Standard 7-16

Minimum Design Loads and Associated Criteria for Buildings and Other Structures

SUPPLEMENT 1

Effective: December 12, 2018

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Chapter 6

SECTION 6.6.1 AS FOLLOWS:

6.6.1 Maximum Inundation Depth and Flow Velocities Based on Runup.

The maximum inundation depths and flow velocities associated with the stages of tsunami flooding shall be determined in accordance with Section 6.6.2. Calculated flow velocity shall not be taken as less than 10 ft / s (3.0 m / s) and need not be taken as greater than the lesser of $1.5(gh_{\max})^{1/2}$ and 50 ft / s (15.2 m / s).

Where the maximum topographic elevation along the topographic transect between the shoreline and the inundation limit is greater than the runup elevation, one of the following methods shall be used:

1. The site-specific procedure of Section 6.7.6 shall be used to determine inundation depth and flow velocities at the site, subject to the above range of calculated velocities.

2. For determination of the inundation depth and flow velocity at the site, the procedure of Section 6.6.2, Energy Grade Line Analysis, shall be used, assuming a runup elevation and horizontal inundation limit that has at least 100% of the maximum topographic elevation along the topographic transect.
3. Where the site lies within a completely overwashed area for which Inundation Depth Points are provided in the ASCE Tsunami Design Geodatabase, the inundation elevation profiles shall be determined using the Energy Grade Line Analysis with the following modifications:
    a. The Energy Grade Line Analysis shall be initiated from the inland edge of the overwashed land with an inundation elevation equal to the maximum topographic elevation of the overwashed portion of the transect.
    b. The Froude number shall be 1 at the inland edge of the overwashed land and shall vary linearly with distance to match the value of the Froude number determined at the shoreline per the coefficient α.
    c. The Energy Grade Line Analysis flow elevation profile shall be uniformly adjusted with a vertical offset such that the computed inundation depth at the Inundation Depth Point is at least the depth specified by the ASCE Tsunami Design Geodatabase, but the flow elevation profile shall not be adjusted lower than the topographic elevations of the overwashed land transect.

TABLE 6.10-1 AS FOLLOWS:

Table 6.10-1 Drag Coefficients for Rectilinear Structures

Width to Inundation Depth[a] Ratio $B/h_{\cancel{sx}}$	Drag Coefficient C_d
<12	1.25
16	1.3
26	1.4
36	1.5
60	1.75
100	1.8
≥ 120	2.0

a Inundation depth for each of the three Load Cases of inundation specified in Section 6.8.3.1.
Interpolation shall be used for intermediate values of width to inundation depth ratio $B/h_{\cancel{sx}}$.
Where building setbacks occur, drag coefficients shall be determined for each portion of a constant width. For each portion along the inundated height of the building, its equivalent inundated depth is taken as its submerged vertical dimension.

SECTION 6.12.4.1 AS FOLLOWS:

6.12.4.1 Fill.

Fill used for structural support and protection shall be placed in accordance with ASCE 24~~(2005)~~, Sections 1.5.4 and 2.4.1. Structural fill shall be designed to be stable during inundation and to resist the loads and effects specified in Section 6.12.2.

SECTION 6.17 AS FOLLOWS:

6.17 Consensus Standards and Other Referenced Documents

ASCE/SEI 24-14~~05~~, *Flood Resistant Design and Construction*, American Society of Civil Engineers,2015~~2005~~.
Cited in: Section 6.12.4.1

Chapter 11

TABLES 11.4-1 and 11.4-2 AS FOLLOWS:

Table 11.4-1 Short-Period Site Coefficient, F_a

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
Site Class	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use ~~straight-line~~ linear interpolation for intermediate values of S_s.

Table 11.4-2 Long-Period Site Coefficient, F_v

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
Site Class	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4

D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	3.3[a] ~~See Section 11.4.8~~	2.8[a] ~~See Section 11.4.8~~	2.4[a] ~~See Section 11.4.8~~	2.2[a] ~~See Section 11.4.8~~	2.0[a] ~~See Section 11.4.8~~
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use ~~straight-line~~ linear interpolation for intermediate values of S_1.

[a]~~Also s~~ See requirements for site-specific ground motions in Section 11.4.8. These values of F_v shall be used only for calculation of T_S.

Chapter 12

SECTION 12.11.2.1 AS FOLLOWS:

12.11.2.1 Wall Anchorage Forces

Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. (12.11-1) is permitted to be multiplied by the factor $(1+2z/h)/3$, where z is the height of the anchor above the base of the structure and h is the height of the roof above the base; however, F_p shall not be less than required by Section ~~12.11.2~~ 12.11.1 with a minimum anchorage force of $F_p = 0.2W_p$.

SECTION 12.13.9.2 AS FOLLOWS:

12.13.9.2 Shallow Foundations

12.13.9.2.1.1 Foundation Ties. Individual footings shall be interconnected by ties in accordance with Section 12.13.8.2 and the additional requirements of this section. ~~The ties shall be designed to accommodate the differential settlements between adjacent footings per Section 12.13.9.2, item b.~~ Reinforced concrete sections shall be detailed in accordance with Sections 18.6.2.1 and 18.6.4 of ACI 318.

SECTION 12.13.9.3 AS FOLLOWS:

12.13.9.3 Deep Foundations

12.13.9.3.1 Downdrag Design of piles shall incorporate the effects of downdrag caused by liquefaction. For geotechnical design, the liquefaction-induced downdrag shall be determined as the downward skin friction on the pile within and above the liquefied zone(s). The net geotechnical ultimate capacity of the pile shall be the ultimate geotechnical capacity of the pile below the liquefiable layer(s) reduced by the downdrag load. For structural design, downdrag load induced by liquefaction shall be treated and factored as a seismic load, although it need not be considered concurrently with axial loads resulting from inertial response of the structure, determined according to Section 12.4.~~and factored accordingly.~~

Chapter 15

SECTION 15.5.3.1 AS FOLLOWS:

15.5.3.1 Steel Storage Racks.

Steel storage racks supported at or below grade shall be designed in accordance with ANSI/RMI MH 16.1, ~~and~~ its force and displacement requirements, and the seismic design ground motion values determined according to Section 11.4, except as follows:

Chapter 21

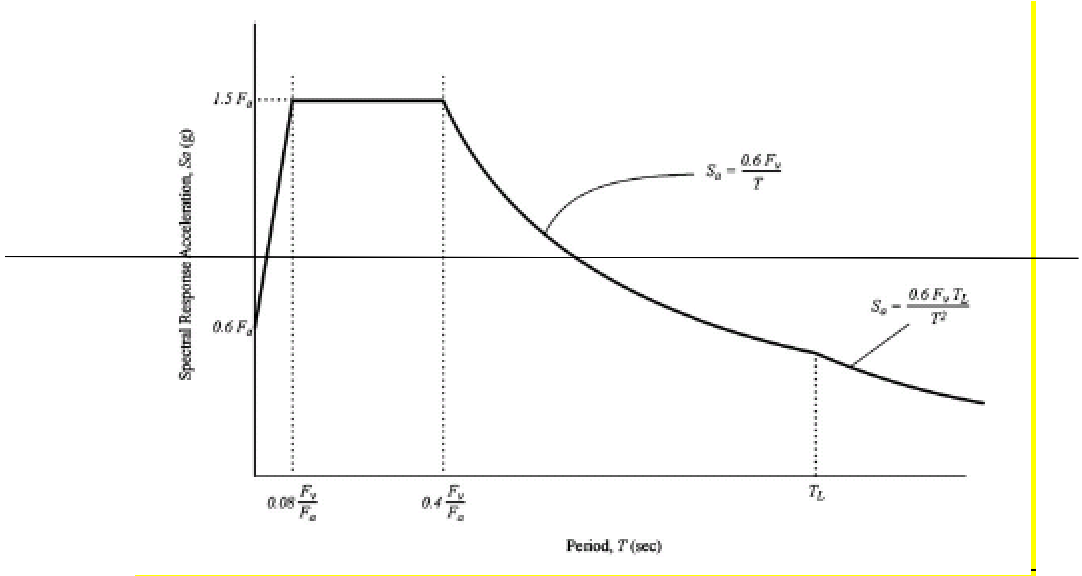
SECTION 21.2.2 AS FOLLOWS:

21.2.2 Deterministic (MCE_R) Ground Motions.

The deterministic spectral response acceleration at each period shall be calculated as an 84th-percentile 5% damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. If the largest spectral response acceleration of the resulting deterministic ground motion response spectrum is less than $1.5F_a$, then this response spectrum shall be scaled by a single factor such that the maximum response spectral acceleration equals $1.5F_a$. For Site Classes A, B, C and D, Fa shall be determined using Table 11.4.1, with the value of S_s taken as 1.5; for Site Class E, F_a shall be taken as 1.0. ~~The ordinates of the deterministic ground motion response spectrum shall not be taken as lower than the corresponding ordinates of the response spectrum determined in accordance with Fig. 21.2-1. For the purposes of calculating the ordinates.~~

EXCEPTION: The deterministic ground motion response spectrum need not be calculated when the largest spectral response acceleration of the probabilistic ground motion response spectrum of 21.2.1 is less than $1.2F_a$.

~~(i) for Site Classes A, B or C: F_a and F_v shall be determined using Tables 11.4-1 and 11.4-2, with the value of S_S taken as 1.5 and the value of S_1 taken as 0.6;~~
~~(ii) for Site Class D: F_a shall be taken as 1.0, and F_v shall be taken as 2.5; and~~
~~(iii)for Site Classes E and F: F_a shall be taken as 1.0, and F_v shall be taken as 4.0.~~



~~**FIGURE 21.2-1 Deterministic Lower Limit on MCE$_R$ Response Spectrum**~~

SECTION 21.2.3 AS FOLLOWS:

21.2.3 Site-Specific MCE$_R$.

The site-specific MCE$_R$ spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2.

EXCEPTION: The site-specific MCE$_R$ ground motion response spectrum shall be taken as the probabilistic ground motion response spectrum of 21.2.1 when the largest spectral response acceleration of the probabilistic ground motion response spectrum of 21.2.1 is less than 1.2F_a. For Site Classes A, B, C and D, F_a shall be determined using Table 11.4.1, with the value of S_S taken as 1.5; for Site Class E, F a shall be taken as 1.0.

The site-specific MCE$_R$ spectral response acceleration at any period shall not be taken less than 150% of the site-specific design response spectrum determined in accordance with 21.3.

SECTION 21.3 AS FOLLOWS:

21.3 DESIGN RESPONSE SPECTRUM

The design spectral response acceleration at any period shall be determined from Eq. (21.3-1):

$$S_a = \frac{2}{3} S_{aM} \tag{21.3-1}$$

where S_{aM} is the MCE spectral response acceleration obtained from Section 21.1 or 21.2.

The design spectral response acceleration at any period shall not be taken as less than 80% of S_a determined in accordance with Section 11.4.6, where F_a and F_v are determined as follows:

(i) for Site Class A, B, and C: F_a and F_v are determined using Tables 11.4-1 and 11.4-2, respectively;

(ii) for Site Class D: F_a is determined using Table 11.4-1, and F_v is taken as 2.4 for $S_1 < 0.2$ or 2.5 for $S_1 \geq 0.2$; and

(iii) for Site Class E: F_a is determined using Table 11.4-1 for $S_S < 1.0$ or taken as 1.0 for $S_S \geq 1.0$, and F_v is taken as 4.2 for $S_1 \leq 0.1$ or 4.0 for $S_1 > 0.1$.

For sites classified as Site Class F requiring site-specific analysis in accordance with Section 11.4.~~7~~8, the design spectral response acceleration at any period shall not be less than 80% of S_a determined for Site Class E. ~~in accordance with Section 11.4.5~~.

EXCEPTION: Where a different site class can be justified using the site-specific classification procedures in accordance with Section 20.3.3, a lower limit of 80% of S_a for the justified site class shall be permitted to be used.

SECTION C2.3.2 AS FOLLOWS:

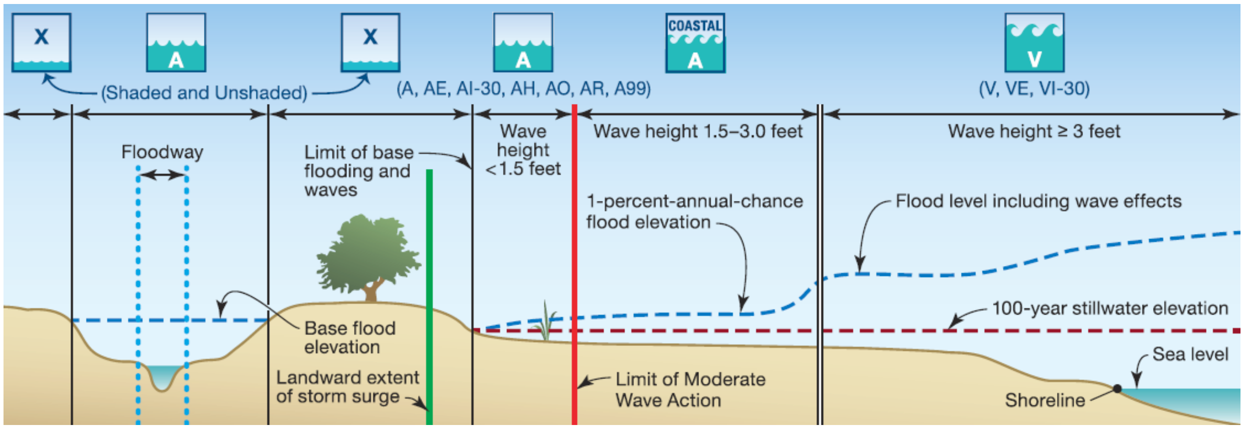
C2.3.2 Load Combinations Including Flood Load.

The nominal flood load, F_a, is based on the 100-year flood (Section 5.1). 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 caused by steady overland flow, and hydrodynamic pressures caused by waves, as specified in Section 5.4.

The flood load criteria were derived from an analysis of hurricane-generated storm tides produced along the United States East and Gulf coasts (Mehta et al. 1998), 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 5.3 and 5.4. Load factors for these loads were then obtained using established reliability methods (Ellingwood et al. 1982; Galambos et al. 1982) 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, extratropical 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 stillwater flood depths ranging from approximately 4 to 9 ft (1.2–2.7 m) (average stillwater flood depth of approximately 6 ft (1.8 m)) 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 (1.2 m) equal 2.8 (Mehta et al. 1998). 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.

The variability in hydrostatic loads under flood conditions is small when compared with the variability in wave loads and hydrodynamic loads from overland flooding. For coastal flood situations where overland waves are small (in the Coastal A zone and A zone), application of the load factor of 2.0 to below-grade flood-induced (hydrostatic) loads is too conservative, and the 1.6 load factor specified for H loads in Section 2.3.1 is more appropriate. Fig. C2.3-1 illustrates the flood zones and load factors for Fa and H for flood water above grade and below grade.



	Flood Zone					
	X Zone	A Zone	X Zone	A Zone	Coastal A Zone	V Zone
LRFD Load Factor on F_a for Structural Elements Above Grade	N/A	1.0	N/A	1.0	2.0	2.0

	Flood Zone					
	X Zone	A Zone	X Zone	A Zone	Coastal A Zone	V Zone
LRFD Load Factor on H (hydrostatic uplift and lateral pressure due to groundwater) for Structural Elements Below Grade	1.6	1.6	1.6	1.6	1.6	1.6

	Flood Zone					
	X Zone	A Zone	X Zone	A Zone	Coastal A Zone	V Zone
ASD Load Factor on F_a for Structural Elements Above Grade	N/A	0.75	N/A	0.75	1.5	1.5

	Flood Zone					
	X Zone	A Zone	X Zone	A Zone	Coastal A Zone	V Zone
ASD Load Factor on H (hydrostatic uplift and lateral pressure due to groundwater) for Structural Elements Below Grade	1.0	1.0	1.0	1.0	1.0	1.0

FIGURE C2.3-1 Illustration of flood zones and ASCE 7-16 load factors for F_a and H for (reading from right to left in figure): coastal flood zones (V Zone, Coastal A Zone, and A Zone), areas outside the 100-yr floodplain (X Zone Shaded are areas inside the 500-yr flood plain and X Zone Unshaded are areas outside the 500-yr flood plain), and riverine flood zone (A Zone).

Chapter 6 Commentary

SECTION C6.6.1 AS FOLLOWS:

C6.6.1 Maximum Inundation Depth and Flow Velocities Based on Runup.

The Energy Grade Line Analysis stepwise procedure consists of the following steps:

1. Obtain the runup and inundation limit values from the Tsunami Design Zone Map generated by the ASCE Tsunami Design Geodatabase.
2. Approximate the principal topographic transect by a series of $x - z$ grid coordinates defining a series of segmented slopes, in which x is the distance inland from the shoreline to the point and z is the ground elevation of the point. The horizontal spacing of transect points should be less than 100 ft (30.5 m), and the transect elevations should be obtained from a topographic Digital Elevation Model (DEM) of at least 33-ft (10-m) resolution.
3. Compute the topographic slope, ϕ_i, of each segment as the ratio of the increments of elevation and distance from point to point in the direction of the incoming flow.
4. Obtain the Manning's coefficient, n, from Table 6.6-1 for each segment based on terrain analysis.
5. Compute the Froude number at each point on the transect using Eq. (6.6-3).
6. Start at the point of runup with a boundary condition of $E_R = 0$ at the point of runup.

i. Per Section 6.6.1, where the maximum topographic elevation along the topographic transect between the shoreline and the inundation limit is greater than the runup elevation, use a runup elevation that has at least 100% of the maximum topographic elevation along the topographic transect.*

7. Select a nominally small value of inundation depth [~ 0.1 ft (0.03 m)] h_r at the point of runup.
8. Calculate the hydraulic friction slope, s_i, using Eq. (6.6-2).
9. Compute the hydraulic head, E_i, from Eq. (6.6-1) at successive points toward the shoreline.
10. Calculate the inundation depth, h_i, from the hydraulic head, E_i.
11. Using the definition of Froude number, determine the velocity u. Check against the minimum flow velocity required by Section 6.6.1.
12. Repeat through the transect until the h and u are calculated at the site. These are used as the maximum inundation depth, h_{max}, and the maximum velocity, u_{max}, at the site.

*Where Inundation Depth Points are provided for completely overwashed areas in the ASCE Tsunami Design Geodatabase, where the tsunami flows over an island or peninsula into a second water body, the horizontal distance of the inundation limit shall be taken to be the length of the overwashed land. There are two modifications of Section 6.6.1 given for this case relating to the initial conditions of nonzero depth and velocity at the inland edge of the overwashed area and the Froude number profile that follows a linear interpolation with distance across the overwashed land. To complete the analysis, the inundation elevation profiles are adjusted so that the computed inundation depths are at least the depths specified at the Inundation Depth Points. The adjusted inundation elevation profile should not be lower than the topographic elevation transect. An example is given in Fig. C6.6-3.

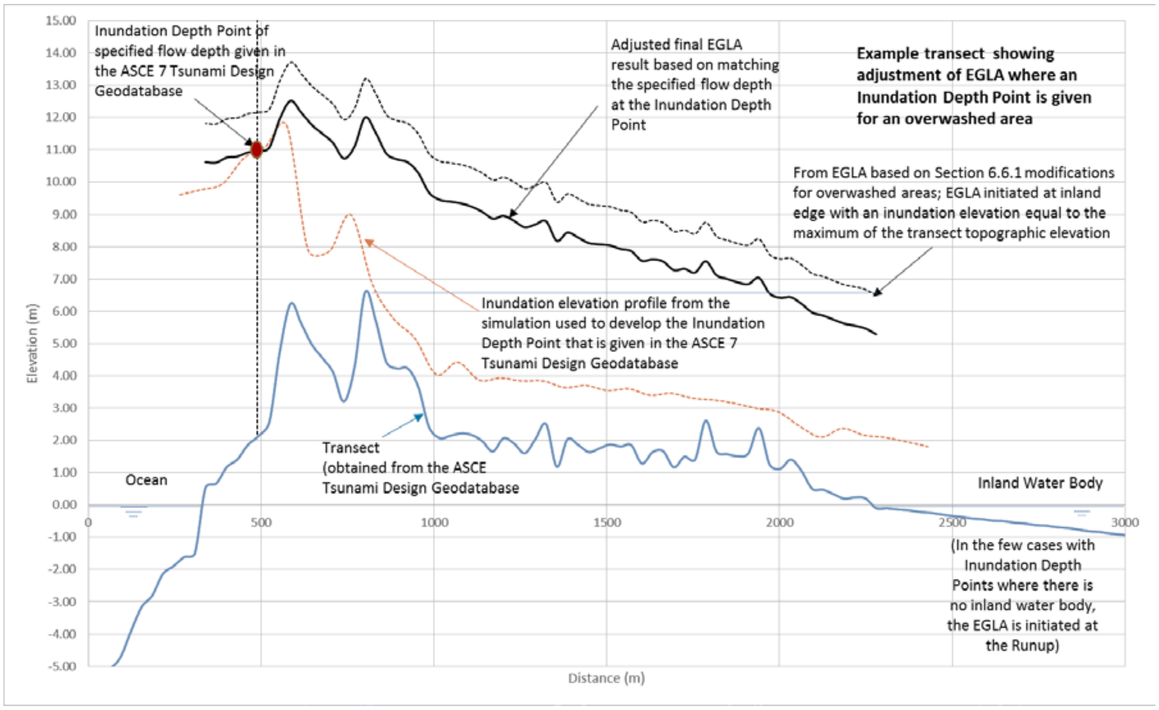


FIGURE C6.6-3 Example EGLA with adjustment where an Inundation Depth Point is specified in the ASCE Tsunami Design Geodatabase

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Chapter 12 Commentary

SECTION C12.13.9.3 AS FOLLOWS:

C12.13.9.3 Deep Foundations.

Pile foundations are intended to remain elastic under axial loadings, including those from gravity, seismic, and downdrag loads. Since geotechnical design is most frequently performed using allowable stress design (ASD) methods, and liquefaction-induced downdrag is assessed at an ultimate level, the requirements state that the downdrag is considered as a reduction in the ultimate capacity. Since structural design is most frequently performed using load and resistance factor design (LRFD) methods, and the downdrag is considered as a load for the pile structure to resist, the requirements clarify that the downdrag is considered as a seismic axial load, to which a factor of 1.0 would be applied for design.

Although downdrag load is to be factored as a seismic load, it is not intended to be considered concurrently with seismic loads because of inertial response of the structure. Significant excess pore pressure dissipation and settlement occurs after the cessation of shaking. This effect has been borne out in the laboratory, as documented by Wilson et al. (1997).

REFERENCES AS FOLLOWS:

REFERENCES

Wilson, D.W., R.W. Boulanger, B.L. Kutter, A. Abghari, (1997) "Aspects of dynamic centrifuge testing of soil-pile-superstructure interaction", ASCE Geotechnical Special Publication No. 64, pp.47-63.

Chapter 21 Commentary

SECTION C21.2.2 AS FOLLOWS:

C21.2.2 Deterministic ($\mathrm{MCE_R}$) Ground Motions.

Deterministic ground motions are to be based on characteristic earthquakes on all known active faults in a region. The magnitude of a characteristic earthquake on a given fault should be a best estimate of the maximum magnitude capable for that fault but not less than the largest magnitude that has occurred historically on the fault. The maximum magnitude should be estimated considering all seismic-geologic evidence for the fault, including fault length and paleoseismic observations. For faults characterized as having more than a single segment, the potential for rupture of multiple segments in a single earthquake should be considered in assessing the characteristic maximum magnitude for the fault.

For consistency, the same attenuation equations and ground motion variability used in the PSHA should be used in the deterministic seismic hazard analysis (DSHA). Adjustments for directivity and/or directional effects should also be made, when appropriate. In some cases, ground motion simulation methods may be appropriate for the estimation of long-period motions at sites in deep sedimentary basins or from great ($M \geq 8$) or giant ($M \geq 9$) earthquakes, for which recorded ground motion data are lacking.

When the maximum ordinate of the deterministic ($\mathrm{MCE_R}$) ground motion response spectrum is less than 1.5F_a, it is scaled up to 1.5F_a to put a lower limit or floor on the deterministic ground motions. A single factor is used to maintain the shape of the response spectrum. The intent of the exception defining site-specific $\mathrm{MCE_R}$ ground motions solely in terms of probabilistic $\mathrm{MCE_R}$ ground motions (i.e., when peak $\mathrm{MCE_R}$ response spectral accelerations are less than 1.2F_a) is to preclude unnecessary calculation of deterministic $\mathrm{MCE_R}$ ground motions.

~~Values of the site coefficients (F_a and F_v) for setting the deterministic ($\mathrm{MCE_R}$) ground motion floor are introduced to incorporate both site amplification and spectrum shape adjustment as described in the research study "Investigation of an Identified Short-Coming in the Seismic Design Procedures of ASCE 7-16 and Development of Recommended Improvements for ASCE 7-16" (Kircher 2015). This study found that the shapes of the response spectra of ground motions were not accurately represented by the shape of the design response spectrum of Figure 11.4-1 for the following site conditions and ground motion intensities: (1) Site Class D where values of $S_1 \geq 0.2$; and (2) Site Class E where values of $S_S \geq 1.0$ and/or $S_1 \geq 0.2$. An adjustment of the corresponding values of F_a and F_v was required to account for this difference in~~

~~spectrum shape, which was causing the design response spectrum to underestimate long-period motions. Two options were considered to address this shortcoming. For the first option, the subject study developed values of new "spectrum shape adjustment" factors (C_a and C_v) that could be used with site factors (F_a and F_v) to develop appropriate values of design ground motions (S_{DS} and S_{D1}). The second option, ultimately adopted by ASCE 7-16, circumvents the need for these new factors by requiring site-specific analysis for Site Class D site conditions where values of $S_1 \geq 0.2$, and for Site Class E site conditions where values of $S_S \geq 1.0$ and/or $S_1 \geq 0.2$ (i.e., new requirements of Section 11.4.8 of ASCE 7-16). The spectrum shape adjustment factors developed by the subject study for Option 1 provide the basis for the values of site coefficients (F_a and F_v) proposed for Section 21.2.2 and Section 21.3 that incorporate both site amplification and adjustment for spectrum shape. Specifically, the proposed value of $F_v = 2.5$ for Site Class D is based on the product of 1.7 (Site Class D amplification at $S_1 = 0.6$, without spectrum shape adjustment) and 1.5 (spectrum shape adjustment factor); the proposed value of $F_v = 4.0$ is based on the product of 2.0 (Site Class E amplification at $S_1 = 0.6$ without spectrum shape adjustment) and 2.0 (spectrum shape adjustment factor), where values of spectrum shape adjustment are taken from Section 6.2.2 (Table 11.4-4) of the subject study. The proposed value of $F_a = 1.0$ is based on the product of 0.8 (Site Class E amplification at $S_S = 1.5$ without spectrum shape adjustment) and 1.25 (spectrum shape adjustment factor), where the value of the spectrum shape adjustment is taken from Section 6.2.2 (Table 11.4-3) of the subject study. Site amplification adjusted for spectrum shape effects is approximately independent of ground motion intensity and, for simplicity, the proposed values of site factors adjusted for spectrum shape are assumed to be valid for all ground motion intensities.~~

SECTION C21.2.3 AS FOLLOWS:

C21.2.3 Site-Specific $\mathrm{MCE_R}$.

Because of the deterministic lower limit on the $\mathrm{MCE_R}$ spectrum (Fig. 21.2-1), the site-specific $\mathrm{MCE_R}$ ground motion is equal to the corresponding risk-targeted probabilistic ground motion wherever it is less than the deterministic limit (e.g., $1.5g$ and $0.6g$ for 0.2 and 1.0 s, respectively, and Site Class B). Where the probabilistic ground motions are greater than the lower limits, the deterministic ground motions sometimes govern, but only if they are less than their probabilistic counterparts. On the $\mathrm{MCE_R}$ ground motion maps in ASCE/SEI 7-10, the deterministic ground motions govern mainly near major faults in California (like the San Andreas) and Nevada. The deterministic ground motions that govern are as small as 40% of their probabilistic counterparts.

The exception defining site-specific $\mathrm{MCE_R}$ ground motions solely in terms of probabilistic $\mathrm{MCE_R}$ ground motions (i.e., when peak $\mathrm{MCE_R}$ probabilistic ground motions are less than $1.2F_a$) precludes unnecessary calculation of deterministic MCE_R ground motions. Probabilistic MCE_R ground motions are presumed to govern at all periods where the peak probabilistic MCE_R response spectral acceleration (i.e., < $1.2F_a$) is less than 80% of peak deterministic (MCE_R) response spectral acceleration (i.e., ≥ $1.5F_a$).

The requirement that the site-specific MCE_R response spectrum not be less than 150% of the site-specific design response spectrum of Section 21.3 effectively applies the 80% limits of Section 21.3 to the site-specific MCE_R response spectrum (as well as the site-specific design response spectrum).

SECTION C21.3 AS FOLLOWS:

C21.3 DESIGN RESPONSE SPECTRUM

Eighty percent of the design response spectrum determined in accordance with Section 11.4.6 was established as the lower limit to prevent the possibility of site-specific studies generating unreasonably low ground motions from potential misapplication of site-specific procedures or misinterpretation or mistakes in the quantification of the basic inputs to these procedures. Even if site-specific studies were correctly performed and resulted in ground motion response spectra less than the 80% lower limit, the uncertainty in the seismic potential and ground motion attenuation across the United States was recognized in setting this limit. Under these circumstances, the allowance of up to a 20% reduction in the design response spectrum based on site-specific studies was considered reasonable.

~~As described in Section 21.2.2, values of the site coefficients (F_a and F_v) for setting the deterministic (MCE_R) ground motion floor are introduced to incorporate both site amplification and spectrum shape adjustment.~~

Values of the site coefficients (F_a and F_v) for setting the 80% lower limit are introduced to incorporate both site amplification and spectrum shape adjustment, as described in the research study "Investigation of an Identified Short-Coming in the Seismic Design Procedures of ASCE 7-16 and Development of Recommended Improvements for ASCE 7-16" (Kircher 2015). This study found that the shapes of the response spectra of ground motions were not accurately represented by the shape of the design response spectrum of Fig. 11.4-1 for the following site conditions and ground-motion intensities: (1) Site Class D, where values of $S_1 \geq 0.2$, and (2) Site Class E, where values of $S_S \geq 1.0$ and/or $S_1 \geq 0.2$. An adjustment of the corresponding values of F_a and F_v was required to account for this difference in spectrum shape, which was causing the design response spectrum to underestimate long period motions. Two options were considered to address this shortcoming. For the first option, the subject study developed values of new "spectrum shape adjustment" factors (C_a and C_v) that could be used with site factors (F_a and F_v) to develop appropriate values of design ground motions (S_{DS} and S_{D1}). The second option, ultimately adopted by ASCE 7-16, circumvents the need for these new factors by requiring site-specific analysis for Site Class D site conditions, where values of $S_1 \geq 0.2$, and for Site Class E site conditions, where values of $S_S \geq 1.0$ and/or $S_1 \geq 0.2$ (i.e., new requirements of Section 11.4.8 of ASCE 7-16). The spectrum shape adjustment factors developed by the subject study for Option 1 provide the basis for the values of site coefficients (F_a and F_v) of Section 21.3 that incorporate both site amplification and adjustment for spectrum shape. Specifically, the value of F_v = 2.5 for Site Class D is based on the product of 1.7 (Site Class D amplification at S_1 = 0.6, without spectrum shape adjustment) and 1.5 (spectrum shape adjustment factor); the value of F_v = 4.0 for Site Class E is based on the product of 2.0 (Site Class E amplification at S_1 = 0.6 without spectrum shape adjustment) and 2.0 (spectrum shape adjustment factor), where values of spectrum shape adjustment are taken from Section 6.2.2 (Table 11.4-4) of the subject study. The value of F_a = 1.0 for Site Class E is based on the product of 0.8 (Site Class E amplification at S_S = 1.5 without spectrum shape adjustment) and 1.25 (spectrum shape adjustment factor), where the value of the spectrum shape adjustment is taken from Section 6.2.2 (Table 11.4-3) of the subject study. Site amplification adjusted for spectrum shape effects is approximately independent of ground motion intensity and, for simplicity, the proposed values of site factors adjusted for spectrum shape are assumed to be valid for all ground motion intensities.

Although the 80% lower limit is reasonable for sites not classified as Site Class F, an exception has been introduced at the end of this section to permit a site class other than E to be used in establishing this limit when a site is classified as F. This revision eliminates the possibility of an overly conservative design spectrum on sites that would normally be classified as Site Class C or D.

Standard 7-16
Minimum Design Loads and Associated Criteria for Buildings and Other Structures

SUPPLEMENT 2

Effective: October 14, 2021

This document contains errata to the above title, which is posted on the ASCE Library at https://doi.org/10.1061/97807844xxxxx

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CHAPTER 12: Seismic Design Requirements for Building Structures

SECTION 12.9.1.5 AS FOLLOWS:

12.9.1.5 Horizontal Shear Distribution. The distribution of horizontal shear shall be in accordance with Section 12.8.4.~~, except that amplification of torsion in accordance with Section 12.8.4.3 is not required where accidental torsion effects are included in the dynamic analysis model.~~ The effects of accidental torsion shall be accounted for by applying a static accidental torsional moment (M_{ta}) determined in accordance with Section 12.8.4.2 and 12.8.4.3 to the mathematical model, and combining the results with the scaled design values computed in accordance with Section 12.9.1.4.

EXCEPTION: For structures without a horizontal irregularity Type 1b, the effects of accidental torsion may be included in the dynamic analysis model in lieu of applying M_{ta}. When the effects of accidental torsion are included in the dynamic analysis model, amplification of torsion in accordance with Section 12.8.4.3 is not required.

SECTION 16.4.2.1 AS FOLLOWS:

16.4.2.1 Force-Controlled Actions. Force-controlled actions shall satisfy Eqs. (16.4-1) and (16.4-2):

$$\cancel{\gamma I_e(Q_u - Q_{ns}) + Q_{ns} \leq Q_e} \qquad \cancel{\textbf{(16.4.1)}}$$

$$(1.2 + 0.12S_{MS})D + 0.5L + 1.3I_e(Q_u - Q_{ns}) \leq \emptyset BR_n \qquad \textbf{(16.4-1)}$$

$$(0.9 - 0.12S_{MS})D + 1.3I_e(Q_u - Q_{ns}) \leq \emptyset BR_n \qquad \textbf{(16.4-2)}$$

where *D* and *L* are as defined in Section 16.3.2, S_{MS} is the site-adjusted Maximum Considered Earthquake Spectral Acceleration at a period of 0.2 seconds; I_e is the Importance Factor prescribed in Section 1.5.1; Q_{ns} is the demand caused by loads other than seismic; ~~Q_e is the expected component strength; and γ is the load factor obtained from Table 16.4-1.~~ R_n is the nominal strength specified by the applicable material standard. The resistance factor ϕ for critical elements shall be taken as the value specified by the applicable material standard. The resistance factor ϕ for Ordinary elements shall be taken as 0.9. The resistance factor ϕ for Noncritical elements shall be taken as 1.0. *B* is a factor to account for differences between expected strength R_{ne} and nominal resistance R_n. It is permitted to assign *B* a value of 1.0, or, alternatively, *B* can be taken as $0.9R_{ne}/R_n$, where R_{ne} is the expected strength of the element. Where an industry standard referenced in Chapter 14 defines expected strength, that value shall be used. Where this is not defined, it shall be permitted to calculate expected strength as the nominal strength defined in industry standards, except that expected material properties as defined in ASCE 41 shall be used in lieu of specified values.

EXCEPTIONS:

1. **Noncritical force-controlled actions that are modeled, including consideration of strength loss effects, need not satisfy Eqs. (16.4-1) or (16.4-2).**
2. **~~For f~~Force-controlled actions limited by formation of a yield mechanism, other than shear in structural walls, ~~and columns,~~ ~~the nominal element strength need not exceed the effects of gravity load plus the force demand determined by plastic mechanism analysis, where the analysis is based on expected material properties.~~ the action need only satisfy Eqs. (16.4-3) and (16.4-4):**

   $$(1.2 + 0.12S_{MS})D + 0.5L + E_{mc} \leq \emptyset BR_n \qquad \textbf{(16.4-3)}$$

   $$(0.9 - 0.12S_{MS})D + E_{mc} \leq \emptyset BR_n \qquad \textbf{(16.4-4)}$$

   Where E_{mc} is the capacity-limited earthquake effect associated with developing the plastic capacity of yielding components, determined in accordance with the applicable material standard, or alternatively, determined by rational analysis considering expected material properties including strain hardening effects where applicable.
3. **Where response to vertical earthquake shaking is directly included in the analysis, the first term in equations 16.4-1 through 16.4-4 can be taken as 1.2D (16.4-1 and 16.4-3) or 0.9D (16.4-2 and 16.4-4).**

~~Table 16.4-1 Load Fractor for Force Controlled Behaviors~~

~~Action Type~~	~~γ~~
~~Crticial~~	~~2.0~~
~~Ordinary~~	~~1.5~~

~~**Noncritical 1.0**~~

CHAPTER 12: Commentary

SECTION C12.9.1.5 AS FOLLOWS:

C12.9.1.5 Horizontal Shear Distribution. Torsion effects in accordance with Section 12.8.4 must be included in the modal response spectrum analysis (MRSA) as specified in Section 12.9 by requiring use of the procedures in Section 12.8 for the determination of the seismic base shear, V.

Prior to ASCE 7-16 Supplement 2, there were two permissible approaches to account for the effects of accidental torsion for all structures. The first approach accounted for accidental torsion by applying a static accidental torsional moment to the MRSA results, and the second approach directly assessed the dynamic effects of accidental torsion by physically offsetting the center of mass in the three-dimensional analysis model. The ATC-123 (FEMA P-2012 Assessing Seismic Performance of Buildings with Configuration Irregularities - Calibrating Current Standards and Practices, September 2018) project found that the second method of physically offsetting the mass could actually de-amplify the effects of accidental torsion for some buildings with torsional first modes, resulting in designs that have significantly less collapse resistance than designs proportioned by applying a static accidental torsional moment to the MRSA results. This occurs because the torsional response can become un-coupled from the translational response in extremely torsionally irregular structures, and Section 12.9 only requires translational response to be considered and scaled in a MRSA. This potential de-amplification effect was particularly pronounced for buildings with extreme torsional irregularities. Consequently, the second approach is now only permissible for structures without an extreme torsional irregularity.

The ~~first~~ permissible approach that can be used for all structures follows the static procedure discussed in Section C12.8.4.2, where the total seismic lateral forces obtained from MRSA—using the computed locations of the centers of mass and rigidity—are statically applied at an artificial point offset from the center of mass to compute the accidental torsional moments. Most computer programs can automate this procedure for three-dimensional analysis. Alternatively, the torsional moments can be statically applied as separate load cases and added to the results obtained from MRSA.

Because this approach is a static approximation, amplification of the accidental torsion in accordance with Section 12.8.4.3 is required. MRSA results in a single, positive response, thus inhibiting direct assessment of torsional response. One method to circumvent this problem is to determine the maximum and average displacements for each mode participating in the direction being considered and then apply modal combination rules (primarily the CQC method) to obtain the total displacements used to check torsional irregularity and compute the amplification factor, A_x. The analyst should be attentive about how accidental torsion is included for individual modal responses.

The alternate approach is only permissible for structures without an extreme torsional irregularity. This approach, which applies primarily to three-dimensional analysis, is to modify the dynamic characteristics of the structure so that dynamic amplification of the accidental torsion is directly considered. This modification can be done, for example, by either reassigning the lumped mass for each floor and roof (rigid diaphragm) to alternate points offset from the initially calculated center of mass and modifying the mass moment of inertia, or physically relocating the initially calculated center of mass on each floor and roof by modifying the horizontal mass distribution (typically presumed to be uniformly distributed). This approach increases the computational demand significantly because all possible configurations would have to be analyzed, primarily two additional analyses for each principal axis of the structure. The advantage of this approach is that the dynamic effects of direct loading and accidental torsion are assessed automatically. Practical disadvantages are the increased bookkeeping required to track multiple analyses and the cumbersome calculations of the mass properties.

Where this "dynamic" approach is used in structures without an extreme torsional irregularity, amplification of the accidental torsion in accordance with Section 12.8.4.3 is not required because repositioning the center of mass increases the coupling between the torsional and lateral modal responses, directly capturing the amplification of the accidental torsion.

Most computer programs that include accidental torsion in a MRSA do so statically (~~first~~ permissible approach discussed above) and do not physically shift the center of mass. The designer should be aware of the methodology used for consideration of accidental torsion in the selected computer program.

FEMA. (2018). "Assessing Seismic Performance of Buildings with Configuration Irregularities - Calibrating Current Standards and Practices." FEMA P-2012, Applied Technology Council, FEMA, Washington, DC.

CHAPTER 16: Commentary

SECTION C16.4.2.1 AS FOLLOWS:

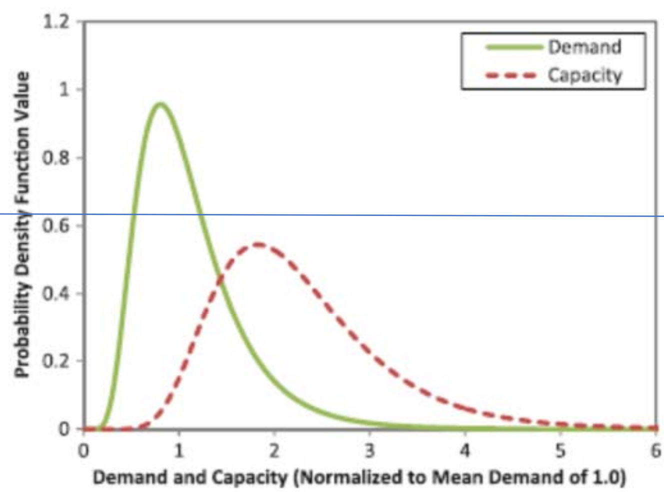
C16.4.2.1 Force-Controlled Actions. The application of the load combinations and resistance factors in Eqns 16.4-1, 16-4-2, 16.4-3, and 16.4-4 are limited to load effects determined using NLRHA for structures subject to an MCE_R level event. These load and resistance factors should not be used for structural design with the design earthquake effects, E determined in accordance with Chapter 12 of the standard. When evaluating earthquake effects in accordance with Chapter 12 of the standard, the load combinations of Chapter 2 apply.

The acceptance criteria for force-controlled actions under MCE_R demands (Eqns 16.4-1 and 16.4-2) follow the same framework ~~established by the PEER TBI guidelines (Bozorgnia et al. 2009), shown in Eq. (C16.4-1):~~

$$\cancel{\lambda F_u \leq \emptyset F_{n,e}} \qquad \cancel{\text{(C16.4-1)}}$$

~~where λ is a calibration parameter, Fu is the mean demand for the response parameter of interest, φ is the strength reduction factor from a material standard, and Fn,e is the nominal strength computed from a material standard considering expected material properties.~~ that underlies the load combinations contained in Chapter 2 of the standard, except that both the load factors and the capacity (resistance) factors have been adjusted for consistency with the NLRHA approach and to maintain compatibility with the target reliability in Table 1.3-2. The load factor of $1.3I_e$ on seismic demands was determined assuming a demand dispersion associated with record-to-record variability of 0.3, based on values observed for several real buildings and additional modeling uncertainty of 0.2, which was selected based on engineering judgment consistent with approaches used in FEMA P695. Based on data presented in National Bureau of Standard Special Publication SP 577, the typical bias in resistance, i.e. the ratio of the expected strength of an element, to the nominal strength, was taken as 1.1, while a 0.15 coefficient of variation on resistance was assumed. The materials standards have resistance factors that generally vary from values of approximately 0.7 to 0.9. An average value of 0.85 was used in computing the 1.3 load factor on seismic demand.

Exception 2 to the Section permits the use of an alternative set of load combinations (16.4-3 and 16.4-4) to evaluate force-controlled actions when demands are limited by the development of a plastic mechanism. For example, shear in a column in a moment frame cannot exceed the sum of the plastic moments at the ends of the column, divided by the free height of the column. A load factor of unity is used on the capacity-limited earthquake demand in this case, recognizing the very low probability that demand can exceed the computed value and also for consistency with similar criteria in ACI 318 and AISC 341.



~~FIGURE C16.4-2 Illustration of Component Capacity and Demand Lognormal Distributions (Normalized to a Mean Capacity of 1.0); the Mean Component Capacity Is Calibrated to Achieve~~
~~$P[C|MCE_R] = 10\%$~~

~~To determine appropriate values of λ, we begin with the collapse probability goals of Table 1.3-2 (for Risk Categories I and II) for MCER motions. These collapse probability goals include a 10% chance of a total or partial structural collapse and a 25% chance of a failure that could result in endangerment of individual lives. For the assessment of collapse, we then make the somewhat conservative assumption that the failure of a single critical force-controlled component would result in a total or partial structural collapse of the building.~~

~~Focusing first on the goal of a 10% chance of a total or partial structural collapse, we assume that the component force demand and component capacity both follow a lognormal distribution and that the estimate of $F_{n,e}$ represents the true expected strength of the component. We then calibrate the λ value required to achieve the 10% collapse probability goal. This value is depicted in Fig. C16.4-2, which shows the lognormal distributions of component capacity and component demand.~~

~~The calibration process is highly dependent on the uncertain-ties in component demand and capacity. Table C16.4-3a shows typical uncertainties in force demand for analyses at the MCER ground motion level for both the general case and the case where the response parameter is limited by a well-defined yield mechanism. Table C16.4-3b shows typical uncertainty values for the component capacity. The values are based on reference materials, as well as the collective experience and professional judgment of the development team.~~

~~In the calibration process, the λ and φ values both directly affect the required component strength. Therefore, the calibration is completed to determine the required value of λ/φ needed to fulfill the 10% collapse safety objective. This calibration is done by assuming a value of λ/φ, convolving the lognormal distributions of demand and capacity and iteratively determining the capacity required to meet the 10% collapse safety objective by adjusting λ/φ~~

~~Table C16.4-4 reports the final λ/φ values that come from such integration.~~

~~It should be clearly stated that this approach of calibrating the λ/φ ratio means that the final acceptance criterion is independent of the φ value specified by a material standard. If it is desirable for the acceptance criteria to be partially dependent on the value of φ, then the uncertainty factors of Table C16.4-3b would need to be made dependent on the φ value in some manner.~~

~~Since the Table C16.4-4 values are similar, for simplicity the acceptance criterion is based on $\lambda/\varphi = 2.0$ for all cases, and a separate case for the existence of a well-defined mechanism is not included. Additionally, the strength term is defined slightly differently. For Risk Categories III and IV, this full calculation was redone using the lower collapse probability goals of 6% and 3%, respectively, and it was found that scaling the force demands by I_e sufficiently achieves these lower collapse probability goals.~~

~~This statistical calculation was then repeated for the goal of 25% chance of a failure that could result in endangerment of individual lives. This resulted in a required ratio of 1.5 for such force-controlled failure modes; deemed as "ordinary."~~

Force-controlled actions are deemed noncritical if the failure does not result in structural collapse or any meaningful endangerment to individual lives; this occurs in situations where gravity forces can reliably redistribute to an alternate load path and no failure will ensue. For noncritical force-controlled components, the acceptance criteria allow the use of ~~λ~~ ϕ = 1.0, consistent with the reduced reliability for such elements in Table 1.3-3..

~~Where an industry standard does not define expected strength expected (or mean) strength, Fe, is computed as follows. First, a standard strength-prediction equation is used from a material standard, using a strength reduction factor, φ, of 1.0; the expected material properties are also used in place of nominal material properties.~~ Typically, the nominal resistance values specified in the materials standards incorporate a moderate level of conservatism, on the order of 10% to 15% associated with material strength that exceeds specified strength, and also intentional conservatism associated with fitting design equations to research data. In some cases, ~~this estimate of strength (Fn,e) may still be conservative in comparison with the mean expected strength shown by experimental tests (Fe) caused by~~ inherent conservatism in the strength equations adopted by the materials standards can be substantially larger than this. If such conservatism exists, the ~~Fn,e~~ value of R_n, which is a calculated capacity that is most closely represented by $V_{ne,\,ACI}$ in Fig. C16.4-3, may be multiplied by a ~~"component reserve strength factor"~~ bias factor, B, greater than 1.0, to account for empirical capacity (R_{ne}). ~~to produce the estimate of the mean expected strength (Fe)~~. This process is illustrated in Fig. C16.4-3, which shows the ~~Fe/Fn,e~~ R_{ne}/R_n ratios for the shear strengths from test data of reinforced concrete shear walls (Wallace et al. 2013). This figure shows that the ratio of ~~Fe/Fn,e~~ R_{ne}/R_n depends on the flexural ductility of the shear wall, demonstrating that ~~Fe~~ $B = 1.0$ ~~Fn,e~~ is appropriate for the shear strength in the zone of high flexural damage and ~~Fe = 1.5 Fn,e~~ a value of $B = 1.5$ is appropriate in zones with no flexural damage.

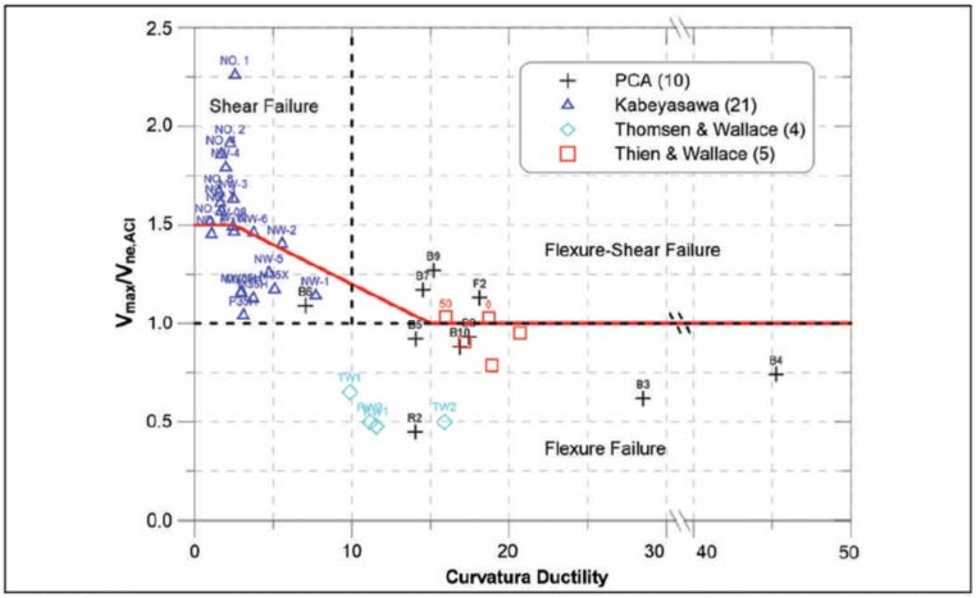


FIGURE C16.4-3 Expected Shear Strengths (in Terms of $F_e/F_{n,e}$) for Reinforced Concrete Shear Walls When Subjected to Various Levels of Flexural Ductility

Source: Courtesy of John Wallace.

~~For purposes of comparison, Eq. (C16.4-1) is comparable to the PEER TBI acceptance criteria (Bozorgnia et al. 2009) for the case that φ = 0.75 and Fe = 1.0 Fn,e.~~

~~The exception allows for use of the capacity design philosophy for force-controlled components that are "protected" by inelastic fuses, such that the force delivered to the force-controlled component is limited by the strength of the inelastic fuse.~~

~~**Table C16.4-3a Assumed Variability and Uncertainty Values for Component Force Demand**~~

Demand Dispersion (β_D)		
General	Well-Defined Mechanism	Variabilities and Uncertainties in the Force Demand
0.40	0.20	Record-to-record variability (for MCE_R ground motions)
0.20	0.20	Uncertainty from estimating force demands using structural model
0.13	0.06	Variability from estimating force demands from mean of only 11 ground motions
0.46	**0.29**	$\beta_{D-\mathrm{Total}}$

~~**Table C16.4-3b Assumed Variability and Uncertainty Values for Component Force Capacity**~~

Capacity Dispersion (β_C)		
General	Well-Defined Mechanism	Variabilities and Uncertainties in the Final As-Built Capacity of the Component
0.30	0.30	Typical variability in strength equation for $F_{n,e}$ (from available data)
0.10	0.10	Typical uncertainty in strength equation for $F_{n,e}$ (extrapolation beyond available data)
0.20	0.20	Uncertainty in as-built strength because of construction quality and possible errors
0.37	**0.37**	$\beta_{C-\mathrm{Total}}$

~~**Table C16.4-4 Required Ratios of λ/ϕ to Achieve the 10% Collapse Probability Objective**~~

Dispersion	Required Ratios of λ/ϕ
General	2.1
Well-Defined Mechanism	1.9

[REST OF SECTION REMAINS UNCHANGED]

Standard 7-16

Minimum Design Loads and Associated Criteria for Buildings and Other Structures

SUPPLEMENT 3

Effective: November 5, 2021

This document contains CHANGES to the above title, which is posted in the ASCE Library at 9780784414248

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Chapter 11

SECTION 11.4.8 and TABLES 11.4-1 and 11.4-2 AS FOLLOWS:

11.4.8 Site-Specific Ground Motion Procedures

A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless exempted in accordance with Section 20.3.1. A ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following:

~~1. seismically isolated structures and structures with damping systems on sites with~~ $\cancel{S_1}$ ~~greater than or equal to 0.6.~~

~~2. structures on Site Class E sites with~~ $\cancel{S_S}$ ~~greater than or equal to 1.0, and~~

1~~3~~. structures on Site Class D ~~and E~~ site~~s~~ with S_1 greater than or equal to 0.2.

EXCEPTION: A ground motion hazard analysis is not required where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1}

in this Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard.

2. structures on Site Class E sites with S_s greater than or equal to 1.0 or S_1 greater than or equal to 0.2.

EXCEPTION: A ground motion hazard analysis is not required:

1. where the equivalent lateral force procedure is used for design and the value of C_s is determined by Eq. (12.8-2) for all values of T, or
2. where (i) the value of S_{ai} is determined by Eq. (15.7-7) for all values of T_i and (ii) the value of the parameter S_{D1} is replaced with $1.5S_{D1}$ in Eq. (15.7-10) and Eq. (15.7-11).

~~**EXCEPTION**: A ground motion hazard analysis is not required for structures other than seismically isolated structures and structures with damping systems where:~~

~~1. Structures on Site Class E sites with S_S greater than or equal to 1.0, provided the site coefficient F_a is taken as equal to that of Site Class C.~~

~~2. Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_s$ or Eq. 12.8-4 for $T > T_L$.~~

~~3. Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_s and the equivalent static force procedure is used for design.~~

It shall be permitted to perform a site response analysis in accordance with Section 21.1 and/or a ground motion hazard analysis in accordance with Section 21.2 to determine ground motions for any structure.

When the procedures of either Section 21.1 or Section 21.2 are used, the design response spectrum shall be determined in accordance with Section 21.3, the design acceleration parameters shall be determined in accordance with Section 21.4 and, if required, the MCEG peak ground acceleration parameter shall be determined in accordance with Section 21.5.

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	1.2[a] ~~See Sec. 11.4.8~~	1.2[a] ~~See Sec. 11.48~~	1.2[a] ~~See Sec. 11.4.8~~

F	See Section 11.4.8

Note: Use linear interpolation for intermediate values of S_s.
[a]See requirements for site-specific ground motions in Section 11.4.8. These values of F_a shall only be used for calculation of T_s, determination of Seismic Design Category, linear interpolation for intermediate values of S_s, and when taking the exception under Item 2 within Section 11.4.8.

Table 11.4-2 Long-Period Site Coefficient, F_v

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
Site Class	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	3.3[a]	2.8[a]	2.4[a]	2.2[a]	2.0[a]
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use linear interpolation for intermediate values of S_1.
[a]See requirements for site-specific ground motions in Section 11.4.8. These values of F_v shall only be used for calculation of T_S~~.~~, determination of Seismic Design Category, linear interpolation for intermediate values of S_1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of S_{D1}.

Chapter 11 Commentary

SECTION C11.4.8 AS FOLLOWS:

Modify the last four paragraphs of the commentary to Section 11.4.8 to read as follows. The preceding paragraphs in the commentary to Section 11.4.8 remain unchanged.

In general, Section 11.4.8 requires site-specific hazard analysis for structures on Site Class E with values of S_S greater than or equal to 1.0 ~~g~~, and for structures on Site Class D or Site Class E for values of S_1 greater than or equal to 0.2 ~~g~~. These requirements significantly limit the use of practical Equivalent Lateral Force (ELF) and Modal Response Spectrum Analysis (MRSA) design methods, which is of particular significance for Site Class D sites. To lessen the effect of these requirements on design practice, two ~~three~~ exceptions permit the use of conservative values of design parameters for certain conditions for which conservative values of design were identified by the ELF study. ~~These exceptions do not apply to seismically isolated structures and structures with damping systems for which site-specific analysis is required in all cases at sites with S_1 greater than or equal to 0.6.~~

The Item 1 exception is intended as an acceptable way to address the inaccuracy of the spectral shape observed in the velocity domain for Site Class D sites subject to high ground motions. Increasing S_{M1} by 50% in Eq. (11.4-2) results in an increase in the value of S_{D1} determined by Eq. (11.4-4) by 50 percent. These increased values of S_{M1} and S_{D1} are to be used for all applications of these parameters throughout the Standard, including for the formulation of the design response spectrum where a design response spectrum is needed per this standard. It should be noted that the 50% increase in S_{D1} also increases T_s by 50% resulting in an extension of the acceleration-controlled plateau of the design response spectrum. It is important to appropriately capture the change in the spectrum shape when evaluating building drifts using the linear dynamic analysis procedures of Section 12.9 since drift results are not scaled unless C_s is determined in accordance with Eq. (12.8-6).

~~The first exception permits use of the value of the site coefficient F_a of Site Class C ($F_a = 1.2$) for Site Class E sites (for values of S_S greater than or equal to 1.0 g) in lieu of site-specific hazard analysis. The ELF study found that while values of the site coefficient F_a tend to decrease with intensity for softer sites, values of spectrum shape adjustment factor C_a tend to increase such that the net effect is approximately the same intensity of $\mathrm{MCE_R}$ ground motions for Site Classes C, D, and E when $\mathrm{MCE_R}$ ground motion intensity is strong (i.e., $S_{MS} \geq 1.0$). Site Class C was found to not require spectrum shape adjustment, and the value of site coefficient F_a for Site Class C ($F_a = 1.2$) is large enough to represent both site class and spectrum shape effects for Site Class E (and Site Class D).~~

~~The second exception permits bothELF~~ and MRSA ~~design of structures at Site Class D sites for values of S_1 greater than or equal to 0.2 g, provided that the value of the seismic response coefficient C_s is conservatively calculated using Eq. (12.8-2) for $T \leq 1.5T_s$ and using 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$. This exception recognizes that structures are conservatively designed for the response spectral acceleration defined by the domain of constant acceleration (S_{DS}) or by a 50% increase in the value of seismic response coefficient C_s for structures with longer periods ($T \geq 1.5T_s$). The underlying presumption of this exception for MRSA design of structures is that the shape of the design response spectrum (Fig. 11.4-1) is sufficiently representative of the frequency content of Site Class D ground motions to permit use of MRSA and that the potential underestimation of fundamental-mode response using the design response spectrum shape of Fig. 11.4-1 is accounted for by scaling MRSA design values (Section 12.9.1.4) with a conservative value of the seismic response coefficient Cs. In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites (S1>=0.2 g).~~

The ~~third~~ first exception under Item 2 permits ELF design for all periods of T when Eq. (12.8-2) is used to determine C_s ~~of short-period structures ($T \leq T_s$)~~ at Site Class E sites for values of S_S greater than or equal to 1.0 or S_1~~Ss~~ greater than or equal to 0.2~~ g~~. This exception recognizes that designing ~~short-period~~ structures for Eq. (12.8-2) provides a conservative design for all values of T since ~~are conservatively designed using the ELF procedure for values of seismic response~~

~~coefficient~~ C_s is based on the domain of constant acceleration (S_{DS}), which is, in all cases, greater than or equal to response spectral accelerations of the domain of constant velocity and therefore need not consider the effects of spectrum shape at longer periods. Similarly, the second exception modifies S_{ai} in Eq. (15.7-7) for ground supported tanks. For the convective seismic forces, the parameter of S_{D1} is replaced with $1.5S_{D1}$ to account for the change in the response spectrum at long periods. ~~$T > T_s$. In general, the shape of the design response spectrum (Fig. 11.4-1) is not representative of the frequency content of Site Class E ground motions, and MRSA is not permitted for design unless the design spectrum is calculated using the site-specific procedures of Section 21.2.~~