8 Rain Loads.

8.1 Symbols and Notation.

R = rain load on the undeflected roof, in pounds per square foot (kilonewtons per square meter) When the phrase "undeflected roof" is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

d_s = depth of water on the undeflected roof up to the inlet of the secondary drainage system when the primary drainage system is blocked (i e., the static head), in inches (millimeters)

d_h = additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow (i e , the hydraulic head), in inches (millimeters)

8.2 Roof Drainage. Roof drainage systems shall be designed in accordance with the provisions of the code having jurisdiction. The flow capacity of secondary (overflow) drains or scuppers shall not be less than that of the primary drains or scuppers.

8.3 Design Rain Loads. Each portion of a roof shall be designed to sustain the load of all rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

$R = 5.2\,(d_s + d_h)$ (In SI $R = 0.0098\,(d_s + d_h)$) (Eq 8-1)

If the secondary drainage systems contain drain lines such lines and their point of discharge shall be separate from the primary drain lines.

8.4 Ponding Instability. "Ponding" refers to the retention of water due solely to the deflection of relatively flat roofs Roofs with a slope less than 1/4 in /ft (1.19°) shall be investigated by structural analysis to assure that they possess adequate stiffness to preclude progressive deflection (i.e. instability) as rain falls on them or meltwater is created from snow on them. The larger of snow load or rain load shall be used in this analysis. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis

8.5 Controlled Drainage. Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation Such roofs shall be designed to sustain the load of all rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow (determined from Section 8 3)

Such roofs shall also be checked for ponding instability (determined from Section 8 4)

Commentary to American Society of Civil Engineers Standard ASCE 7-95

(This Commentary is not a part of the ASCE Standard Minimum Design Loads for Buildings and Other Structures. It is included for information purposes.)

This Commentary consists of explanatory and supplementary material designed to assist local building code committees and regulatory authorities in applying the recommended requirements. In some cases it will be necessary to adjust specific values in the standard to local conditions, in others, a considerable amount of detailed information is needed to put the provisions into effect. This Commentary provides a place for supplying material that can be used in these situations and is intended to create a better understanding of the recommended requirements through brief explanations of the reasoning employed in arriving at them.

The sections of the commentary are numbered to correspond to the sections of the standard to which they refer. Since it is not necessary to have supplementary material for every section in the standard, there are gaps in the numbering in the Commentary.

C1. General

C1.1 Scope. The minimum load requirements contained in this standard are derived from research and service performance of buildings and other structures. The user of this standard, however, must exercise judgement when applying the requirements to "other structures." Loads for some structures other than buildings may be found in Sections 3 to 10 of this standard and additional guidance may be found in the commentary.

Both loads and load combinations are set forth in this document with the intent of the that they be used together. If one were to use loads from some other source with the load combinations set forth herein or vice versa, the reliability of the resulting design may be affected.

Earthquake loads contained herein are developed for structures that possess certain qualities of ductility and post-elastic energy dissipation capability. For this reason provisions for design, detailing, and construction are provided in Appendix A. In some cases, these provisions modify or add to provisions contained in design specifications.

C1.3 Basic Requirements

C1.3.1 Strength. Buildings and other structures must satisfy strength limit states in which members are proportioned to carry the design loads safely to resist buckling, yielding, fracture, etc. It is expected that other standards produced under consensus procedures and intended for use in connection with building code requirements will contain recommendations for resistance factors for strength design methods or allowable stresses (or safety factors) for allowable stress design methods.

C1.3.2 Serviceability. In addition to strength limit states, buildings and other structures must also satisfy serviceability limit states which define functional performance and behavior under load and include such items as deflection and vibration. In the United States, strength limit states have traditionally been specified in building codes because they control the safety of the structure. Serviceability limit states, on the other hand, are usually non-catastrophic, define a level of quality of the structure or element and are a matter of judgement as to their application. Serviceability limit states involve the perceptions and expectations of the owner or user and are a contractual matter between the owner or user and the designer and builder. It is for these reasons, and because the benefits themselves are often subjective and difficult to define or quantify, that serviceability limit states for the most part are not included within the three model U.S. Building Codes. The fact that serviceability limit states are usually not codified should not diminish their importance. Exceeding a serviceability limit state in a building or other structure usually means that its function is disrupted or impaired because of local minor damage or deterioration or because of occupant discomfort or annoyance.

C1.3.3 Self-Straining Forces. Constrained structures that experience dimensional changes develop self-straining forces. Examples include moments in rigid frames that undergo differential foundation settlements and shears in bearing wall which support concrete slabs that shrink. Unless provisions are made for self-straining forces, stresses in structural elements, either alone or in combination with stresses from external loads, can be high enough to cause structural distress.

In many cases, the magnitude of self-straining forces can be anticipated by analyses of expected shrinkage, temperature fluctuations, foundation movement, etc. However, it is not always practical to calculate the magnitude of self-straining forces. Designers often provide for self-straining forces by specifying relief joints, suitable framing systems, or other details to minimize the effects of self-straining forces.

This section of the standard is not intended to require the designer to provide for self-straining forces that cannot be anticipated during design. An example is settlement resulting from future adjacent excavation.

C1.4 General Structural Integrity. Through accident or misuse, properly designed structures may be subject to conditions that could lead to either general or local collapse Except for specially designed protective systems, it is impractical for a structure to be designed to resist general collapse caused by gross misuse of a large part of the system or severe abnormal loads acting directly on a large portion of it. However, precautions can be taken in the design of structures to limit the effects of local collapse, that is, to prevent progressive collapse, which is the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it.

Since accidents and misuse are normally unforeseeable events, they cannot be defined precisely. Likewise, general structural integrity is a quality that cannot be stated in simple terms It is the purpose of 1.4 and this commentary to direct attention to the problem of local collapse, present guidelines for handling it that will aid the design engineer, and promote consistency of treatment in all types of structures and in all construction materials.

Accidents, Misuse, and Their Consequences. In addition to unintentional or willful misuse, some of the incidents that may cause local collapse are [1][3]: explosions due to ignition of gas or industrial liquids: boiler failures; vehicle impact; impact of falling objects, effects of adjacent excavations, gross construction errors; and very high winds such as tornadoes. Generally, such abnormal events would not be ordinary design considerations.

The distinction between general collapse and limited local collapse can best be made by example.

The immediate demolition of an entire structure by a high-energy bomb is an obvious instance of general collapse. Also, the failure of one column in a one-, two-, three-, or possibly even four-column structure could precipitate general collapse, because the local failed column is a significant part of the total structure at that level. Similarly, the failure of a major bearing element in the bottom story of a two- or three-story structure might cause general collapse of the whole structure. Such collapses are beyond the scope of the provisions discussed herein. There have been numerous instances of general collapse that have occurred as the result of such abnormal events as wartime bombing, landslides, and floods.

An example of limited local collapse would be the containment of damage to adjacent bays and stories following the destruction of one or two neighboring columns in a multibay structure. The restriction of damage to portions of two or three stories of a higher structure following the failure of a section of bearing wall in one story is another example. A prominent case of local collapse that progressed to a disproportionate part of the whole building (and is thus an example of the type of failure of concern here) was the Ronan Point disaster

Ronan Point was a 22-story apartment building of large, precast-concrete, loadbearing panels in Canning Town, England. In March 1968, a gas explosion in an 18th-story apartment blew out a living room wall. The loss of the wall led to the collapse of the whole corner of the building The apartments above the 18th story, suddenly losing support from below and being insufficiently tied and reinforced, collapsed one after the other The falling debris ruptured successive floors and walls below the 18th story, and the failure progressed to the ground. Another example is the failure of a one-story parking garage reported in [2]. Collapse of one transverse frame under a concentration of snow led to the later progressive collapse of the whole roof, which was supported by 20 transverse frames of the same type Similar progressive collapses are mentioned in [3]

There are a number of factors that contribute to the risk of damage propagation in modern structures [4]. Among them are,

1. There can be a lack of awareness that structural integrity against collapse is important enough to be regularly considered in design.

2. In order to have more flexibility in floor plans and to keep costs down, interior walls and partitions are often non-loadbearing and hence may be unable to assist in containing damage

3 In attempting to achieve economy in structures through greater speed of erection and less site labor, systems may be built with minimum continuity, ties between elements, and joint rigidity

4. Unreinforced or lightly reinforced load-bearing walls in multistory structures may also have inadequate continuity, ties, and joint rigidity

5. In roof trusses and arches there may not be sufficient strength to carry the extra loads or sufficient diaphragm action to maintain lateral stability of the adjacent members if one collapses

6 In eliminating excessively large safety factors, code changes over the past several decades have reduced the large margin of safety inherent in many older structures. The use of higher-strength materials permitting more slender sections compounds the problem in that modern structures may be more flexible and sensitive to load variations and, in addition, may be more sensitive to construction errors

Experience has demonstrated that the principle of taking precautions in design to limit the effects of local collapse is realistic and can be satisfied economically. From a public-safety viewpoint it is reasonable to expect all multistory structures to possess general structural integrity comparable to that of properly designed, conventional framed structures [4,5].

Design Alternatives. There are a number of ways to obtain resistance to progressive collapse. In [6] a distinction is made between direct and indirect design, and the following approaches are defined

Direct design: explicit consideration of resistance to progressive collapse during the design process through either:

> **alternate path method:** a method that allows local failure to occur but seeks to provide alternate load paths so that the damage is absorbed and major collapse is averted, or
>
> **specific local resistance method:** a method that seeks to provide sufficient strength to resist failure from accidents or misuse.

Indirect design: implicit consideration of resistance to progressive collapse during the design process through the provision of minimum levels of strength, continuity and ductility

The general structural integrity of a structure may be tested by analysis to ascertain whether alternate paths around hypothetically collapsed regions exist. Alternatively alternate path studies may be used as guides for developing rules for the minimum levels of continuity and ductility needed in applying the indirect design approach to ensuring general structural integrity. Specific local resistance may be provided in regions of high risk since it may be necessary for some elements to have sufficient strength to resist abnormal loads in order for the structure as a whole to develop alternate paths. Specific suggestions for the implementation of each of the defined methods are contained in [6]

Guidelines for the Provision of General Structural Integrity. Generally, connections between structural components should be ductile and have a capacity for relatively large deformations and energy absorption under the effect of abnormal conditions. This criterion is met in many different ways, depending on the structural system used. Details that are appropriate for resistance to moderate wind loads and seismic loads often provide sufficient ductility

Work with large precast panel structures [7,8,9] provides an example of how to cope with the problem of general structural integrity in a building system that is inherently discontinuous. The provision of ties combined with careful detailing of connections can overcome difficulties associated with such a system. The same kind of methodology and design philosophy can be applied to other systems [10]. The ACI Building Code Requirements for Reinforced Concrete [11] includes such requirements in Section 7.13.

There are a number of ways of designing for the required integrity to carry loads around severely damaged walls, trusses, beams, columns, and floors. A few examples of design concepts and details relating particularly, but not solely, to precast and bearing-wall structures are:

1. Good Plan Layout. An important factor in achieving integrity is the proper plan layout of walls (and columns). In bearing-wall structures there should be an arrangement of interior longitudinal walls to support and reduce the span of long sections of crosswall, thus enhancing the stability of individual walls and of the structures as a whole. In the case of local failure this will also decrease the length of wall likely to be affected.

2. Returns on Walls. Returns on interior and exterior walls will make them more stable

3. Changing Directions of Span of Floor Slab. Where a floor slab is reinforced in order that it can, with a low safety factor, span in another direction if a load-bearing wall is removed, the collapse of the slab will be prevented and the debris loading of other parts of the structure will be minimized. Often, shrinkage and temperature steel will be enough to enable the slab to span in a new direction.

4. Load-Bearing Interior Partitions. The interior walls must be capable of carrying enough load to achieve the change of span direction in the floor slabs.

5. Catenary Action of Floor Slab. Where the slab cannot change span direction, the span will increase if an intermediate supporting wall is removed. In this case, if there is enough reinforcement throughout the slab and enough continuity and restraint, the slab may be capable of carrying the loads by catenary action, though very large deflections will result.

6. Beam Action of Walls. Walls may be assumed to be capable of spanning an opening if

sufficient tying steel at the top and bottom of the walls allows them to act as the web of a beam with the slabs above and below acting as flanges (see [7]).

C1.5 Classification of Buildings and Other Structures. The categories in Table 1-1 are used to relate the criteria for maximum environmental loads or distortions specified in this standard to the consequence of the loads being exceeded for the structure and its occupants. The category numbering is unchanged from that in the previous edition of the standard (ASCE 7-95). Classification continues to reflects a progression of the anticipated seriousness of the consequence of failure from lowest hazard to human life (Category I) to highest (Category IV).

In Sections 6 and 7, importance factors are presented for the four categories identified. The specific importance factors differ according to the statistical characteristics of the environmental loads and the manner in which the structure responds to the loads. The principle of requiring more stringent loading criteria for situations in which the consequence of failure may be severe has been recognized in previous versions of this standard by the specification of mean recurrence interval maps for wind speed and ground snow load.

This section now recognizes that there may be situations when it is acceptable to assign multiple categories to a structure based on the expected occupancy of the structure during extreme loading events. For instance, a structure that is to be used as a hurricane shelter and has no expected use as a shelter during other extreme events, such as an earthquake, need not be designed for loads consistent with an earthquake shelter. In this case, the structure would be classified in Category IV for wind design and in Category II for seismic design.

Category I contains buildings and other structures that represent a low hazard to human life in the event of failure either because they have a small number of occupants or have a limited period of exposure to extreme environmental loadings. Category II contains all occupancies other than those in Categories I, III and IV and are sometimes referred to as "ordinary" for the purpose of risk exposure. Category III contains those buildings and other structures that have large numbers of occupants, are designated for public assembly or for which the occupants are restrained or otherwise restricted from movement or evacuation. Buildings and other structures in Category III therefore represent a substantial hazard to human life in the event of failure.

Buildings and other structures normally falling into Category III could be classified into Category II if means (secondary containment) are provided to contain the toxic or explosive substances in the case of a spill or to contain a blast. To qualify, secondary containment systems ~~must~~ should be designed, installed, and operated to prevent migration of harmful quantities of toxic or explosive substances out of the system to the air, soil, ground water, or surface water at any time during the use of the structure. This requirement is not to be construed as requiring a secondary containment system to prevent a release of any toxic or explosive substance into the air. By recognizing that secondary containment shall not allow releases of "harmful" quantities of contaminants, this standard acknowledges that there are substances that might contaminate ground water but do not produce a sufficient concentration of toxic or explosive material during a vapor release to constitute a health or safety risk to the public.

If the beneficial effect of secondary containment can be negated by external forces, such as the overtopping of dike walls by flood waters, then the buildings or other structures in question should not be classified into Category II. If the secondary containment is to contain a flammable substance, then implementation of a program of emergency response and preparedness combined with an appropriate fire suppression system would be a prudent action associated with a Category II classification. In many jurisdictions, such actions are required by local fire codes.

Buildings and other structures containing toxic or explosive substances also could be classified as Category II for hurricane wind loads when mandatory procedures are used to reduce the risk of release of hazardous substances during and immediately after these predictable extreme loadings. Examples of such procedures include draining hazardous fluids from a tank when a hurricane is predicted or, conversely, filling a tank with fluid to increase its buckling and overturning resistance. As appropriate to minimize the risk of damage to structures containing toxic or explosive substances, mandatory procedures necessary for the Category II classification should include preventative measures such as the removal of objects that might become air-borne missiles in the vicinity of the structure.

Category IV contains buildings and other structures that are designated as essential facilities and are intended to remain operational in the event of extreme environmental loadings. Such occupancies include, but are not limited to hospitals, fire, rescue and other emergency response facilities. Ancillary structures required for the operation of Category IV facilities during an emergency also are included in this category.

C1.7 Load Tests. No specific method of test for completed construction has been given in this standard, since it may be found advisable to vary the procedure according to conditions. Some codes require the construction to sustain a superimposed load equal to a stated multiple of the design load without evidence of serious damage. Others specify that the superimposed load shall be equal to a stated multiple of the live load plus a portion of the dead load. Limits are set on maximum deflection under load and after removal of the load. Recovery of at least three-quarters of the maximum deflection, within 24 hours after the load is removed, is a common requirement [11].

References

[1] Leyendecker, E.V., Breen, J E., Somes, N F., and Swatta, M.Abnormal loading on buildings and progressive collapse--An annotated bibliography. Washington, D C : U.S. Dept of Commerce. National Bureau of Standards. NBS BSS 67, Jan. 1976

[2] Granstrom, S., and Carlsson, M Byggfurskningen T3 Byggnaders beteende vid overpaverkningar [The behavior of buildings at excessive loadings] Stockholm. Sweden. Swedish Institute of Building Research. 1974

[3] Seltz-Petrash, A. Winter roof collapses Bad luck bad construction, or bad design Civil Engineering, Dec 1979, 42-45.

[4] Breen, J E., Ed Progressive collapse of building structures [summary report of a workshop held at the Univ of Texas at Austin, Oct 1975] Washington, D C.. U.S. Dept of Housing and Urban Development Rep. PDR-182. Sept 1976

[5] Burnett, E F.P. The avoidance of progressive collapse Regulatory approaches to the problem Washington, D C , U S Dept of Commerce. National Bureau of Standards. NBS GCR 75-48. Oct 1975 [Available from: National Technical Information Service. Springfield, Va]

[6] Leyendecker, E.V , and Ellingwood, B R Design methods for reducing the risk of progressive collapse in buildings Washington, D C. U S Dept. of Commerce, National Bureau of Standards. NBS BSS 98, 1977

[7] Schultz, D M., Burnett, E.F.P., and Fintel, M A design approach to general structural integrity, design and construction of large-panel concrete structures. Washington, D.C.. U.S. Dept. of Housing and Urban Development. 1977

[8] PCI Committee on Precast Bearing Walls Considerations for the design of precast bearing-wall buildings to withstand abnormal loads J Prestressed Concrete Institute. 21(2), 46-69, March/April 1976.

[9] Fintel, M , and Schultz, D M Structural integrity of large-panel buildings J. Am Concrete Inst , 76(5). 583-622. May 1979

[10] Fintel, M , and Annamalai, G Philosophy of structural integrity of multistory load-bearing concrete masonry structures Concrete Int , 1(5), 27-35. May 1979

[11] Building Code Requirements for Reinforced Concrete, ACI Standard 318-89, American Concrete Institute, Detroit, Mich., 1989

[12] NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1991 Edition, Part 1 Provisions, Building Seismic Safety Council, Washington, D C , 1991

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 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 limit states 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 [Ellingwood (1980), Galambos (1982) and Ellingwood (1982)] 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 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 Sections 3 through 9 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 (NEHRP, 1992). 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 [17]. 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 Sections 3 through 9 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 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) has been increased to 1.6 in the current standard from the value of 1.3 appearing in ASCE 7-95. The reasons for this increase are two fold.

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 windspeed occurs in a direction that is most unfavorable for building response (Ellingwood, 1981). This directionality effect was not taken into account in ASD. Recent wind engineering research has made it possible to identify wind directionality factors explicity for a number of common structures. Accordingly, new wind directionality factors, K_d, are presented in Table 6-6 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 previous value, 1.3, was based on a statistical analysis of wind forces on buildings at sites not exposed to hurricane winds (Ellingwood, 1981). Studies have shown that, owing to differences between statistical characteristics of wind forces in hurricane-prone coastal areas of the United States (Vickery and Twisdale, 1995, Whalen, 1996, Peterka and Shahid, 1998) 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 (Ellingwood and Tekie, 1997; Mehta, et. al. 1998) 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 windspeed contours in hurricane-prone areas were adjusted to take the differences in extreme hurricane wind speed probability distributions into account (as explained in 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.

Load combination 6 applies 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 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 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 load 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, Fa, is based on the 100-year flood (Section 5.3.3 1) 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 U.S East and Gulf coasts (Mehta, et al. 1991, 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 3 3 and in other publications (US Army Corps of Engineers) Load factors for these loads were then obtained using established reliability methods (Ellingwood, et al, 1981, 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 0Fa in both equations 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 0Fa 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 four to nine feet (average still-water flood depth of approximately six feet), 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 four feet equal 2 8 (Mehta, et al, 19981. However, in such circumstances, the flood load itself 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.

C2.4 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 judgement 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, since such factors depend on the design philosophy adopted by the particular material specification.

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 (4) and (5) are new in ASCE 7-98 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 9 2.2.6 which is based on strength principles

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 Reduction. 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 sum of the transient 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% 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 transient loads. Effects apply to the total stress; that is, the stress resulting from the combination of all loads *Load combination reduction* factors for combined transient loads are different in that they apply only to the transient loads, and they do not affect the permanent loads nor the stresses caused by permanent loads This explains why the 0.75 factor applied to the sum of all loads, to this edition, in which the 0.75 factor applies only to the sum of the transient loads, not the dead load.

Certain material design standards permit a one-third increase in allowable stress for load combinations with one transient load where that transient is ~~wind or~~ 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 C.9 Commentary section titled "Use of Allowable Stress Design Standards"

C2.5 Load Combinations for Extraordinary Events. ASCE Standard 7 Commentary 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 Commentary 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} - 10^{-4}/yr or greater should be identified, and measures should be taken to ensure that the performance of key loading-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 to 20 percent of building collapses occur in this way (Allen and Schriever, 1973). Although all buildings are susceptible to progressive failures in varying degrees, types of construction that lack inherent continuity and ductility are particularly vulnerable (Been and Siess, 1979) [2] [16]

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 (Commentary C, 1990). 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 (NEHRP, 1992).

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 [3]. 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 (NKB, 1987). 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, 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 (Ellingwood and Leyendecker, 1978; Ellingwood and Corotis, 1991). 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,

$$P_f = P(F|A)\ P\ (A) \qquad \text{C2 5.1}$$

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 0.23×10^{-6} /dwelling unit/year to about 7.8×10^{-6}/ dwelling unit/year (CIB W14, 1983, Ellingwood and Leyendecker, 1978). Thus, the probability that a building structure is affected depends 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 Eqn. 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 (Wilson and Crouch, 1987).

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 (Galambos, et al, 1982, Ellingwood, et al 1982). 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

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

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

$$1.2D + A_k + (0.5L \text{ or } 0.2S) \qquad C2.5.3$$

$$(0.9 \text{ or } 1.2)D + A_k + 0.2W \qquad C2.5.4$$

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 (Burnett, 1975). 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 (Chalk and Corotis, 1980). Companion actions 0.2S and 0.2W are interpreted similarly. A similar set of load combinations for extraordinary events appears in Eurocode 1 (1990).

REFERENCES

Allen, D.E. and Schriever, W. (1973). "Progressive collapse, abnormal loads and building codes." In Structural Failures: Modes, Causes, Responsibilities. ASCE New York, NY, pp. 21-48.

Breen, J.E. and Siess, C.P. (1979). "Progressive collapse - symposium summary." ACI Journal 76(9): 997-1004.

British standard structural use of steelwork in building (BS 5950: Part 8: 1990). British Standards Institution, London, U.K.

Burnett, E.F.P. (1975). "The avoidance of progressive collapse: regulatory approaches to the problem." National Bureau of Standards Report GCR 75-48, Washington, DC. (Available from NTIS, Springfield, VA.)

Chalk, P.L. and Corotis, R.B. (1980). "Probability models for design live loads." J. Str. Div. ASCE 106(10): 2017-2033.

CIB W14 (1983). "A conceptual approach towards a probability based design guide on structural fire safety." Fire Safety Journal 6(1): 1-79.

"Commentary C Structural Integrity." National Building Code of Canada, 1990, National Research Council of Canada, Ottawa, Ontario.

Ellingwood, B. and Leyendecker, E.V. (1978). "Approaches for design against progressive collapse." J. Str. Div. ASCE 104(3): 413-423.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. (1980). "Development of a probability-based load criterion for American National Standard A58." Washington, D.C.: U.S. Dept. of Commerce, National Bureau of Standards. NBS SP 577.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. (1982). "Probability-based load criteria: load factors and load combinations." J. Str. Div. ASCE 108(5): 978-997.

Ellingwood, B. and Corotis, R.B. (1991). "Load combinations for buildings exposed to fires." Engineering Journal, ASIC 28(1): 37-44.

Ellingwood, B. (1981). "Wind and snow load statistics for probabilistic design." Journal of Structural Engineering, ASCE, 107(7): 1345-1349.

Ellingwood, B. And Tekie, P.B. (1997). "Wind load statistics for probability-based structural design." Report prepared for NAHB Research Center, Upper Marlboro, MD (available through NTIS, also submitted to Journal of Structural Engineering, ASCE.

Mehta, K.C., et. al. (1998). "An investigation of load factors for flood and combined wind and flood." Report prepared for Federal Emergency Management Agency, Washington, D.C.

Peterka, J.A. and Shahid, S. (1998). "Design gust wind speeds in the United States." Journal of Structural Engineering, ASCE, 124(2): 207-214.

Vickery, P.J. and Twisdale, L.A. (1995). "Prediction of hurricane windspeeds in the United States." Journal of Structural Engineering, ASCE, 121(11): 1691-1699.

Whalen, T. (1996). "Probabilistic estimates of design load factors for wind-sensitive structures using the 'peaks over threshold' approach." NIST TN 1418, National Institute of Standards and Technology, Gaithersburg, MD.

Eurocode No. 1 (1990). "Common unified rules for different types of construction and material." Commission of the European Communities, Brussels.

Galambos, T.V., Ellingwood, B., MacGregor, J.G., and Cornell, C.A. (1982). "Probability-based load criteria: assessment of current design practice." J. Str. Div. ASCE 108(5): 959-977.

NKB (1987). "Guidelines for loading and safety regulations for structural design (1987)." NKB (Nordic Committee on Building Regulations) Report No. 55E, Copenhagen, Denmark.

"NEHRP recommended provisions for the development of seismic regulations for buildings (1992)." FEMA Report 222, Federal Emergency Management Agency, Washington DC

Taylor D.A. (1975) "Progressive collapse." Canadian J. Civil Engrg 2(4), 517-529.

Turkstra, C.J. and Madsen, H (1980). "Load combinations for codified structural design." J. Str Div. ASCE 106(12) 2527-2543

Wilson, R. and Crouch, E.A. (1987) "Risk assessment and comparisons an introduction." Science 236: 267-270 1