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 $10\ \text{ft/s}\ (3.0\ \text{m/s})$ and need not be taken as greater than the lesser of $1.5(gh_{\max})^{1/2}$ and $50\ \text{ft/s}\ (15.2\ \text{m/s})$.

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

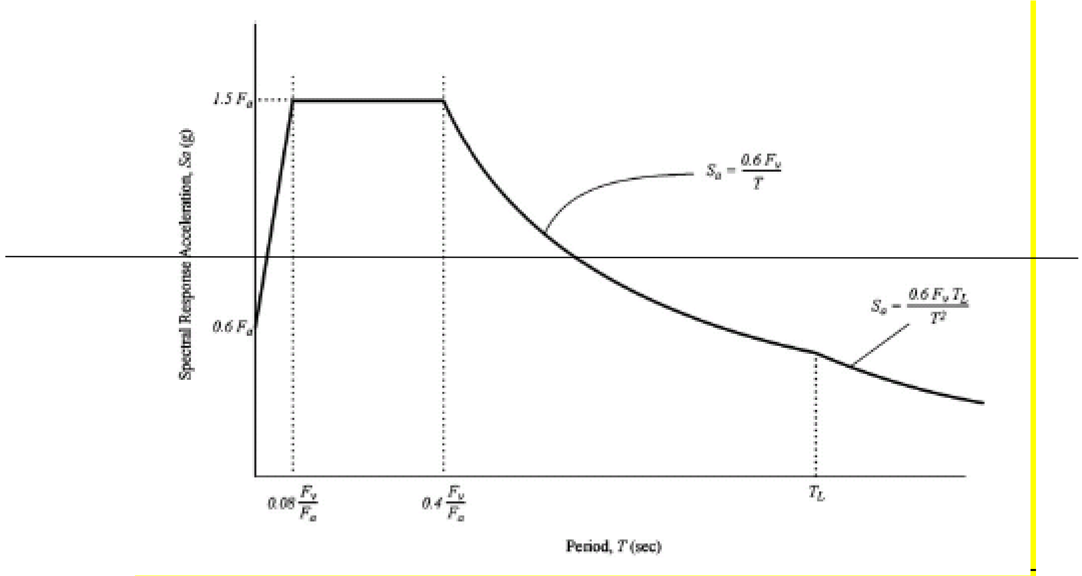
SECTION 21.2.2 AS FOLLOWS:

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):

$$S_a = \frac{2}{3} S_{aM} \qquad (21.3\text{-}1)$$

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:

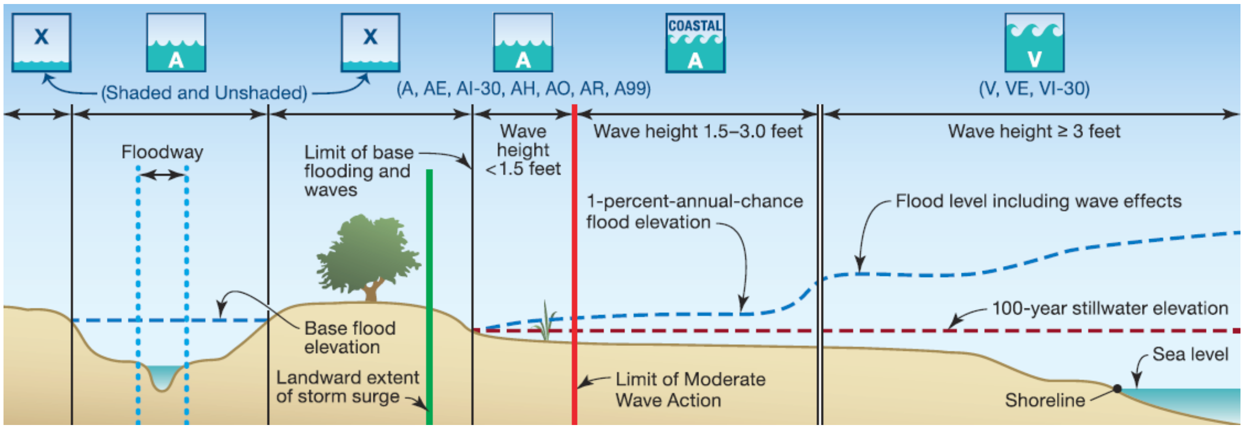
C2.3.2 Load Combinations Including Flood Load.

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.

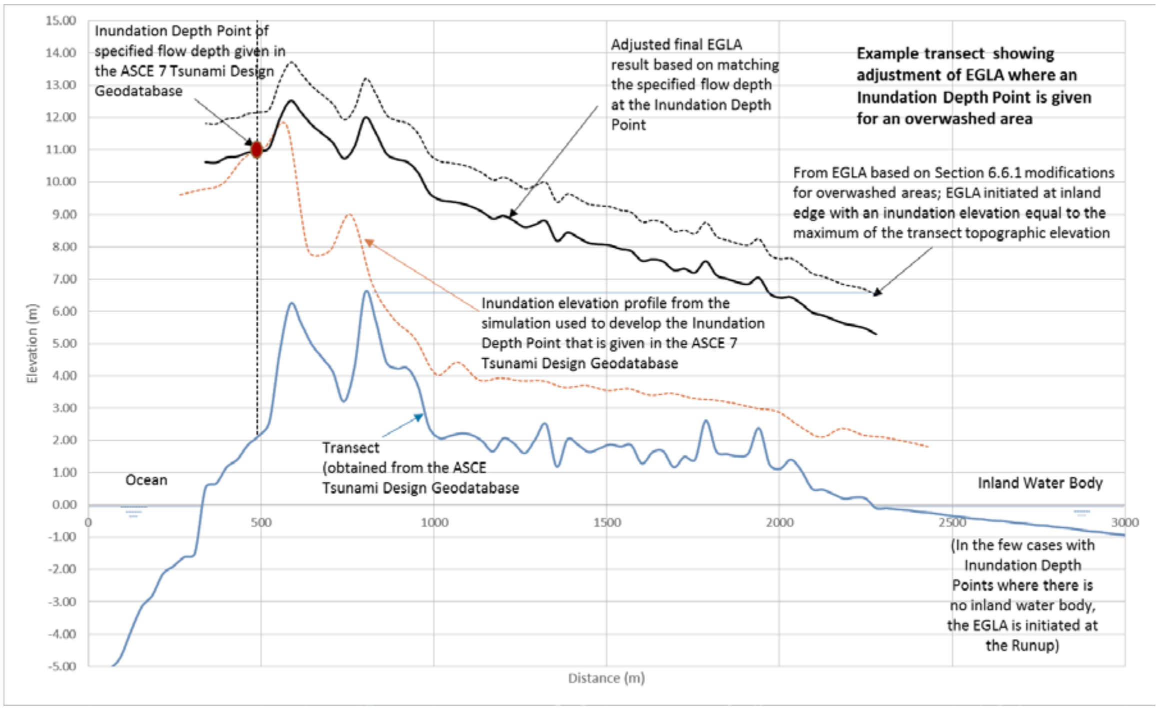


FIGURE C6.6-3 Example EGLA with adjustment where an Inundation Depth Point is specified in the ASCE Tsunami Design Geodatabase

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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 MCE_R.

Because of the deterministic lower limit on the MCE_R spectrum (Fig. 21.2-1), the site-specific 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 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 MCE_R ground motions solely in terms of probabilistic MCE_R ground motions (i.e., when peak 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.