], [C2-2], and [C2-3] also provide guidelines for materials-specification-writing groups to aid them in developing resistance factors that are compatible, in terms of inherent reliability, with load factors and statistical information specific to each structural material.

C2.2 SYMBOLS AND NOTATION

Self-straining forces can be caused by differential settlement foundations, creep in concrete members, shrinkage in members after placement, expansion of shrinkage-compensating concrete, and changes in temperature of members during the service life of the structure. In some cases, these forces may be a significant design consideration. In concrete or masonry structures, the reduction in stiffness that occurs upon cracking may relieve these self-straining forces, and the assessment of loads should consider this reduced stiffness.

Some permanent loads, such as landscaping loads on plaza areas, may be more appropriately considered as live loads for purposes of design.

C2.3 COMBINING FACTORED LOADS USING STRENGTH DESIGN

C2.3.1 Applicability. Load factors and load combinations given in this section apply to limit states or strength design criteria (referred to as "load and resistance factor design" by the steel and wood industries) and they should not be used with allowable stress design specifications.

C2.3.2 Basic Combinations. Unfactored loads to be used with these load factors are the nominal loads of this standard. Load factors are from NBS SP 577 with the exception of the factor 1.0 for *E,* which is based on the more recent NEHRP research on seismic-resistant design [Ref. C2-15]. The basic idea of the load combination scheme is that in addition to dead load, which is considered to be permanent, one of the variable loads takes on its maximum lifetime value while the other variable loads assume "arbitrary point-in-time" values, the latter being loads that would be measured at any instant of time [Ref. C2-4]. This is consistent with the manner in which loads actually combine in situations in which strength limit states may be approached. However, nominal loads in this standard are substantially in excess of the arbitrary point-in-time values. To avoid having to specify both a maximum and an arbitrary point-in-time value for each load type, some of the specified load factors are less than unity in combinations 2 through 6.

Load factors in Section 2.3.2 are based on a survey of reliabilities inherent in existing design practice. The load factor on wind load in combinations 4 and 6 was increased to 1.6 in ASCE 7-98 from the value of 1.3 appearing in ASCE 7-95. The reasons for this increase are twofold.

First, the previous wind load factor, 1.3, incorporated a factor of 0.85 to account for wind directionality, that is, the reduced likelihood that the maximum wind speed occurs in a direction that is most unfavorable for building response [Ref. C2-5]. This directionality effect was not taken into account in allowable stress design. Recent wind engineering research has made it possible to identify wind directionality factors explicitly for a number of common structures. Accordingly, new wind directionality factors, K_d, are presented in Table 6-4 of this standard. These factors now are reflected in the nominal wind forces, *W*, used in both strength design and allowable stress design. This change alone mandates an increase in the wind load factor to approximately 1.53.

Second, the value in ASCE 7-95, 1.3, was based on a statistical analysis of wind forces on buildings at sites not exposed to hurricane winds [Ref. C2-5]. Studies have shown that, owing to differences between statistical characteristics of wind forces in hurricane-prone coastal areas of the United States [Refs. C2-6, C2-7, C2-8] the probability of exceeding the factored (or design-basis) wind force, $1.3W$ is higher in hurricane-prone coastal areas than in the interior regions. Two recent studies [Refs. C2-9, C2-10] have shown that the wind load factor in hurricane-prone areas should be increased to approximately 1.5 to 1.8 (depending on site) to maintain comparable reliability.

To move toward uniform risk in coastal and interior areas across the country, two steps were taken. First, the wind speed contours in hurricane-prone areas were adjusted to take the differences in extreme hurricane wind speed probability distributions into account (as explained in Section C6.5.4); these differences previously were accounted for in ASCE 7-95 by the "importance factor." Second, the wind load factor was increased from 1.3 to 1.6. This approach (a) reflects the removal of the directionality factor, and (b) avoids having to specify separate load criteria for coastal and interior areas.

Exception (3) has been added to permit the companion load *S* appearing in combinations (2), (4), and (5) to be the balanced snow load defined in Sections 7.3 for flat roofs and 7.4 for sloped roofs. Drifting and unbalanced snow loads, as principal loads, are covered by combination (3).

Load combinations 6 and 7 apply specifically to the case in which the structural actions due to lateral forces and gravity loads counteract one another.

Load factors given herein relate only to strength limit states. Serviceability limit states and associated load factors are covered in Appendix B of this standard.

This standard historically has provided specific procedures for determining magnitudes of dead, occupancy live, wind, snow, and earthquake loads. Other loads not traditionally considered by this standard may also require consideration in design. Some of these loads may be important in certain material specifications and are included in the load criteria to enable uniformity to be achieved in the load criteria for different materials. However, statistical data on these loads are limited or nonexistent, and the same procedures used to obtain load factors and load combinations in Section 2.3.2 cannot be applied at the present time. Accordingly, load factors for fluid load (F), lateral pressure due to soil and water in soil (H), and self-straining forces and effects (T) have been chosen to yield designs that would be similar to those obtained with existing specifications, if appropriate adjustments consistent with the load combinations in Section 2.3.2 were made to the resistance factors. Further research is needed to develop more accurate load factors because the load factors selected for H and F_a, are probably conservative.

Fluid load, F, defines structural actions in structural supports, framework, or foundations of a storage tank, vessel, or similar container due to stored liquid products. The product in a storage tank shares characteristics of both dead and live loads. It is similar to a dead load in that its weight has a maximum calculated value, and the magnitude of the actual load may have a relatively small dispersion. However, it is not permanent; emptying and filling causes fluctuating forces in the structure, the maximum load may be exceeded by overfilling; and densities of stored products in a specific tank may vary. Adding F to combination 1 provides additional conservatism for situations in which F is the dominant load.

It should be emphasized that uncertainties in lateral forces from bulk materials, included in H, are higher than those in fluids, particularly when dynamic effects are introduced as the bulk material is set in motion by filling or emptying operations. Accordingly, the load factor for such loads is set equal to 1.6.

C2.3.3 Load Combinations Including Flood Load. The nominal flood load, F_a, is based on the 100-year flood (Section 5.4). The recommended flood load factor of 2.0 in V Zones and Coastal A Zones is based on a statistical analysis of flood loads associated with hydrostatic pressures, pressures due to steady overland flow, and hydrodynamic pressures due to waves, as specified in Section 5.3.3.

The flood load criteria were derived from an analysis of hurricane-generated storm tides produced along the United States East and Gulf coasts [Ref. C2-10], where storm tide is defined as the water level above mean sea level resulting from wind-generated storm surge added to randomly phased astronomical tides. Hurricane wind speeds and storm tides were simulated at 11 coastal sites based on historical storm climatology and on accepted wind speed and storm surge models. The resulting wind speed and storm tide data were then used to define probability distributions of wind loads and flood loads using wind and flood load equations specified in Sections 6.5, 5.4, and in other publications (United States Army Corps of Engineers). Load factors for these loads were then obtained using established reliability methods [Ref. C2-2], and achieve approximately the same level of reliability as do combinations involving wind loads acting without floods. The relatively high flood load factor stems from the high variability in floods relative to other environmental loads. The presence of $2.0F_a$ in both combinations (4) and (6) in V Zones and Coastal A Zones is the result of high stochastic dependence between extreme wind and flood in hurricane-prone coastal zones. The $2.0F_a$ also applies in coastal areas subject to northeasters, extra tropical storms, or coastal storms other than hurricanes, where a high correlation exists between extreme wind and flood.

Flood loads are unique in that they are initiated only after the water level exceeds the local ground elevation. As a result, the statistical characteristics of flood loads vary with ground elevation. The load factor 2.0 is based on calculations (including hydrostatic, steady flow, and wave forces) with still-water flood depths ranging from approximately 4 to 9 ft (average still-water flood depth of approximately 6 ft), and applies to a wide variety of flood conditions. For lesser flood depths, load factors exceed 2.0 because of the wide dispersion in flood loads relative to the nominal flood load. As an example, load factors appropriate to water depths slightly less than 4 ft equal 2.8 [Ref. C2-10]. However, in such circumstances, the flood load generally is small. Thus, the load factor 2.0 is based on the recognition that flood loads of most importance to structural design occur in situations where the depth of flooding is greatest.

C2.3.4 Load Combinations Including Atmospheric Ice Loads. Load combinations 1 and 2 in Sections 2.3.4 and 2.4.3 include the simultaneous effects of snow loads as defined in Chapter 7 and Atmospheric Ice Loads as defined in Chapter 10. Load combinations 2 and 3 in Sections 2.3.4 and 2.4.3 introduce the simultaneous effect of wind on the atmospheric ice. The wind load on the atmospheric ice, W_i, corresponds to an event with approximately a 500-year Mean Recurrence Interval (MRI). Accordingly, the load factors on W_i and D_i are set equal to 1.0 and 0.7 in Sections 2.3.4 and 2.4.3, respectively. The rationale is exactly the same as that used to specify the earthquake force as $0.7E$ in the load combinations applied in working stress design. The snow loads defined in Chapter 7 are based on measurements of frozen precipitation accumulated on the ground, which includes snow, ice due to freezing rain, and rain that falls onto snow and later freezes. Thus the effects of freezing rain are included in the snow loads for roofs, catwalks, and other surfaces to which snow loads are normally applied. The atmospheric ice loads defined in Chapter 10 are applied simultaneously to those portions of the structure on which ice due to freezing rain, in-cloud icing, or snow accrete that are not subject to the snow loads in Chapter 7. A trussed tower installed on the roof of a building is one example. The snow loads from Chapter 7 would be applied to the roof with the atmospheric ice loads from Chapter 10 applied to the trussed tower. If a trussed tower has working platforms, the snow loads would be applied to the surface of the platforms with the atmospheric ice loads applied to the tower. If a sign is mounted on a roof, the snow loads would be applied to the roof and the atmospheric ice loads to the sign.

C2.4 COMBINING LOADS USING ALLOWABLE STRESS DESIGN

C2.4.1 Basic Combinations. The load combinations listed cover those loads for which specific values are given in other parts of this standard. However, these combinations are not all-inclusive, and designers will need to exercise judgment in some situations. Design should be based on the load combination causing the most unfavorable effect. In some cases this may occur when one or more loads are not acting. No safety factors have been applied to these loads, because such factors depend on the design philosophy adopted by the particular material specification.

The exception has been added to permit the companion load S appearing in combinations (4) and (6) to be the balanced snow load defined in Sections 7.3 for flat roofs and 7.4 for sloped roofs. Drifting and unbalanced snow loads, as principal loads, are covered by combination (3).

Wind and earthquake loads need not be assumed to act simultaneously. However, the most unfavorable effects of each should be considered separately in design, where appropriate. In some instances, forces due to wind might exceed those due to earthquake, while ductility requirements might be determined by earthquake loads.

Load combinations 7 and 8 were new to the 1998 edition of ASCE 7. They address the situation in which the effects of lateral or uplift forces counteract the effect of gravity loads. This eliminates an inconsistency in the treatment of counteracting loads in allowable stress design and strength design, and emphasizes the importance of checking stability. The earthquake load effect is multiplied by 0.7 to align allowable stress design for earthquake effects with the definition of E in Section 12.4, which is based on strength principles.

Most loads, other than dead loads, vary significantly with time. When these variable loads are combined with dead loads, their combined effect should be sufficient to reduce the risk of unsatisfactory performance to an acceptably low level. However, when more than one variable load is considered, it is extremely unlikely that they will all attain their maximum value at the same time. Accordingly, some reduction in the total of the combined load effects is appropriate. This reduction is accomplished through the 0.75 load combination factor. The 0.75 factor applies only to the variable loads, not to the dead load.

Some material design standards that permit a one-third increase in allowable stress for certain load combinations have justified that increase by this same concept. Where that is the case, simultaneous use of both the one-third increase in allowable stress and the 25 percent reduction in combined loads is unsafe and is not permitted. In contrast, allowable stress increases that are based upon duration of load or loading rate effects, which are independent concepts, may be combined with the reduction factor for combining multiple variable loads. In such cases, the increase is applied to the total stress; that is, the stress resulting from the combination of all loads. Load combination reduction factors for combined variable loads are different in that they apply only to the variable loads, and they do not affect the permanent loads nor the stresses caused by permanent loads. This explains why the 0.75 factor applies only to the sum of the variable loads, not the dead load.

In addition, certain material design standards permit a one-third increase in allowable stress for load combinations with one variable load where that variable is earthquake load. This standard handles allowable stress design for earthquake loads in a fashion to give results comparable to the strength design basis for earthquake loads as explained in the Chapter 9 Commentary section titled "Use of Allowable Stress Design Standards."

C2.4.2 Load Combinations Including Flood Load. The basis for the load combinations involving flood load is presented in detail in Section C2.3.3 on strength design. Consistent with the treatment of flood loads for strength design, F_a has been added to load combinations 3 and 4; the multiplier on F_a aligns allowable stress design for flood load with strength design.

C2.4.3 Load Combinations Including Atmospheric Ice Loads. See Section C2.3.4.

C2.5 LOAD COMBINATIONS FOR EXTRAORDINARY EVENTS

ASCE Standard 7 Section C1.4 recommends approaches to providing general structural integrity in building design and construction. Commentary C2.5 explains the basis for the load combinations that the designer should use if the Direct Design alternative in Section C1.4 is selected. If the authority having jurisdiction requires the Indirect Design alternative, that authority may use these load requirements as one basis for determining minimum required levels of strength, continuity, and ductility. Generally, extraordinary events with a probability of occurrence in the range 10^{-6} through 10^{-4}/yr or greater should be identified, and measures should be taken to ensure that the performance of key load-bearing structural systems and components is sufficient to withstand such events.

Extraordinary events arise from extraordinary service or environmental conditions that traditionally are not considered explicitly in design of ordinary buildings and structures. Such events are characterized by a low probability of occurrence and usually a short duration. Few buildings are ever exposed to such events and statistical data to describe their magnitude and structural effects are rarely available. Included in the category of extraordinary events would be fire, explosions of volatile liquids or natural gas in building service systems, sabotage, vehicular impact, misuse by building occupants, subsidence (not settlement) of subsoil, and tornadoes. The occurrence of any of these events is likely to lead to structural damage or failure. If the structure is not properly designed and detailed, this local failure may initiate a chain reaction of failures that propagates throughout a major portion of the structure and leads to a potentially catastrophic collapse. Approximately 15 percent–20 percent of building collapses occur in this way [Ref. C2-11]. Although all buildings are susceptible to progressive failures in varying degrees, types of construction that lack inherent continuity and ductility are particularly vulnerable [Refs. C2-12, C2-13].

Good design practice requires that structures be robust and that their safety and performance not be sensitive to uncertainties in loads, environmental influences, and other situations not explicitly considered in design. The structural system should be designed in such a way that if an extraordinary event occurs, the probability of damage disproportionate to the original event is sufficiently small [Ref. C2-14]. 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 [Ref. C2-15].

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, between structural elements [C2-24]. 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 [Ref. C2-16]. In either approach, an enhanced quality assurance and maintenance program may be required.

Design limit states include loss of equilibrium as a rigid body, large deformations leading to significant second-order effects, yielding or rupture of members of 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 a membrane or catenary action, may be included. The use of elastic analysis vastly underestimates the load-carrying capacity of the structure. 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 can be developed with a probabilistic basis [Refs. C2-17, C2-18]. One can either attempt to 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 and A be the event that a structurally damaging event occurs. The probability of failure due to event A is

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

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 0.23×10^{-6} /dwelling unit/year to about 7.8×10^{-6}/ dwelling unit/year [Refs. C2-19, C2-17]. 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, $P(F|A) = 0.1 - 0.2$/yr, however, the annual probability of structural failure from Eq. C2.5-1 would be on the order of 10^{-7} to 10^{-6}, placing the risk in the low-magnitude background along with risks from rare accidents [Ref. C2-20].

Design requirements corresponding to this desired $P(F|A) = 0.1 - 0.2$ can be developed using first-order reliability analysis if the limit state function describing structural behavior is available [Refs. C2-2, C2-3]. As an alternative, one can leave material and structural behavior considerations to the responsible material specifications and consider only the load combination aspect of the safety check, which is more straightforward.

For checking a structure to determine its residual load-carrying capacity following occurrence of a damaging extraordinary event, selected load-bearing elements should be notionally removed and the capacity of the remaining structure evaluated using the following load combination:

$$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W \tag{C2.5-2}$$

For checking the capacity of a structure or structural element to withstand the effect of an extraordinary event, the following load combinations should be used:

$$1.2D + A_k + (0.5L \text{ or } 0.2S) \tag{C2.5-3}$$

$$(0.9 \text{ or } 1.2)D + A_k + 0.2W \tag{C2.5-4}$$

The value of the load or load effect 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, and A_k must be specified by the authority having jurisdiction [Ref. C2-21]. 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. 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 action $0.5L$ corresponds, approximately, to the mean of the yearly maximum load [Ref. C2-22]. Companion actions $0.2S$ and $0.2W$ are interpreted similarly. A similar set of load combinations for extraordinary events appears in [Ref. C2-23]. The term $0.2W$ in these combinations is intended to ensure that the lateral stability of the structure is checked. Some recent standards require the stability of the structure to be checked by imposing a small notional lateral force equal in magnitude to $0.002\Sigma P$ at each floor level, in which ΣP is the cumulative gravity force due to the summation of dead and live loads acting on the story above that level. If such a stability check is performed, $0.2W$ need not be considered in combinations C2.5-2 and C2.5-4.

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