Seismic Code Provisions (Geotechnical Considerations)

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Main References

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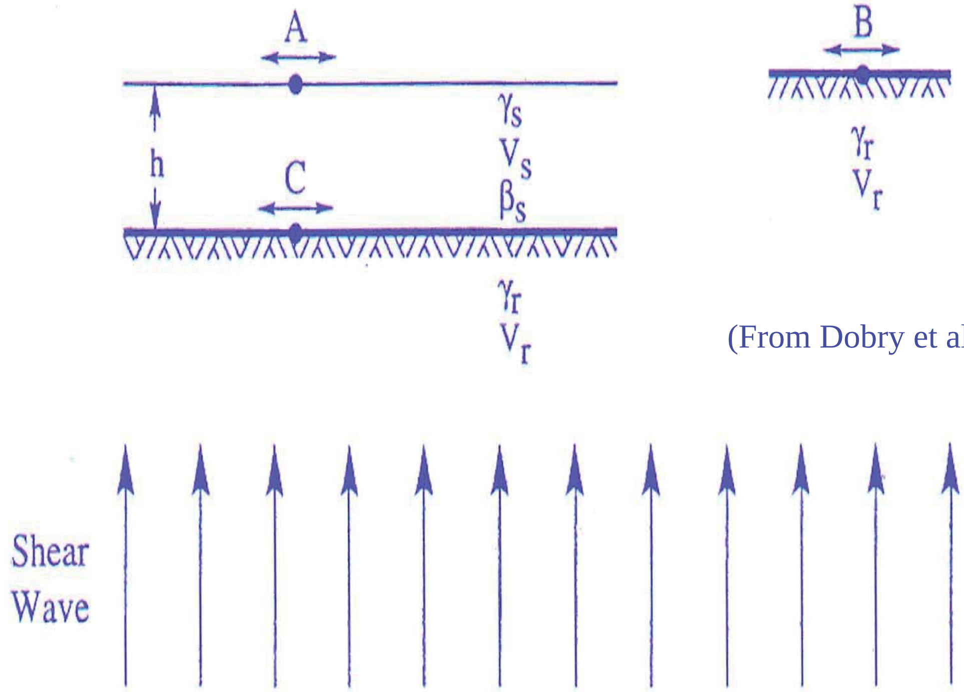
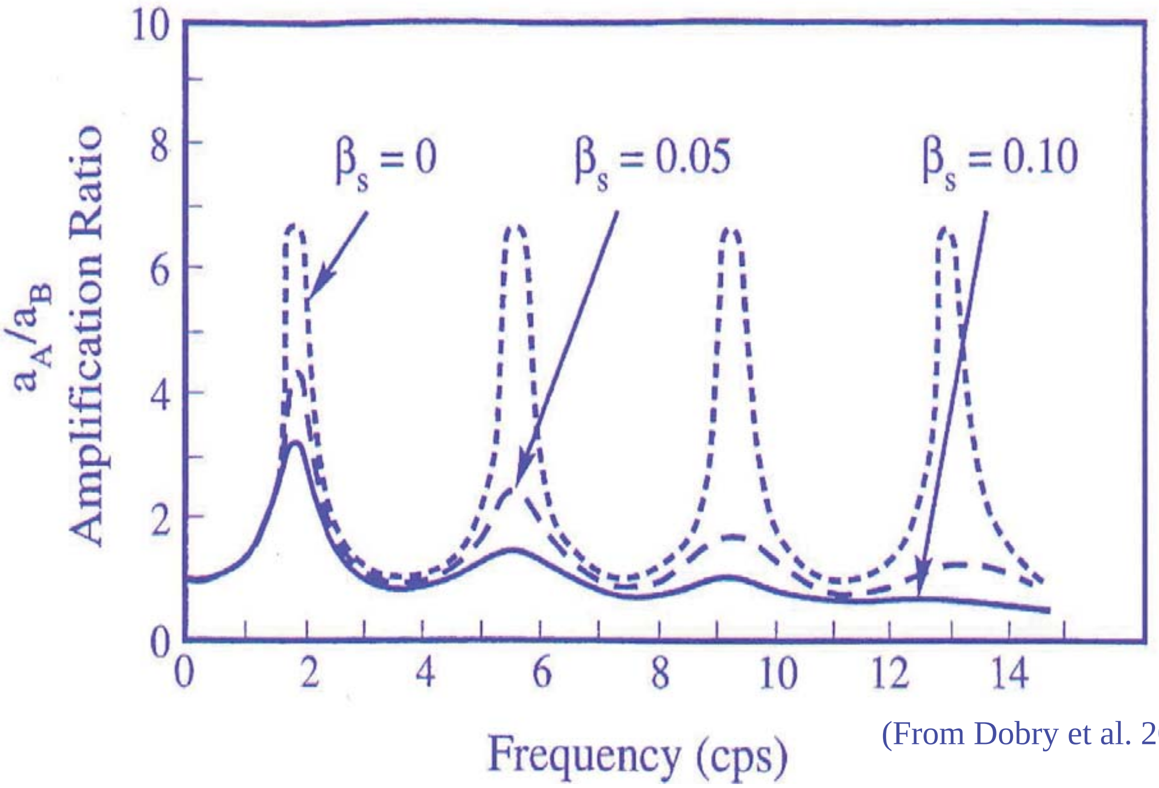
Building Seismic Safety Council, (BSSC), 2004b. NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, FEMA 450-2/2003 Edition, Part 2: Commentary.

Site Response Effects



(From Dobry et al. 2000)



(From Dobry et al. 2000)

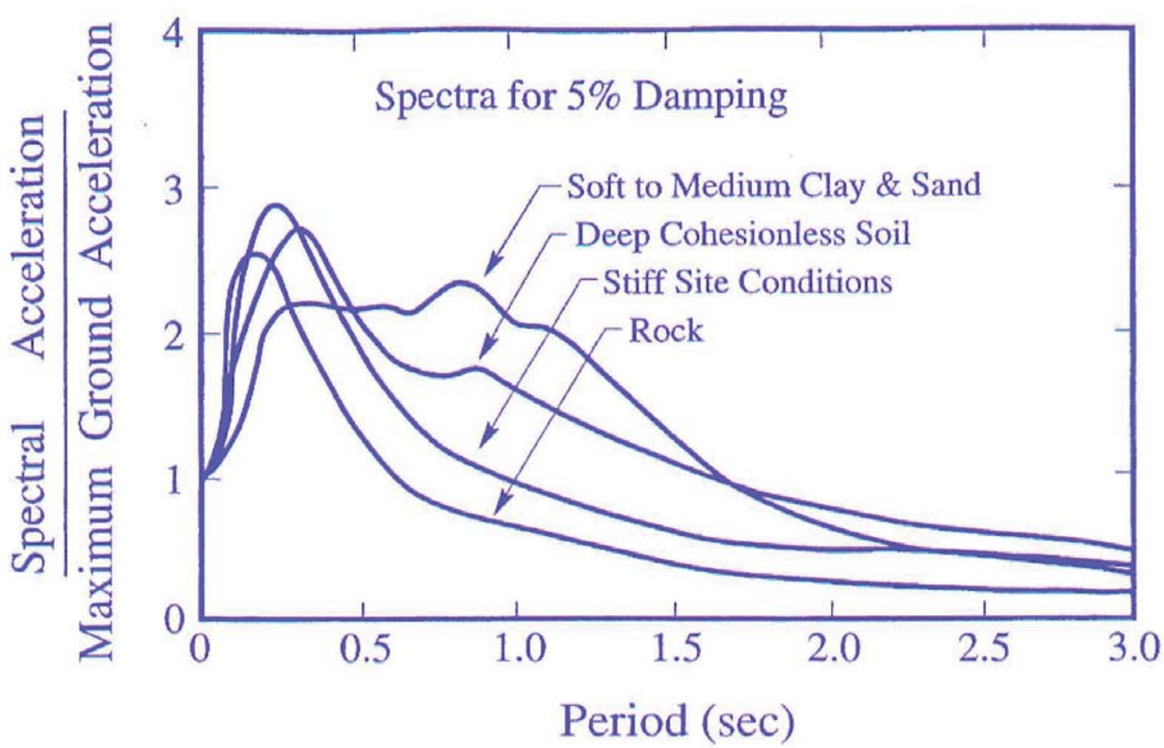
Historical Background

ATC 1978
Three distinct site coefficients S_1-S_3 (include soil type and depth) based on Statistical Studies by Seed and co-workers (1976) and Mohraz (1976).

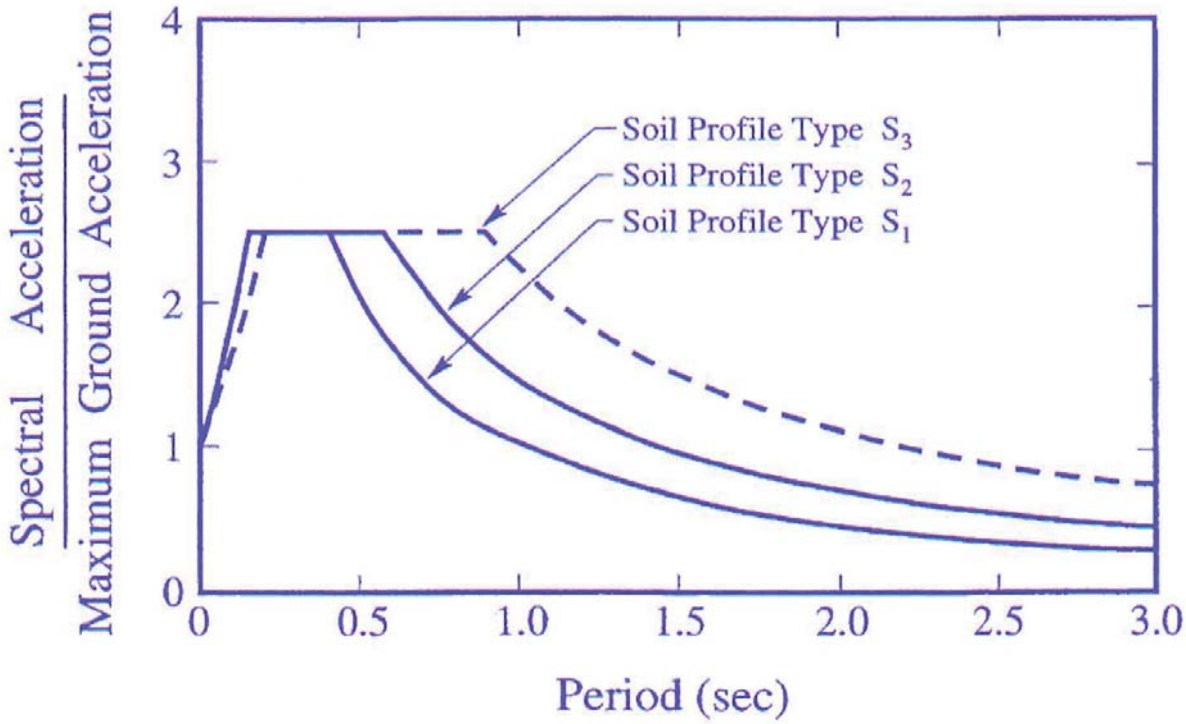
Experience from the 1985 Mexico City Earthquake resulted in the addition of a fourth site category S_4 with a higher value to account for deep soft clay deposits.

Each S value was associated with a different spectral shape

The S value only amplified the long period part of the Spectrum



(From Dobry et al. 2000)



(From Dobry et al. 2000)

Table 1. Soil profile types and site factors for calculation of lateral force contained in seismic codes prior to the *1994 NEHRP Provisions* (modified after Martin and Dobry 1994)

Soil Profile Type	Description	Site Coefficient, **S**
S_1	A soil profile with either (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 feet per second or (2) stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying the rock are stable deposits of sands, gravels, or still clays.	1.0
S_2	A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.2
S_3	A soil profile containing 20 to 40 feet in thickness of soft-to medium-stiff clays with or without intervening layers of cohesionless soils.	1.5
S_4	A soil profile characterized by a shear wave velocity of less than 500 feet per second containing more than 40 feet of soft clays or silts.	2.0

(From Dobry et al. 2000)

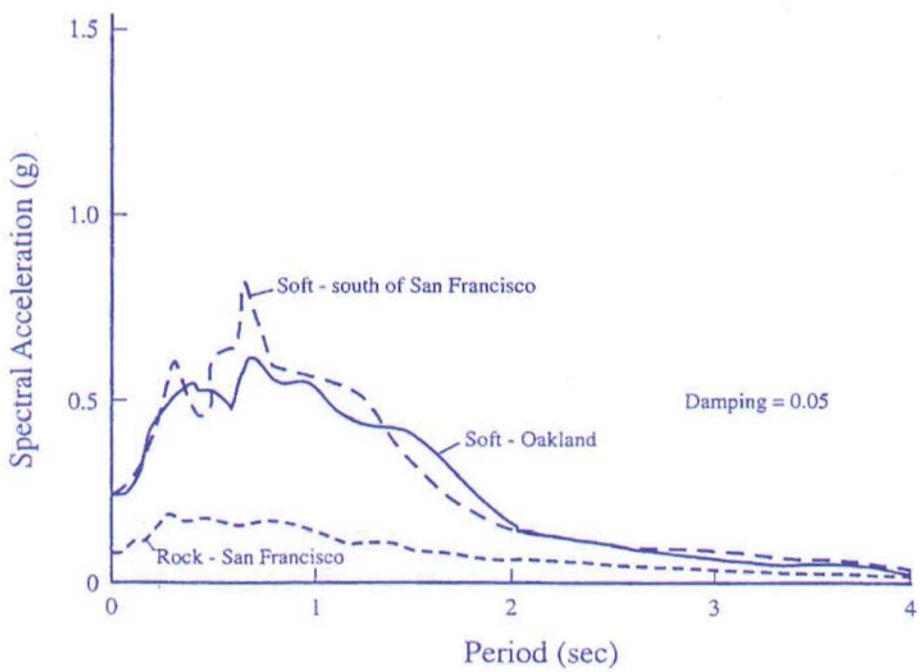
New Developments

Mexico city (1985) and Loma Prieta (1989) earthquakes (among other studies) showed that the effect of the level of shaking (low levels of peak ground acceleration and low spectral levels at short periods can be significantly amplified at soft sites (Idriss 1990 -1991and Fig. 1).

New York City studies (e.g., Jacob 1990, 1994) introduced two new key aspects:

1. Higher values of site coefficients (due to low excitation levels)
2. Addition of hard rock category to account for much stiffer rock in the eastern US (compared to CA).

Fig. 1: Average spectra recorded during 1989 Loma Prieta Earthquake in San Francisco Bay Area at rock sites and soft soil sites (modified after Housner 1990)



(From Dobry et al. 2000)

New Developments

NCEER Workshop (Whitman 1992) with 9 members (authors of Dobry et al. 2000 paper)

Los Angeles Workshop (Martin 1994) with 65 invited geoscientists, geotechnical and structural engineers developed consensus recommendations incorporated in the 1994 and 1997 NEHRP Provisions and 1997 UBC provisions

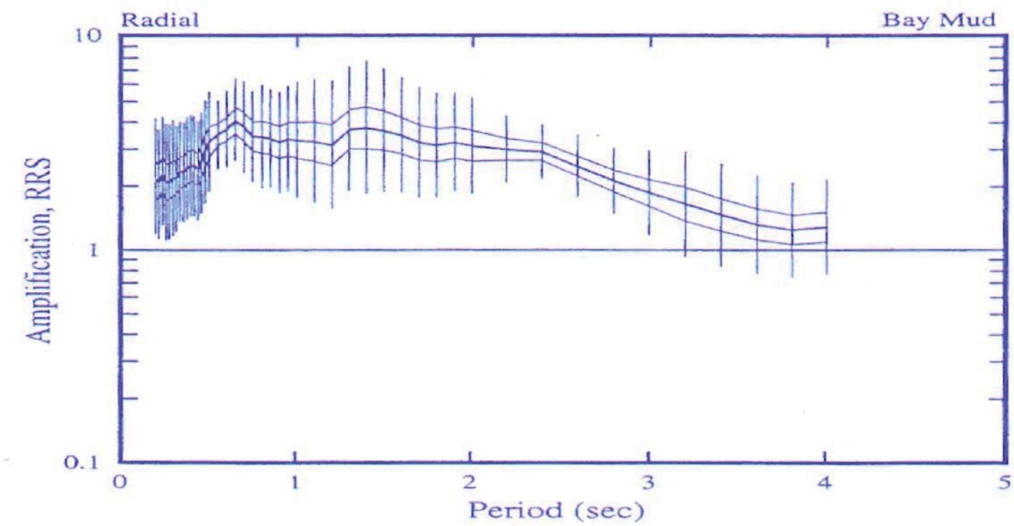
Peak Acc on Rock 0.08 - 0.10g amplified 2-3 times to 0.2g – 0.3g at soft soil sites

Spectral ordinates at short periods (T =0.2 0r 0.3 seconds) were also amplified by factors of 2 or 3

Fig. 2 shows amplification of response spectra between nearby rock and soil sites (Ratio of Response Spectra or RRS)

Both Figs. 1 and 2 show that amplification at T of 0.5 to 1.5 or 2 seconds is even greater than at shorter periods with RRS ranging from 3 to 6.

Similar amplification characteristics (with lower values) was also observed between rock and stiff soil sites.

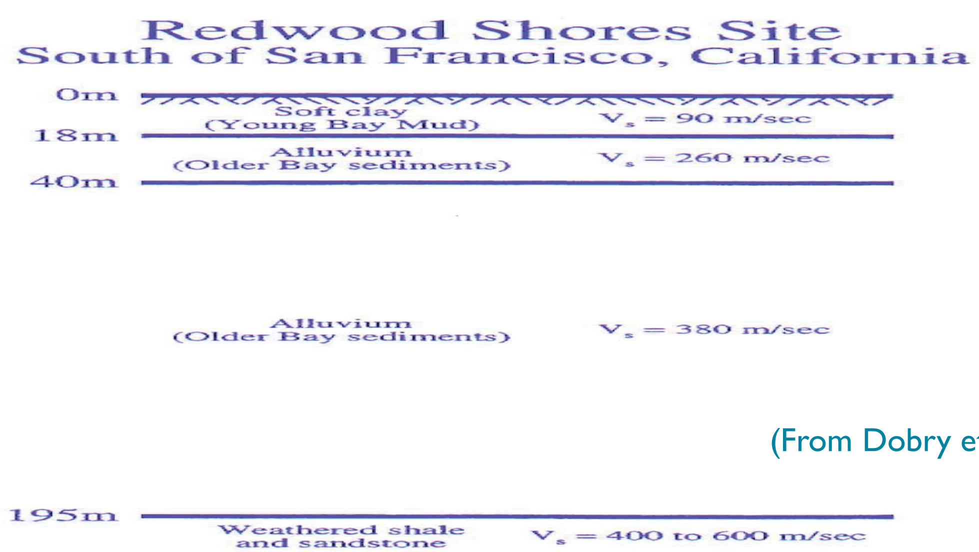


Average RRS curves, Loma Prieta earthquake. Curves show geometric average and plus and minus 1 standard deviation. Vertical lines show range (note log scale), (after Joyner et al. 1994).

At some of these sites, amplification occurred at long periods seemingly related to the characteristics of the soil deposit

(Fig. 2, From Dobry et al. 2000)

Profile of representative instrumented soft clay site, 70 km north
Of epicenter, RSS max about 3.5 at T of about 1.4 second. In other extreme (but unusual) cases (Mexico city), at T of about 2 –3 sec. RSS max ranged from 3 to 20.



(From Dobry et al. 2000)

Fig. 3: Soil profile at instrumented soft clay site with a 1989 Loma Prieta earthquake record.

Code Provisions

Site Classes

Old

Four S_1-S_4

New

Six site classes (A – F) in terms of representative average shear wave velocity to a depth of 30 m ($\overline{V}_S$)

SPT or S_u may be used instead of $\overline{V}_S$

$$\overline{V}_S = \frac{100 \text{ ft}}{\Sigma (d_i / v_{si})} \quad \text{or} \quad \frac{30.5 \text{ m}}{\Sigma (d_i / v_{si})}$$

- A: Hard Rock
- B: Rock
- C: Soft rock and very stiff / very dense soil
- D: stiff soil
- E: Soft soil
- F: Requires site-specific evaluation

Table 1. Soil profile types and site factors for calculation of lateral force contained in seismic codes prior to the *1994 NEHRP Provisions* (modified after Martin and Dobry 1994)

Soil Profile Type	Description	Site Coefficient, S
S_1	A soil profile with either (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 feet per second or (2) stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying the rock are stable deposits of sands, gravels, or still clays.	1.0
S_2	A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.2
S_3	A soil profile containing 20 to 40 feet in thickness of soft-to medium-stiff clays with or without intervening layers of cohesionless soils.	1.5
S_4	A soil profile characterized by a shear wave velocity of less than 500 feet per second containing more than 40 feet of soft clays or silts.	2.0

(From Dobry et al. 2000)

Site Class or Soil Profile Type	Description	Shear Wave Velocity $\bar{V}_S$ Top 30m (m/sec)	Standard Pen. Resistance $\bar{N}$ or $\bar{N}_{ch}$ (blows/ft)	Undrained Shear Strength $\bar{S}_u$ (kP$_a$)
$S1$ A	Hard Rock	> 1500	—	—
$S1$ B	Rock	760 - 1500	—	—
$S1$ and $S2$ C	Very dense soil/soft rock	360 - 760	> 50	> 100
$S1$ and $S2$ D	Stiff soil	180 - 360	15 - 50	50 - 100
$S3$ and $S4$ E	Soft soil	< 180	< 15	< 50
$S3$ and $S4$ F	Special soils requiring site-specific evaluation	—	—	—

(From Dobry et al. 2000)

Code Provisions

Site amplification factors

Old

S_v factor

No short period amplification factor

New

F_a for short periods

F_v for longer periods

New Site Amplification Coefficients

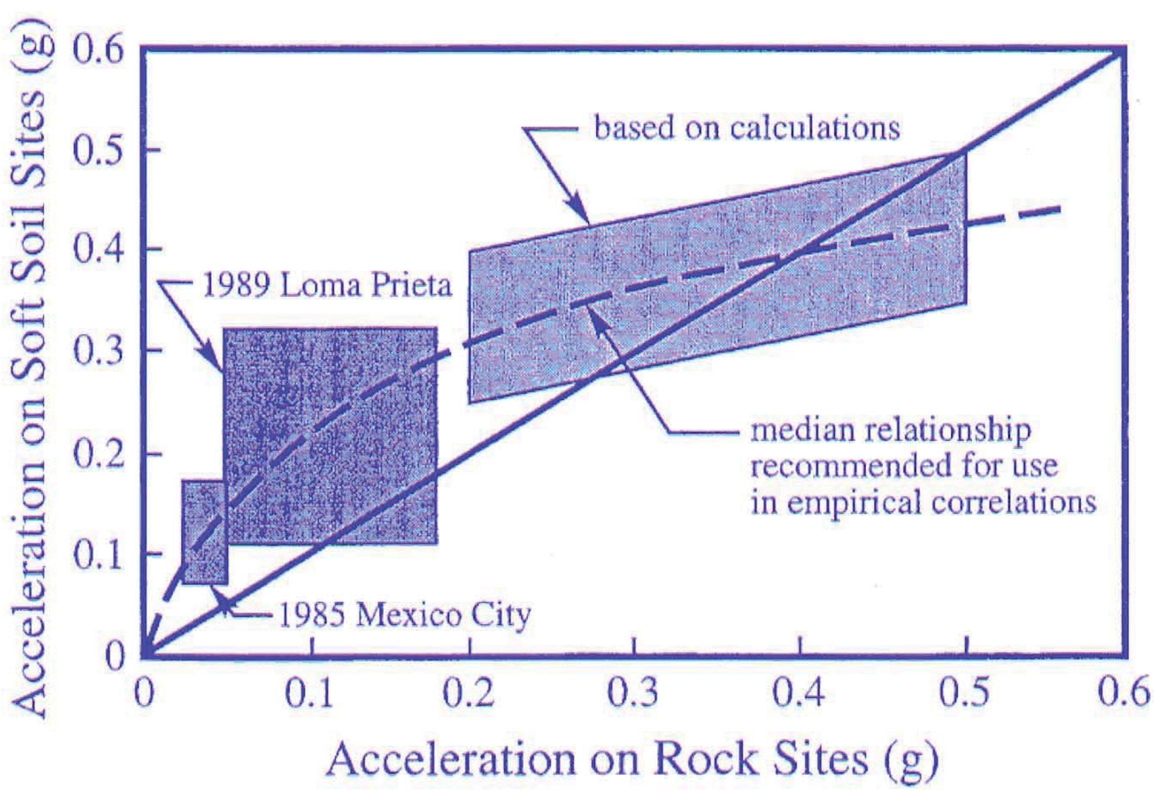
Both F_a and F_v are functions of site class (A-F) AND level of seismic hazard on rock (Intensity of Rock Motions) defined by parameters such as:

A_a and A_v (1994 NEHRP provisions, available in Seismic maps)
Z (1997 UBC, based on seismic zone maps for Z in the US)
S_s and S_l (1997 NEHRP provisions, IBC 2006)

NEHRP: National Earthquake Hazards Reduction Program
UBC: Uniform Building Code
IBC: International Building code

F_a and F_v decrease with the increase of seismic hazard on rock due to soil non-linearity.

Greatest impact of F_a and F_v compared to the old S factors is in the area of low-medium seismic hazard



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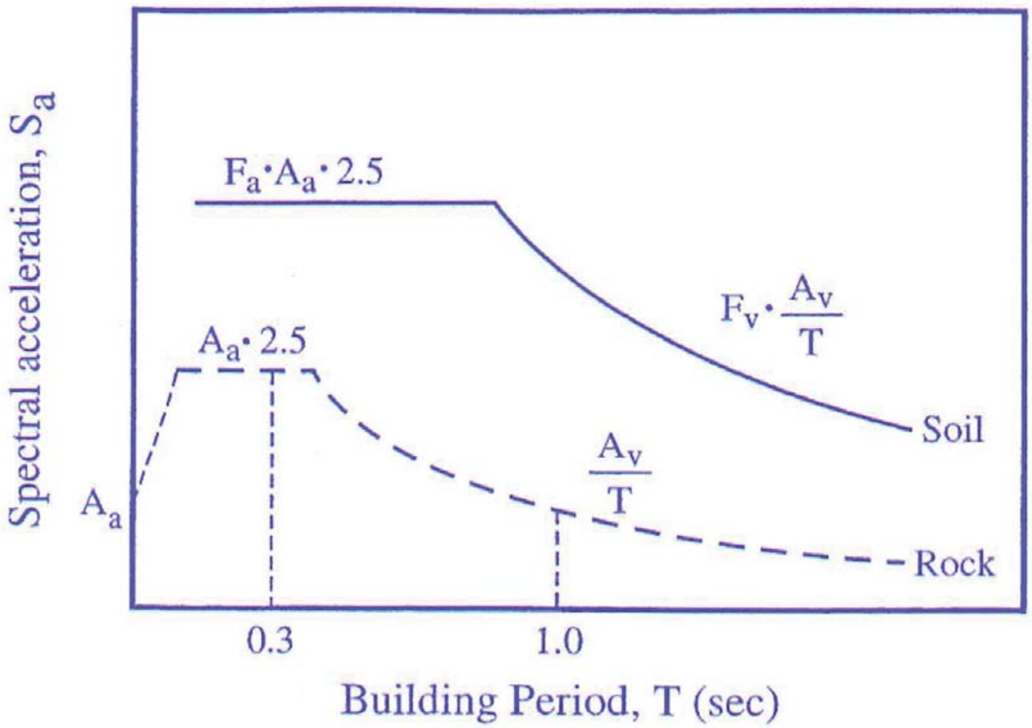


(NEHRP 1997)

(a) Short period site coefficient F_a

Site Class Or Soil Profile Type	Mapped Rock Shaking Level at Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
	$A_a \leq 0.10$	$A_a = 0.20$	$A_a = 0.30$	$A_a = 0.40$	$A_a \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*
F	*	*	*	*	*

(b) Long period site coefficient F_v

Site Class or Soil Profile Type	Mapped Rock Shaking Level at Long Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
	$A_v \leq 0.10$	$A_v = 0.20$	$A_v = 0.30$	$A_v = 0.40$	$A_v \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*
F	*	*	*	*	*

(*) Site-specific geotechnical investigation and dynamic site response analyses shall be performed

(from Dobry et al. 2000)

Codes only set minimum standards and are intended to help prevent catastrophic failures

NEHRP 1994

(National Earthquake Hazard Reduction Program)

Seismic Base Shear $V = C_s W$

$C_s = 1.2 A_v F_v / (R T^{2/3})$

Not to exceed

$C_s = 2.5 A_a F_a / R$

A_a and A_v are available from seismic Hazard maps for particular location of interest

W = Total seismic load
(combination of dead and live loads)

R = Ductility factor

T = Fundamental Period of Structure
(see Code for suggested formulae)

Codes only set minimum standards and are intended to help prevent catastrophic failures

Static Force Procedure (Uniform Building Code 1997)

The total design base shear (V) shall be determined from

$V = ((C_v I) / R T) W$

$V < ((2.5 C_a I) / R) W$

$V > 0.11 C_a I W$

W = Total seismic load
(combination of dead and live loads)

I = Seismic Importance Factor
$1.25 > I > 1.0$

R = Ductility factor
$8.5 > R > 2.2$

T = Fundamental Period of Structure

For seismic zone 4 (highly seismic zone)

$V > ((0.8 Z N_v I) / R) W$

Z= seismic zone factor = 0.4, N_v = Near source factor > 1.0 (15 Km away) and < 2 (2 Km away), and depends on seismic source type

UBC 1997

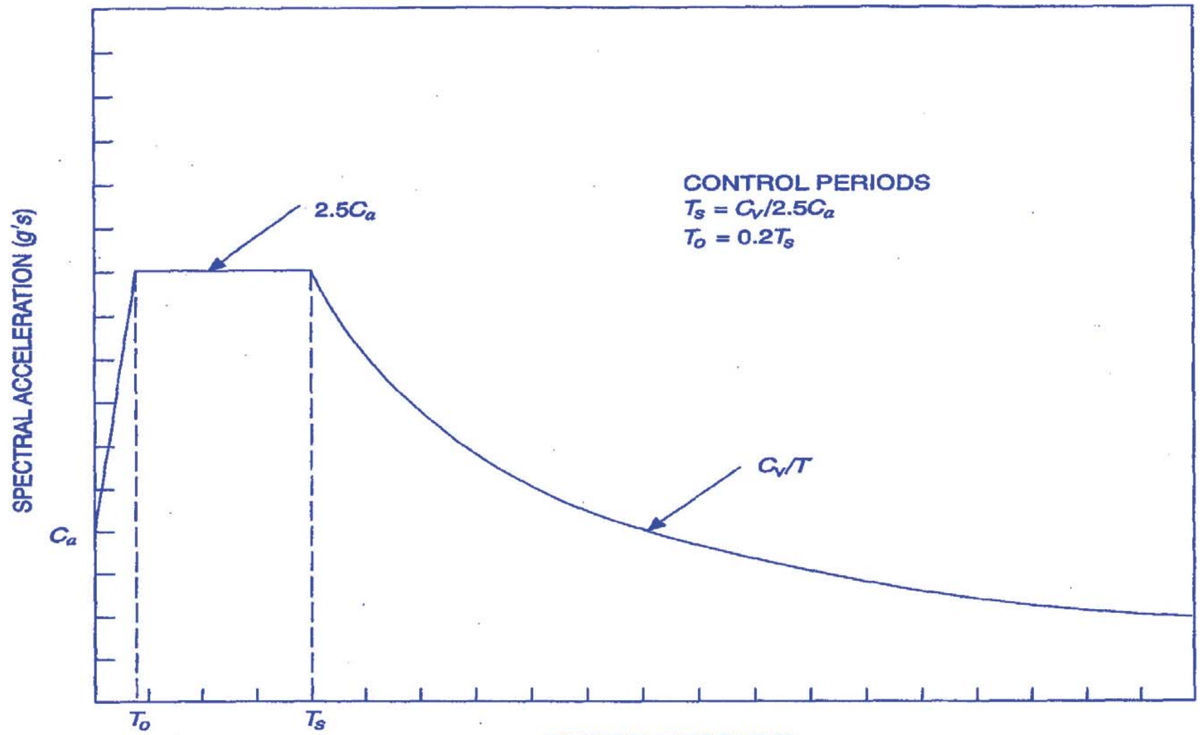


FIGURE 16-3—DESIGN RESPONSE SPECTRA

UBC 1997

TABLE 16-Q—SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16-R—SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

Note that C_a and C_v are the equivalent of A_aF_a and A_vF_v in NEHRP

UBC 1997

TABLE 16-S—NEAR-SOURCE FACTOR N_a[1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]		
	≤ 2 km	5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

[1]The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2]The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3]The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

TABLE 16-T—NEAR-SOURCE FACTOR N_v[1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

[1]The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2]The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3]The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

TABLE 16-U—SEISMIC SOURCE TYPE[1]

SEISMIC SOURCE TYPE	SEISMIC SOURCE DESCRIPTION	SEISMIC SOURCE DEFINITION[2]	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

[1]Subduction sources shall be evaluated on a site-specific basis.

[2]Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining the seismic source type.

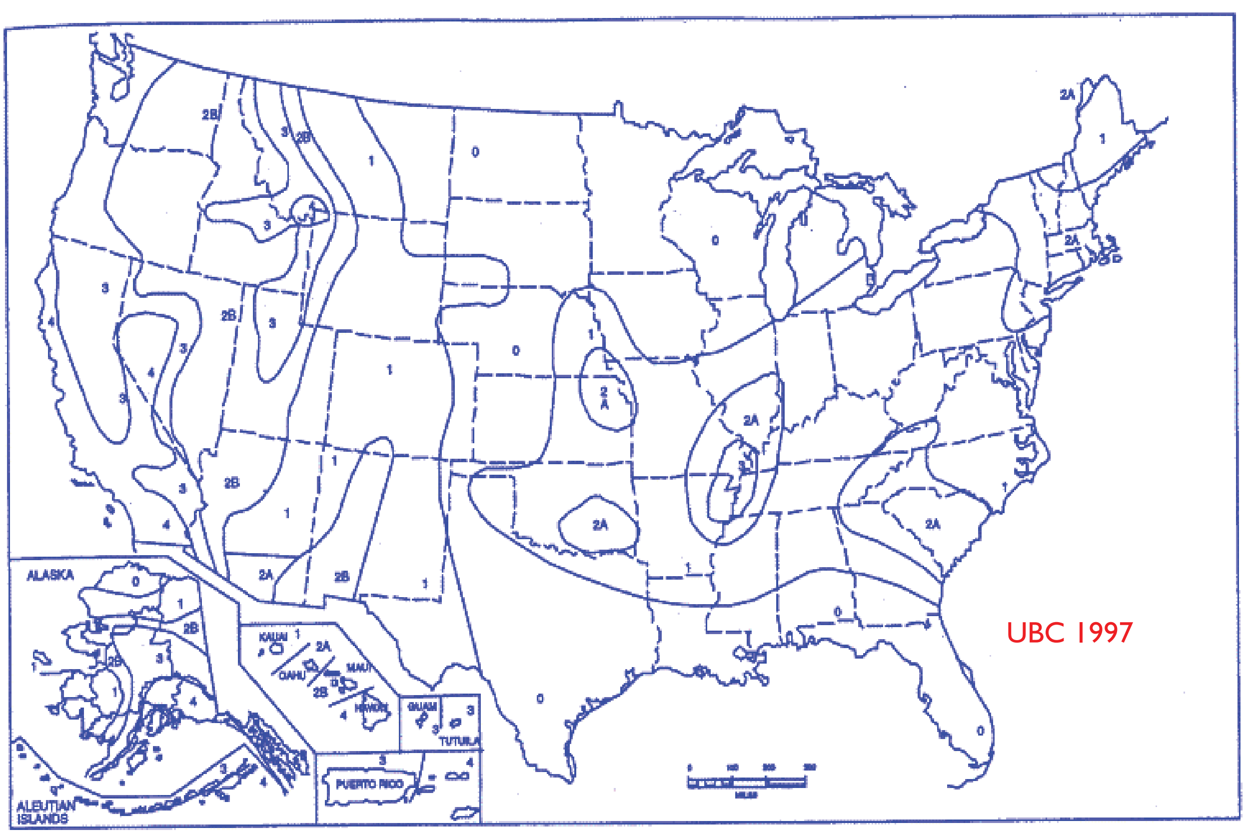


FIGURE 16-2—SEISMIC ZONE MAP OF THE UNITED STATES
For areas outside of the United States, see Appendix Chapter 16.

Codes only set minimum standards and are intended to help prevent catastrophic failures

Static Force Procedure (IBC 2006)

The total design base shear (V) shall be determined from

V = $C_s W$, and $C_s = \dfrac{S_{DS}}{\left(\dfrac{R}{I}\right)}$

with C_s not to exceed:

For $T < T_L$ $C_s = \dfrac{S_{D1}}{T\left(\dfrac{R}{I}\right)}$

For $T > T_L$ $C_s = \dfrac{S_{D1}T_L}{T^2\left(\dfrac{R}{I}\right)}$

(Crouse *et al.* 2006)

W = Total seismic load
(combination of dead and live loads)

I = Seismic Importance Factor
1.5 > I > 1.0

R = Ductility factor
8.0 > R > 1.25

T = Fundamental Period of Structure

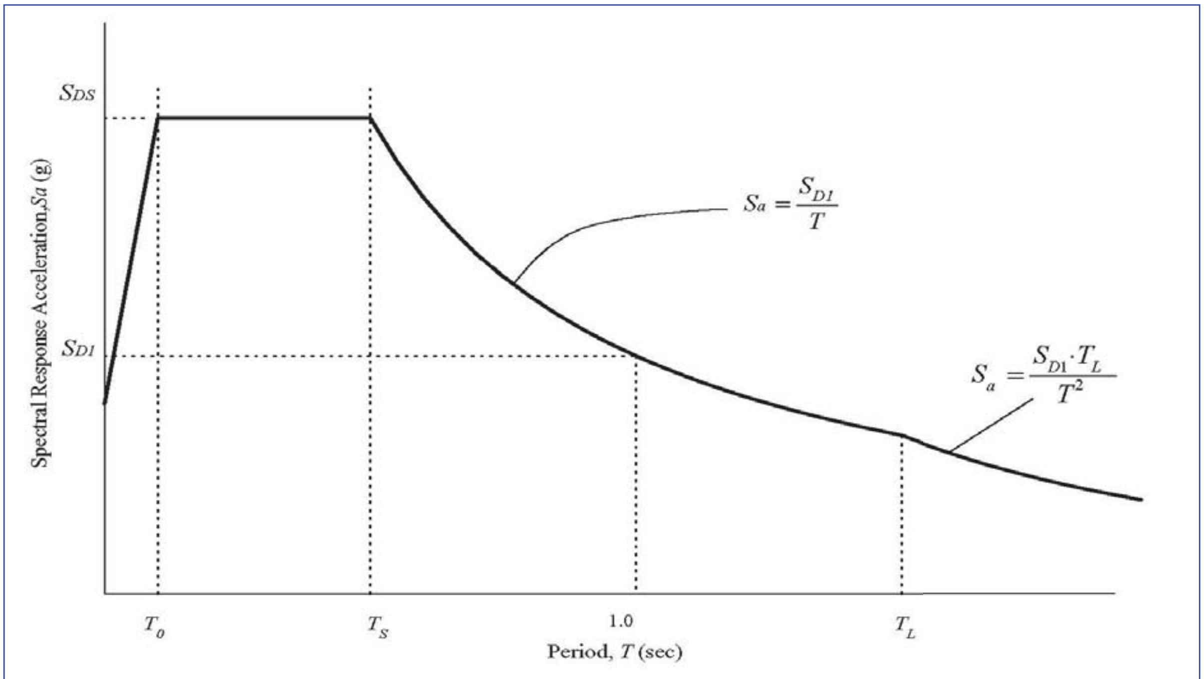


FIGURE 11.4-1 DESIGN RESPONSE SPECTRUM

$$S_{DS} = \frac{2}{3} F_a S_S \qquad S_{D1} = \frac{2}{3} F_v S_1$$

(after IBC 2006)

Check on minimum value of C_s –

Cs shall not be less than

$$C_s = 0.01$$

In addition, for structures located where S_1 (the one-second period spectral acceleration) is equal to or greater than 0.6g, C_s shall not be less than (to implicitly account for near source effects):

$$C_s = \frac{0.5\, S_1}{\left(\frac{R}{I}\right)}$$

Site Coefficient, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Site Specific Ground Motions investigation				

(after IBC 2006)

Site Coefficient, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Site Specific Ground Motions Investigation				

Site Classes

The site soil shall be classified in accordance with the Table below based on the upper 100 ft (30 m) of the site profile.

TABLE 20.3-1 SITE CLASSIFICATION

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: - Plasticity index PI > 20, - Moisture content $w \geq 40\%$, and - Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s 1 lb/ft^2 = 0.0479 kN/m^2

(after IBC 2006)

$\bar{v}_s$ is the average shear wave velocity in the upper 100 ft of the site profile and is calculated as:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

d_i is the thickness of any layer between 0 and 100 ft (30 m).
v_{si} is the shear wave velocity in ft/s (m/s).

Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site. Site Classes A and B shall not be assigned to a site if there is more than 10 ft of soil between the rock surface and the bottom of the spread footing or mat foundation.

$\tilde{N}$ Average Field Standard Penetration Resistance, and

$\tilde{N}_{ch}$ Average Standard Penetration Resistance for Cohesionless Soil Layers, shall be determined in accordance with the following formulas:

$$\tilde{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad \text{and} \quad \tilde{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where N_i and d_i are for cohesionless soil layers only and $\sum_{i=1}^{m} d_i = d_s$

where d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m). N_i is the standard penetration resistance (ASTM D1586) not to exceed 100 blows/ft (328 blows/m) as directly measured in the field without corrections. Where refusal is met for a rock layer, N_i shall be taken as 100 blows/ft (328 blows/m).

(after IBC 2006)

$\overline{s}_u$ is the Average Undrained Shear Strength. $\overline{s}_u$ shall be determined in accordance with the following formula:

$$\overline{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

where $\sum_{i=1}^{k} d_i = d_c$

and,

d_c = the total thickness of cohesive soil layers in the top 100 ft (30 m)

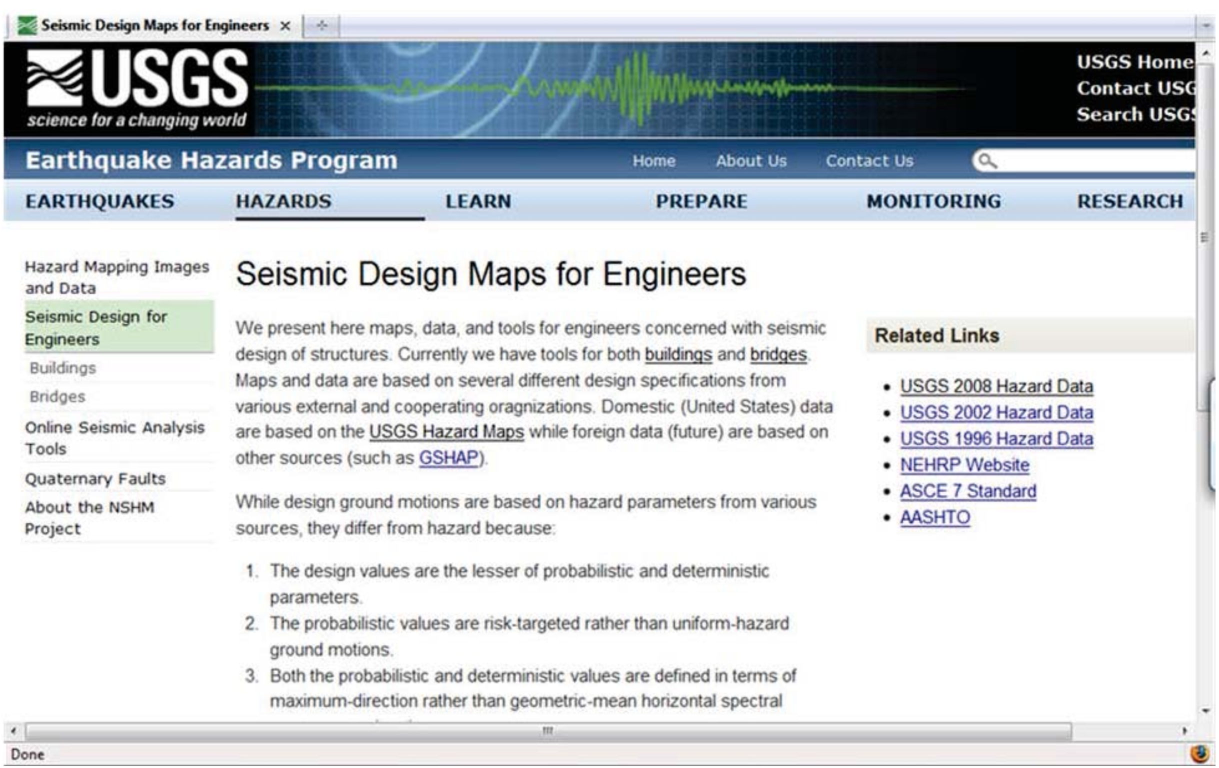
PI = the plasticity index as determined in accordance with ASTM D4318

w = the moisture content in percent as determined in accordance with ASTM D2216

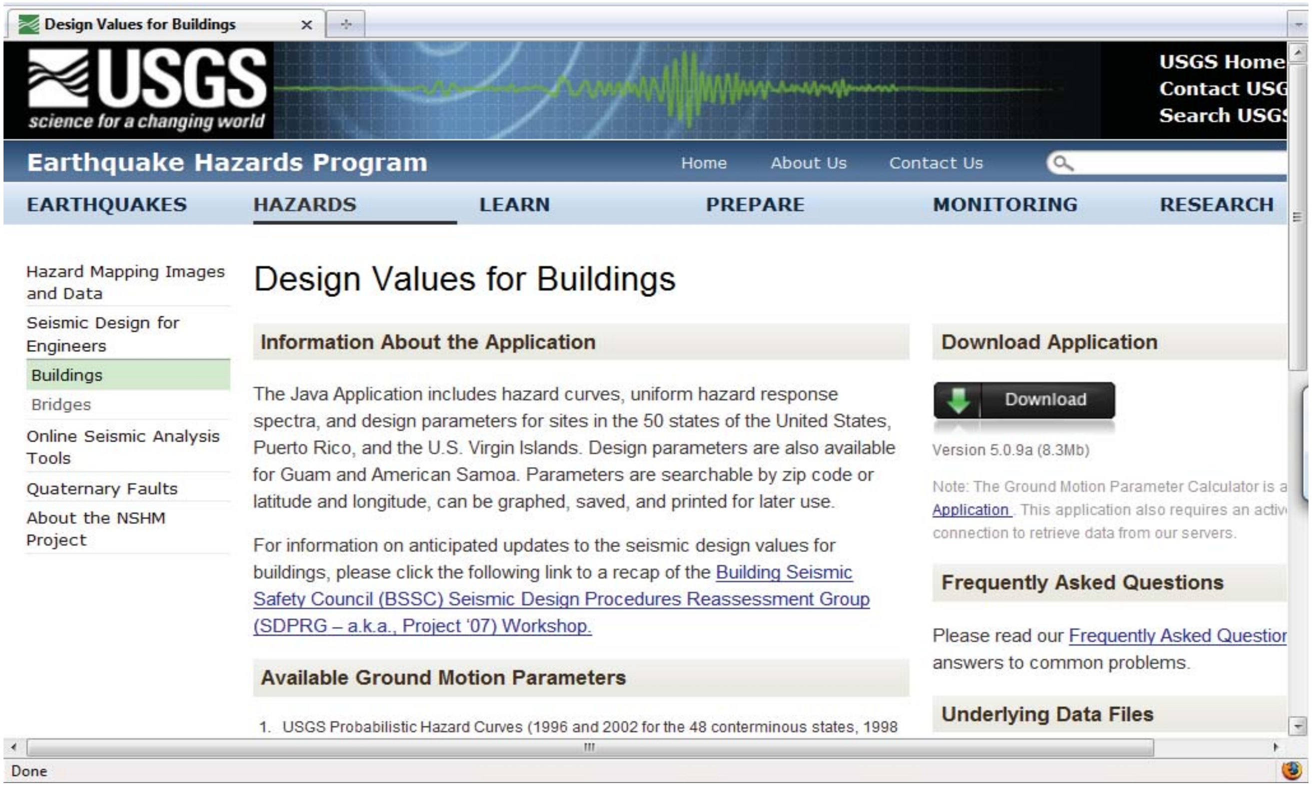
s_{ui} = the undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa) as determined in accordance with ASTM D2166 or ASTM D2850

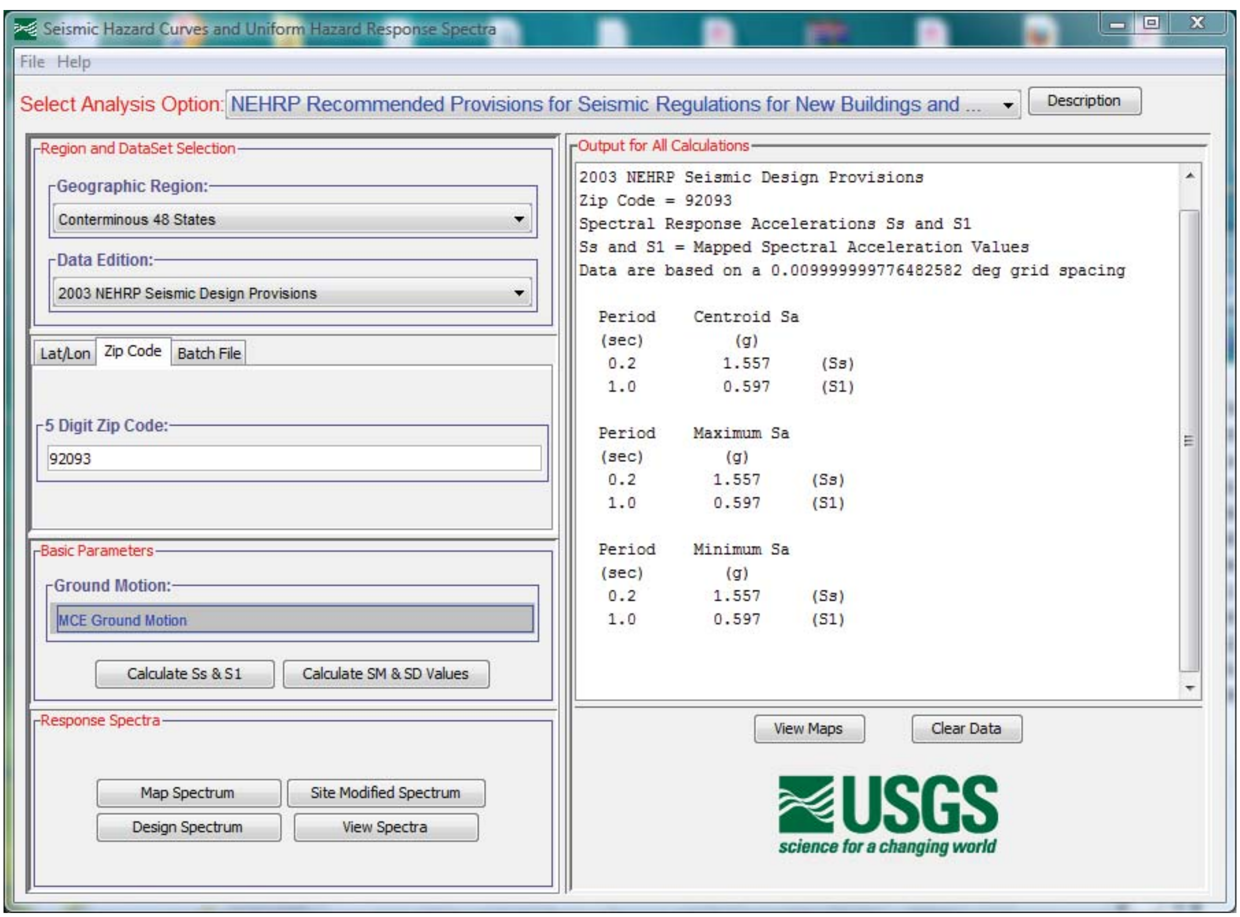
(after IBC 2006)

To determine S_s and S_1 go to (or see maps in the Code):
http://earthquake.usgs.gov/hazards/design/

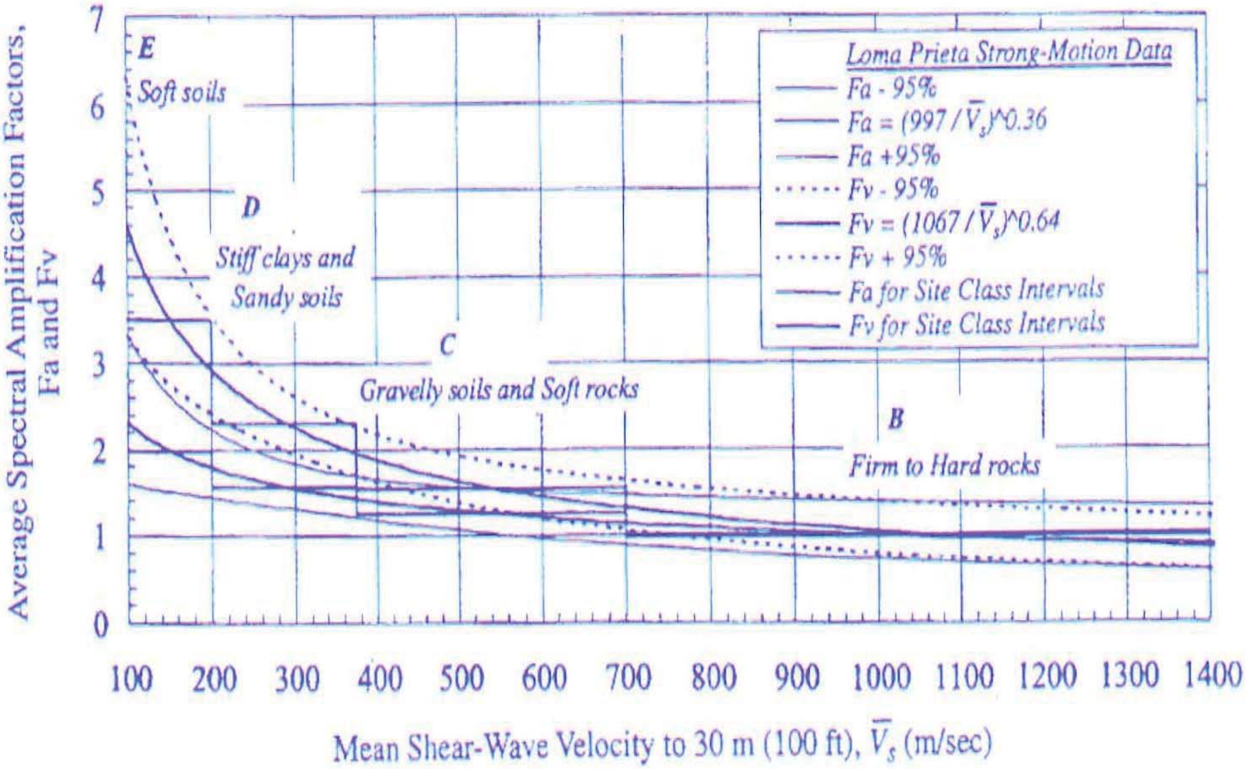


http://earthquake.usgs.gov/hazards/design/buildings.php

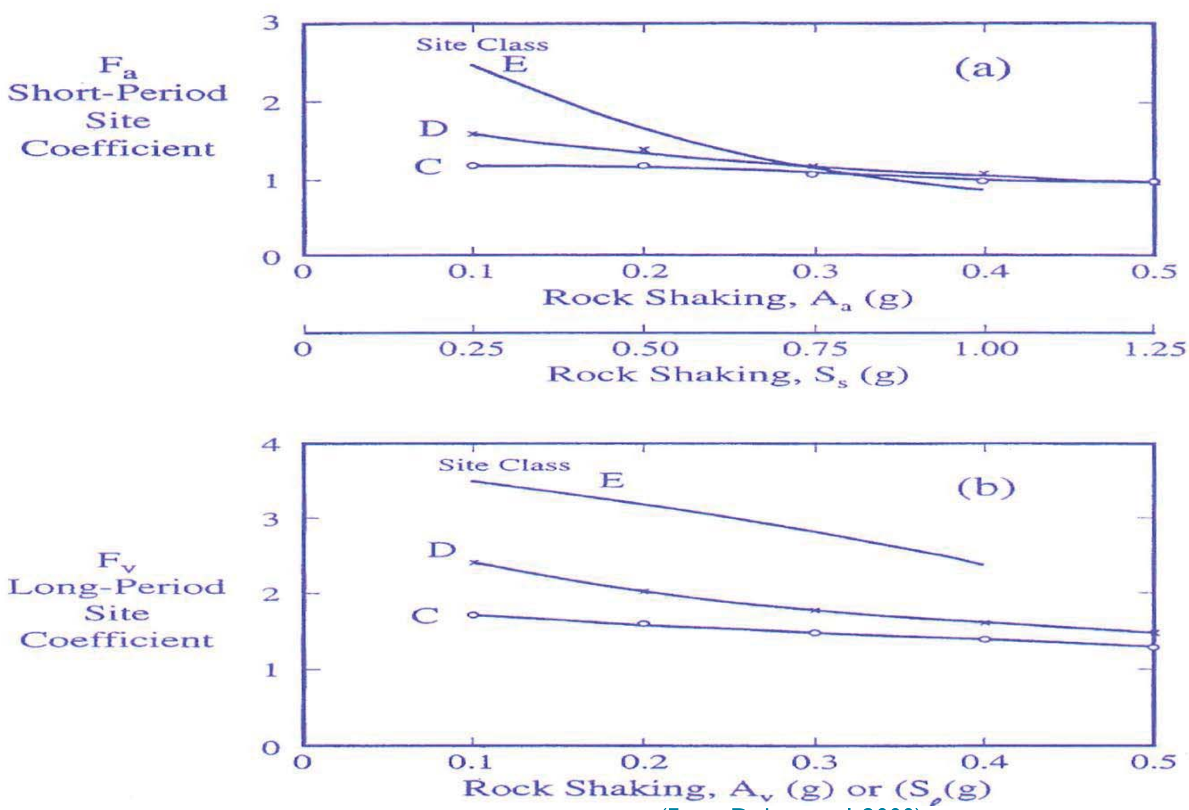




Additional Background (Geotechnical Considerations)



(From Dobry et al. 2000)



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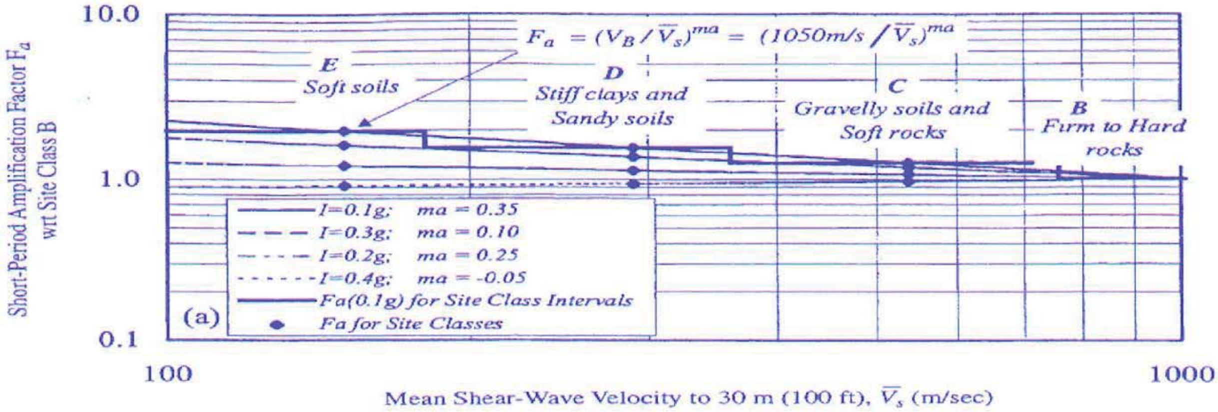
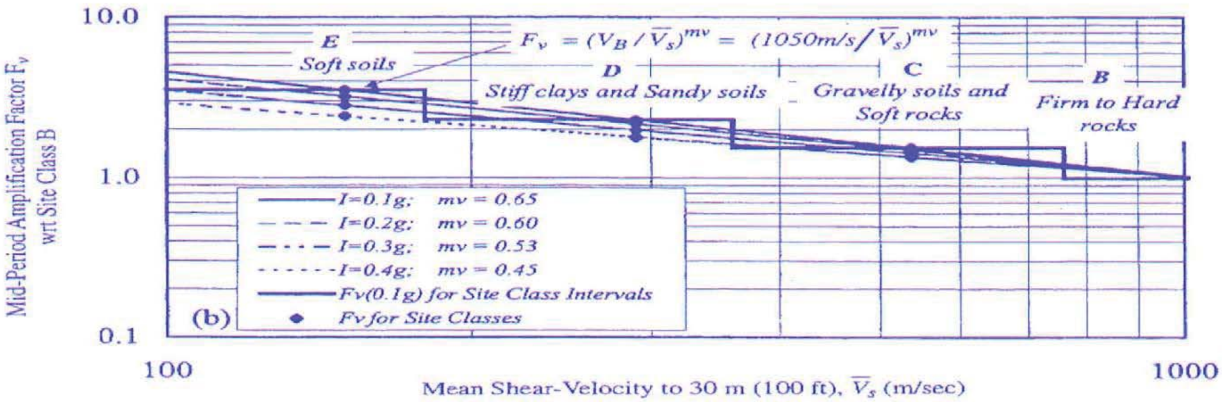
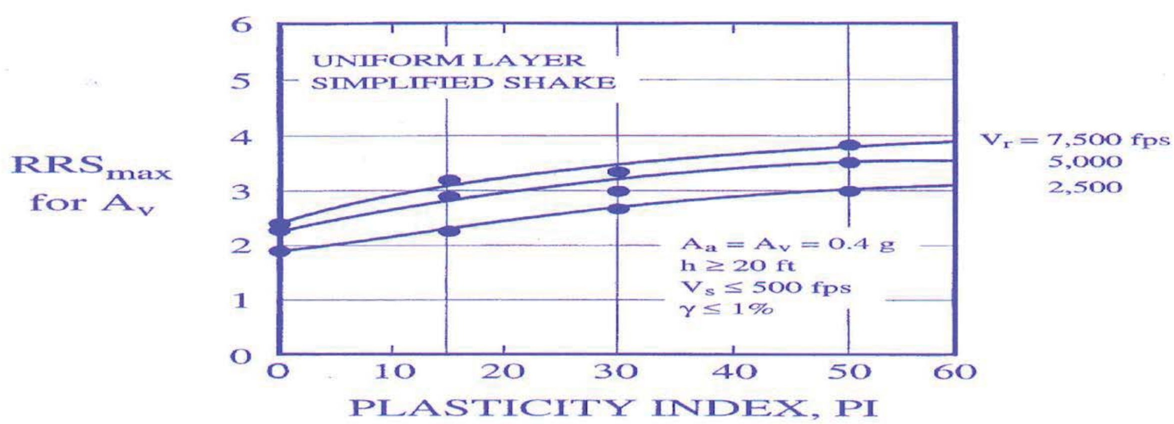
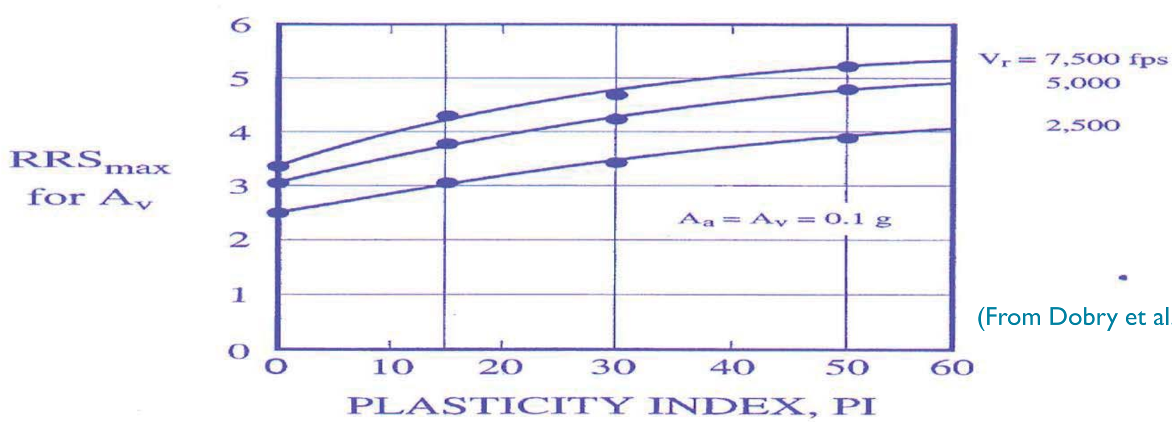




(From Dobry et al. 2000)





(From Dobry et al. 2000)

Ahmed Elgamal
Michael Fraser

International Building Code 2006 – Section 1613 Earthquake Loads

Note: see original reference (the 2006 IBC Code) for full details

The Code specifies that "Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7" (ASCE SEI 7-05, see Appendix 1). As such, **"buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable"** (i.e., the Seismic Code ultimately aims to avoid catastrophic collapse of the structure).

Base Shear per ASCE/SEI 7-05 EQUIVALENT LATERAL FORCE (ELF) Procedure

The following method follows the provisions of the ASCE/SEI 7-05 and may be utilized for determining the seismic base shear.

Calculation of the Seismic Base Shear, V - The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \tag{12.8-1}$$

where,

C_s is the seismic response coefficient, and W is the effective seismic weight.

Evaluation of Effective Seismic Weight, W – see Appendix 2

Determination of the Seismic Response Coefficient C_s – The seismic response coefficient, C_s, is defined by:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \tag{12.8-2}$$

where,

S_{DS} = design, 5 percent damped, spectral response acceleration parameter at short periods (Figure 1), *I* is the occupancy importance factor, as shown in Appendix 3, and

R is the response modification factor determined from Table 12.2-1 (see Appendix 4).

Check the value of C_s versus the Need-Not-Exceed Limits - The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following (see Figure 1):

For $T \le T_L$: $$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \qquad (12.8\text{-}3)$$

For $T > T_L$: $$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} \qquad (12.8\text{-}4)$$

where S_{D1} = design, 5 percent damped, spectral response acceleration parameter at a building Period *T* (see Appendix 5) of *1 second* (Figure 1).

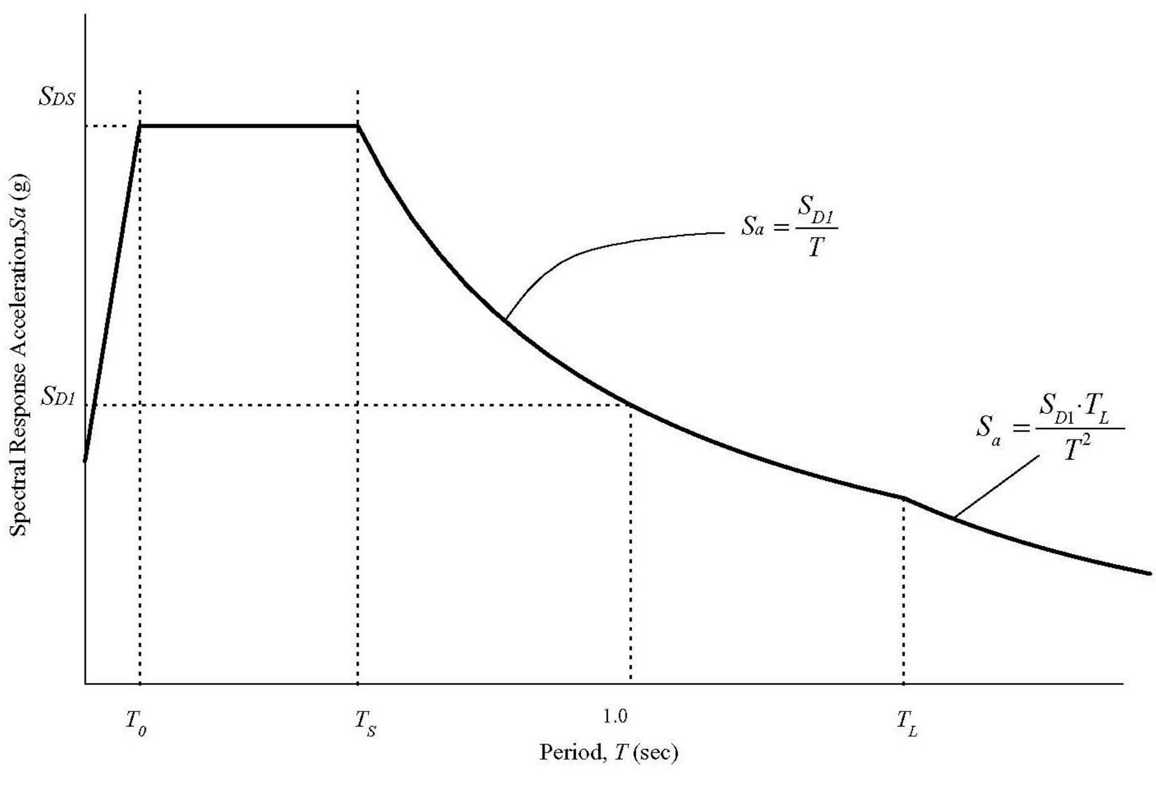


FIGURE 11.4-1 DESIGN RESPONSE SPECTRUM

Figure 1: Design Response Spectrum Configuration (from the 2006 IBC). Note: *$T_0 = 0.2(S_{D1}/S_{DS})$, and S_a at $T = 0$ is $S_{DS}/2.5$.*

In the above, $T_S = S_{D1} / S_{DS}$ and T_L is specified as described in Appendix 6.

In the Design Response Spectrum above, the spectral response acceleration segments are specified on the basis of geotechnical site amplification studies that show overall envelopes/averages that resemble the configuration depicted in Figure 1.

Check on minimum value of C_s - Cs shall not be less than:

$$C_s = 0.01 \tag{12.8-5}$$

In addition, for structures located where S_1 (the one-second period spectral acceleration, see Appendix 7) is equal to or greater than 0.6g, C_s shall not be less than (to implicitly account for near source effects):

$$C_s = \frac{0.5\, S_1}{\left(\frac{R}{I}\right)} \tag{12.8-6}$$

Determination of S_{DS} and S_{D1}

These spectral values are defined by the following expressions:

$$S_{DS} = \frac{2}{3} S_{MS} \tag{11.4-3}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{11.4-4}$$

where ,

M denotes Maximum Considered Earthquake (MCE), an expression that represents ground motions with a 2% probability of being exceeded in 50 years (average return period of approximately 2,500 years); with exceptions where deterministic estimates govern the MCE design motions in certain higher seismic regions near active faults (Crouse *et al.* 2006).

S_{MS} is the MCE, 5 percent damped, spectral response acceleration at short periods adjusted for Site Class effects, and

S_{M1} is the MCE, 5 percent damped, spectral response acceleration at a Period of 1.0 second adjusted for Site Class effects.

These spectral accelerations are defined by:

$S_{MS} = F_a S_s$ (11.4-1)

$S_{M1} = F_v S_1$ (11.4-2)

where the site-specific short period (S_s) and one-second period (S_1) spectral accelerations are defined following the procedures in Appendix 7, and the Site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2 below, respectively.

Table 11.4-1 Site Coefficient, F_a (see Appendix 8 for site Class)

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 (see Appendix 9)				

Table 11.4-2 Site Coefficient, F_v (see Appendix 8 for site Class)

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 (see Appendix 9)				

Vertical Distribution of Seismic Forces

The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined, in accordance with ASCE/SEI 7-05 Section 12.8.3, from the following equations:

$F_x = C_{vx} V$ (12.8-11)

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8\text{-}12)$$

where,

C_{vx} = vertical distribution factor,

w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x,

h_i and h_x = the height (ft or m) from the base to Level i or x,

n = total number of stories,

k = an exponent related to the structure period as follows:
for structures having a period of 0.5 s or less, k = 1,
for structures having a period of 2.5 s or more, k = 2,
for structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

Horizontal Distribution of Forces - The seismic design story shear in any story (V_x) (kip or kN) shall be determined, in accordance with ASCE/SEI 7-05 Section 12.8.4, from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \qquad (12.8\text{-}13)$$

where F_i is the portion of the seismic base shear (V) (kip or kN) induced at Level i.

Determination the Seismic Design Category For Material Design Sections of the Code – see Appendix 10

The Seismic Design Category classification is employed for additional code considerations as shown in Appendix 4.

References

C.B. Crouse, E.V. Leyendecker, P.G. Somerville, M. Power, and W.J. Silva, development of seismic ground-motion criteria for the ASCE 7 standard, Proceedings of the 8th U.S. National Conference on Earthquake Engineering April 18-22, 2006, San Francisco, California, USA, Paper No. 533).

Appendix 1 -- Excerpts from ASCE SEI 7-05

ASCE/SEI 7-05 Minimum Design Loads for Buildings and Other Structures

1.1 SCOPE - This standard provides minimum load requirements for the design of buildings and other structures that are subject to building code requirements. Loads and appropriate load combinations, which have been developed to be used together, are set forth for strength design and allowable stress design. For design strengths and allowable stress limits, design specifications for conventional structural materials used in buildings and modifications contained in this standard shall be followed.

1.3.2 Serviceability - Buildings and other structures, and all parts thereof, shall be designed and constructed to support safely the factored loads in load combinations defined in this document without exceeding the appropriate strength limit states for the materials of construction. Alternatively, buildings and other structures, and all parts thereof, shall be designed and constructed to support safely the nominal loads in load combinations defined in this document without exceeding the appropriate specified allowable stresses for the materials of construction.

1.4 GENERAL STRUCTURAL INTEGRITY - Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. This shall be achieved through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse. This shall be accomplished by providing sufficient continuity, redundancy, or energy-dissipating capacity (ductility), or a combination thereof, in the members of the structure.

C1.3 BASIC REQUIREMENTS
C1.3.1 Strength - Buildings and other structures must satisfy strength limit states in which members are proportioned to carry the design loads safely to resist buckling, yielding, fracture, and so forth. It is expected that other standards produced under consensus procedures and intended for use in connection with building code requirements will contain recommendations for resistance factors for strength design methods or allowable stresses (or safety factors) for allowable stress design methods.

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

Appendix 2

13. Calculate the Effective Seismic Weight, W

The effective seismic weight shall include the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the floor live load (floor live load in public garages and open parking structures need not be included).

2. Where provision for partitions is required by Section 4.2.2 in the floor load design, the actual partition weight or a minimum weight of 10 psf (0.48 kN/m^2) of floor area, whichever is greater.

3. Total operating weight of permanent equipment.

4. Where the flat roof snow load, P_f, exceeds 30 psf (1.44 kN/m^2), 20 percent of the uniform design snow load, regardless of actual roof slope.

Appendix 3 – *Determination of the Building's Importance Factor, I*

The Importance Factor *I* is a factor assigned to each structure according to its Occupancy Category. In turn, the Occupancy Category is defined as follows:

Building's Occupancy Category - Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 1-1 for the purposes of applying flood, wind, snow, earthquake, and ice provisions. The occupancy categories range from I to IV, where Occupancy Category I represents buildings and other structures with a low hazard to human life in the event of failure and Occupancy Category IV represents essential facilities. Each building or other structure shall be assigned to the highest applicable occupancy category or categories. Assignment of the same structure to multiple occupancy categories based on use and the type of load condition being evaluated (e.g., wind or seismic) shall be permissible.
OCCUPANCY: The purpose for which a building or other structure, or part thereof, is used or intended to be used.
The Occupancy Category is determined from Table 1-1 OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES FOR FLOOD, WIND, SNOW, EARTHQUAKE, AND ICE LOADS.

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with daycare facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations[a] • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. Buildings and other structures containing toxic or explosive substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the toxic or explosive substances does not pose a threat to the public.	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency • Aviation control towers, air traffic control centers, and emergency aircraft hangars • Water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. Buildings and other structures containing highly toxic substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

[a]Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.

On this basis, the importance factor, *I*, shall be assigned to each structure by:

Occupancy Category	Importance Factor, I
I	1.0
II	1.0
III	1.25
IV	1.5

Note:
The NEHRP 1997 Provisions in Section 1.1, identifies two purposes of the Occupancy Importance Factor, one of which specifically is to "improve the capability of essential facilities and structures containing substantial quantities of hazardous materials to function during and after design earthquakes." This is achieved by introducing the occupancy importance factor of 1.25 for Seismic Use Group III structures and 1.5 for Seismic Use Group IV structures. The NEHRP Commentary Sections 1.4, 5.2, and 5.2.8 explain that the factor is intended to reduce the ductility demands and result in less damage. When combined with the more stringent drift limits for such essential or hazardous facilities the result is improved performance of such facilities.

Appendix 4 -- *R* factor; and Seismic Design Category (see Appendix 10)

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	2½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	6½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	2½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.1.4.2, and 14.5	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	14.1	8	2	4	NL	NL	160	160	100
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	14.1	7	2	4	NL	NL	160	160	100
3. Special steel concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
4. Ordinary steel concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
5. Special reinforced concrete shear walls	14.2 and 14.2.3.6	6	2½	5	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	5	2½	4½	NL	NL	NP	NP	NP
7. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP	NP	NP
8. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP	NP	NP
9. Intermediate precast shear walls	14.2 and 14.2.3.5	5	2½	4½	NL	NL	40[k]	40[k]	40[k]
10. Ordinary precast shear walls	14.2 and 14.2.3.3	4	2½	4	NL	NP	NP	NP	NP
11. Composite steel and concrete eccentrically braced frames	14.3	8	2	4	NL	NL	160	160	100
12. Composite steel and concrete concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
13. Ordinary composite steel and concrete braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
14. Composite steel plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
15. Special composite reinforced concrete shear walls with steel elements	14.3	6	2½	5	NL	NL	160	160	100
16. Ordinary composite reinforced concrete shear walls with steel elements	14.3	5	2½	4½	NL	NL	NP	NP	NP
17. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
18. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
19. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
20. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
21. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP

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TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS (continued)

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c] Seismic Design Category: B	C	D^d	E^d	F^e
22. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	7	2½	4½	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	14.1	7	2	5½	NL	NL	160	160	100
26. Buckling-restrained braced frames, moment-resisting beam-column connections	14.1	8	2½	5	NL	NL	160	160	100
27. Special steel plate shear wall	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Special steel moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h,i]	NP[h]	NP[i]
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP[h]	NP[h]	NP[i]
5. Special reinforced concrete moment frames	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Ordinary composite moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Special steel concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Composite steel and concrete eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Composite steel and concrete concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
7. Composite steel plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Special composite reinforced concrete shear walls with steel elements	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Ordinary composite reinforced concrete shear walls with steel elements	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Buckling-restrained braced frame	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Special steel plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS (continued)

Seismic Force-Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Special steel concentrically braced frames[f]	14.1	6	2½	5	NL	NL	35	NP	NP[h,k]
2. Special reinforced concrete shear walls	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP
5. Composite steel and concrete concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Ordinary composite braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Ordinary composite reinforced concrete shear walls with steel elements	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	12.2.5.10 and 14.2	4½	2½	4	NL	NP	NP	NP	NP
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Special steel moment frames	12.2.5.5 and 14.1	2½	1¼	2½	35	35	35	35	35
2. Intermediate steel moment frames	14.1	1½	1¼	1½	35	35	35[h]	NP[h,i]	NP[h,i]
3. Ordinary steel moment frames	14.1	1¼	1¼	1¼	35	35	NP	NP[h,i]	NP[h,i]
4. Special reinforced concrete moment frames	12.2.5.5 and 14.2	2½	1¼	2½	35	35	35	35	35
5. Intermediate concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP
6. Ordinary concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
7. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	14.1	3	3	3	NL	NL	NP	NP	NP

[a] Response modification coefficient, R, for use throughout the standard. Note R reduces forces to a strength level, not an allowable stress level.
[b] Reflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, and 12.9.2
[c] NL = Not Limited and NP = Not Permitted. For metric units use 30.5 m for 100 ft and use 48.8 m for 160 ft. Heights are measured from the base of the structure as defined in Section 11.2.
[d] See Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.
[e] See Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.
[f] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.
[g] The tabulated value of the overstrength factor, Ω_0, is permitted to be reduced by subtracting one-half for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.
[h] See Sections 12.2.5.6 and 12.2.5.7 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category D or E.
[i] See Sections 12.2.5.8 and 12.2.5.9 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category F.
[j] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 psf (0.96 kN/m^2) and in penthouse structures.
[k] Increase in height to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.

In this table, the overstrength factor, Ω_0, dictates increase in seismic load for the critical structural elements that would be expected to remain in the elastic state (e.g., to prevent collapse), and the deflection amplification factor, C_d dictates increases to the code-derived deflections, to be representative of the actual expected peak deflection values (since the Code specifies much lower forces to calculate building shear forces).

Determination of Design Coefficients and Factors for Building's Seismic Force-Resisting System(s)

The structural system used shall be in accordance with the Seismic Design Category and height limitations indicated in Table 12.2-1 above. The appropriate response modification coefficient, R, system overstrength factor, Ω_0, and the deflection amplification factor, C_d, indicated in Table 12.2-1 shall be used in determining the base shear, element design forces, and design story drift.

Different seismic force–resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R, C_d, and Ω_0 coefficients shall apply to each system, including the limitations on system use contained in Table 12.2-1 above.

Where different seismic force–resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as dual systems, the more stringent system limitation contained in Table 12.2-1 shall apply and the design shall comply with the requirements of this section.

Appendix 5

Determination of the Approximate Fundamental Period of Structure – The approximate fundamental period (T_a), in seconds, shall be determined from the following equation:

$$T_a = C_t h_n^x \qquad (12.8\text{-}7)$$

where h_n is the height in ft above the base to the highest level of the structure and the period coefficients C_t and x are determined from Table 12.8-2.

TABLE 12.8-2 VALUES OF APPROXIMATE PERIOD PARAMETERS C_t AND x

Structural Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.28 (0.0724)*	0.8
Concrete moment-resisting frames	0.016 (0.0466)*	0.9
Eccentrically braced steel frames	0.03 (0.0731)*	0.75
All other structural systems	0.02 (0.0488)*	0.75

* Metric equivalents are shown in parentheses

For structures not exceeding 12 stories in height in which the seismic force–resisting system consists entirely of concrete or steel moment resisting frames and the story height is at least 10 ft, it is permitted to determine the approximate fundamental period (Ta), in s, from the following equation:

$$T_a = 0.1\ N \qquad (12.8\text{-}8)$$

where N = number of stories.

Appendix 6 -- Map for obtaining T_L (please see sample map below)

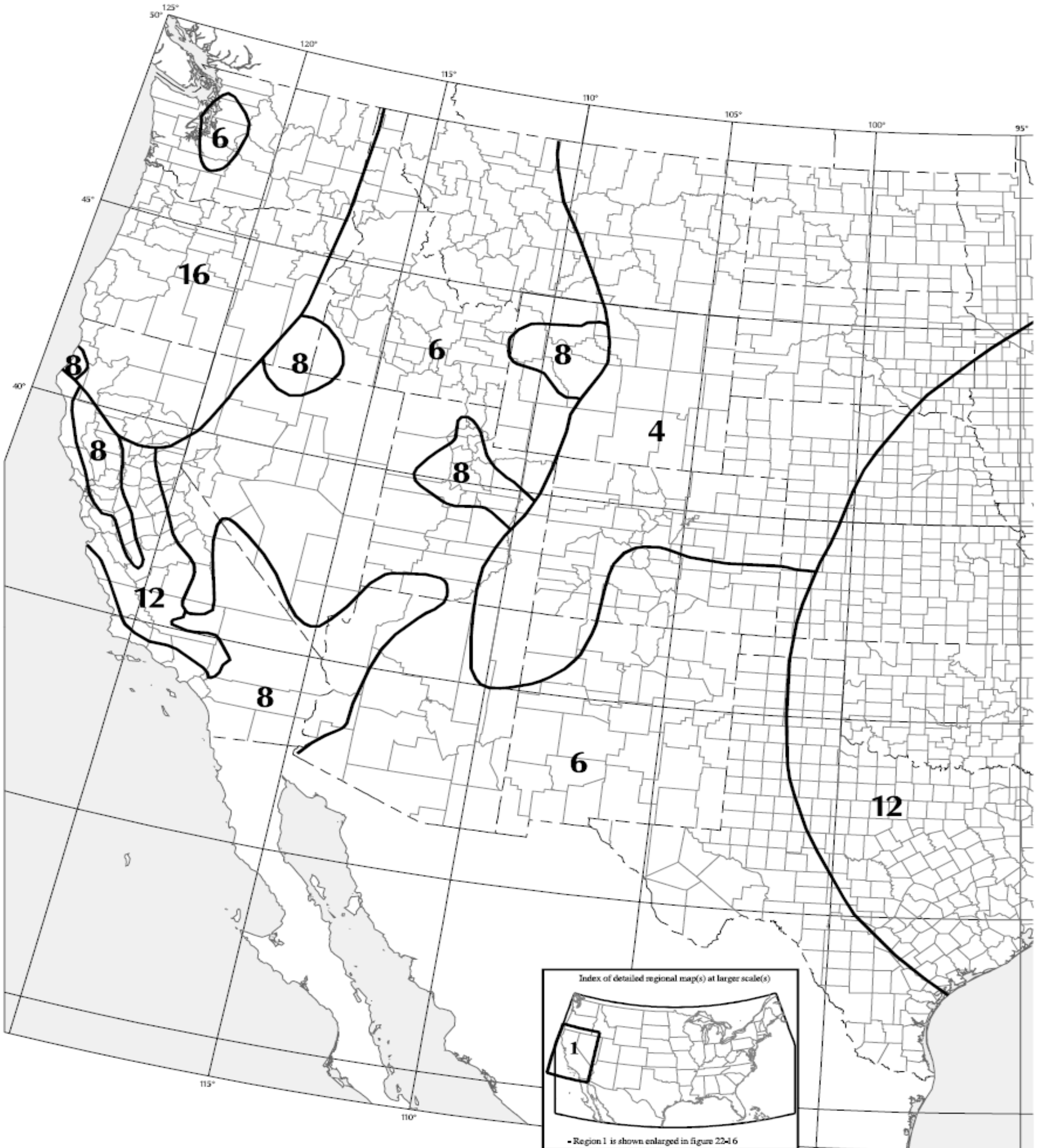

FIGURE 22-15 LONG-PERIOD TRANSITION PERIOD, T_L (SEC), FOR THE CONTERMINOUS UNITED STATES

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Appendix 7

Determination of S_S and S_1 (site-specific short period and one-second period spectral accelerations)

S_S = mapped MCE, 5 percent damped, spectral response acceleration at short periods (0.2 seconds).

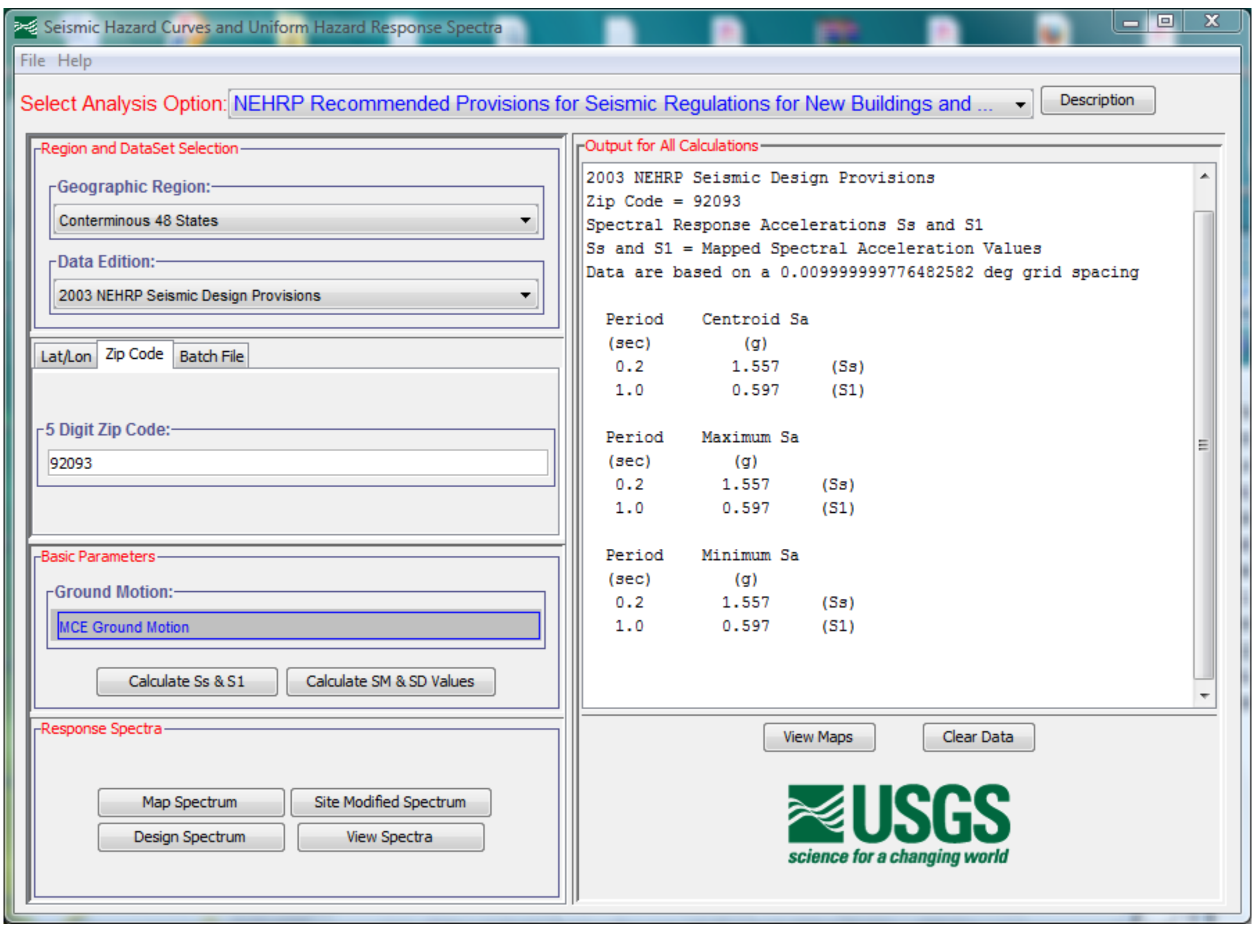
S_1 = mapped MCE, 5 percent damped, spectral response acceleration at a period of 1 second.

The above parameters are readily available for Site Class B (see Site Classes in Appendix 8):

For values over the Internet, go to:

http://earthquake.usgs.gov/hazards/design/

http://earthquake.usgs.gov/hazards/design/buildings.php



Conversely, maps (see samples below) are available in the Code (Figs. 22-1 through 22-14, see sample maps on the following pages).

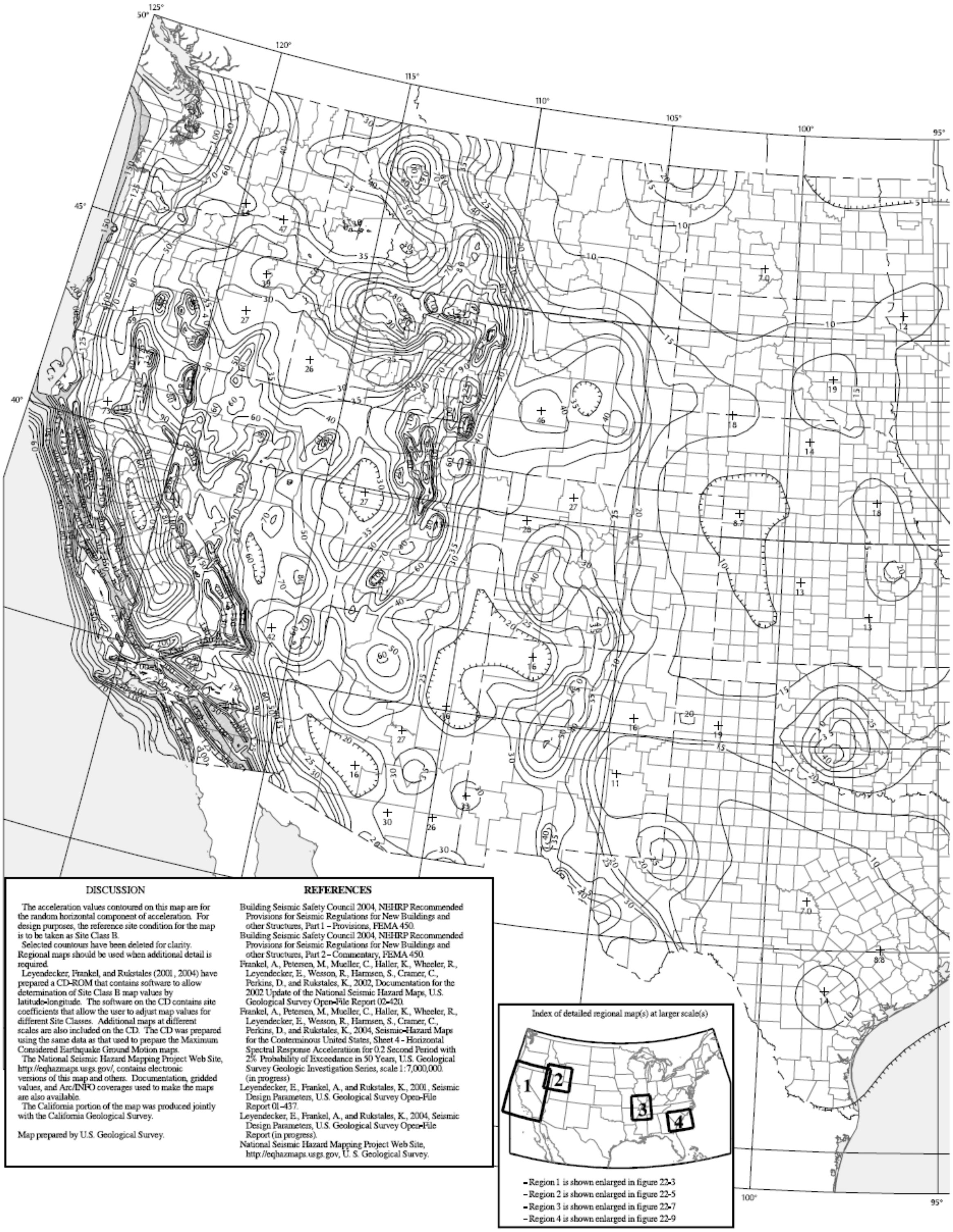


FIGURE 22-1 MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

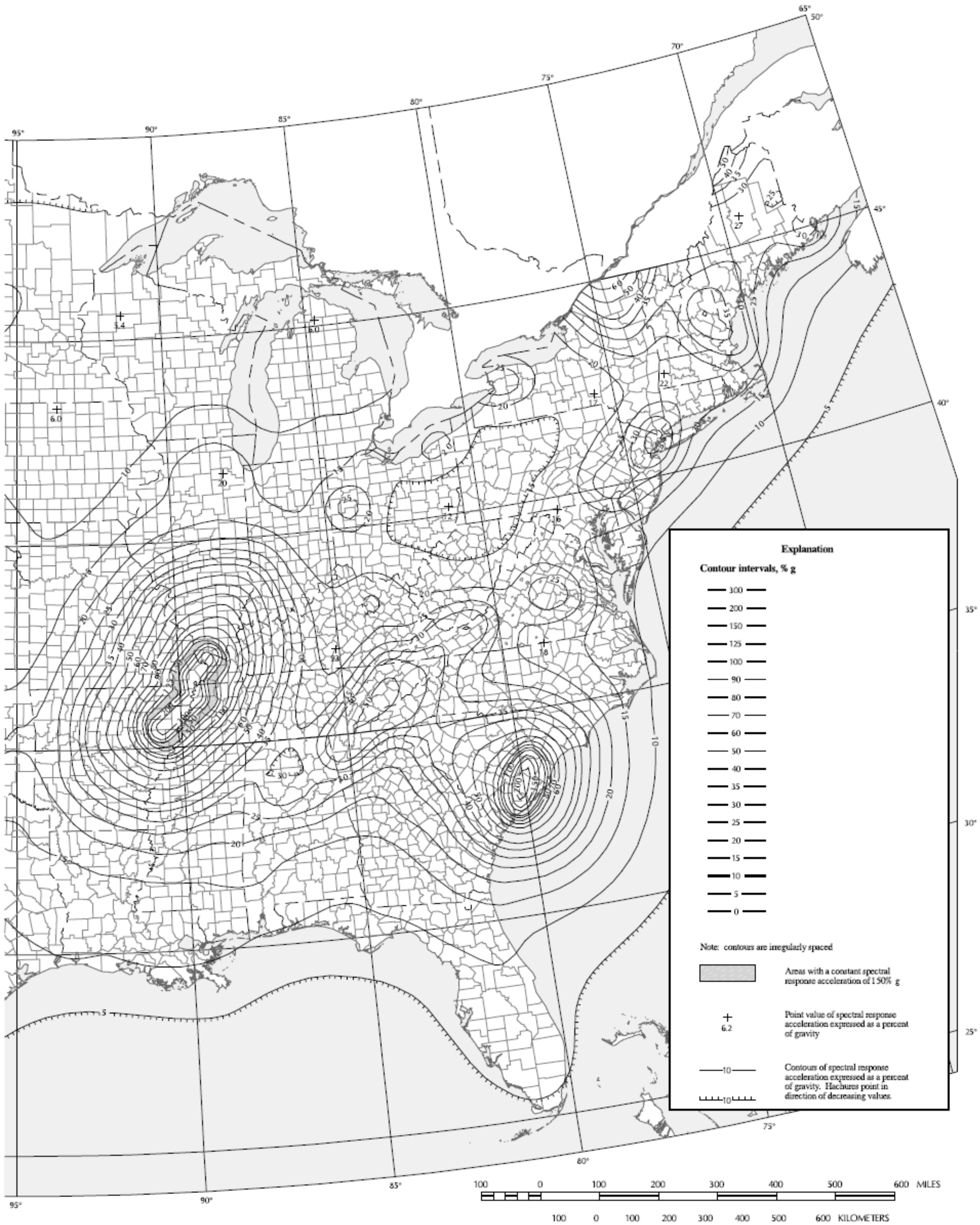


FIGURE 22-1 continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

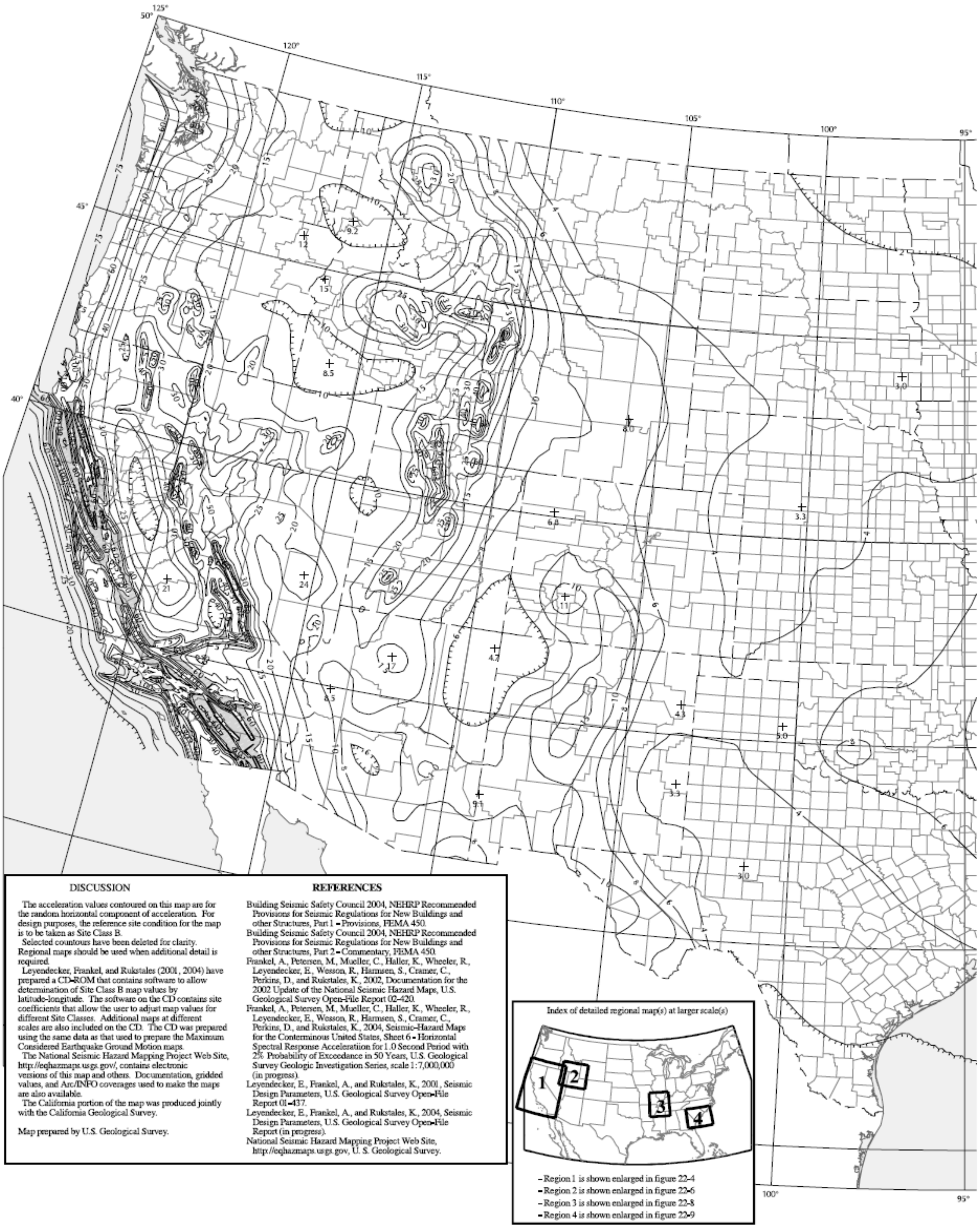


FIGURE 22-2 MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

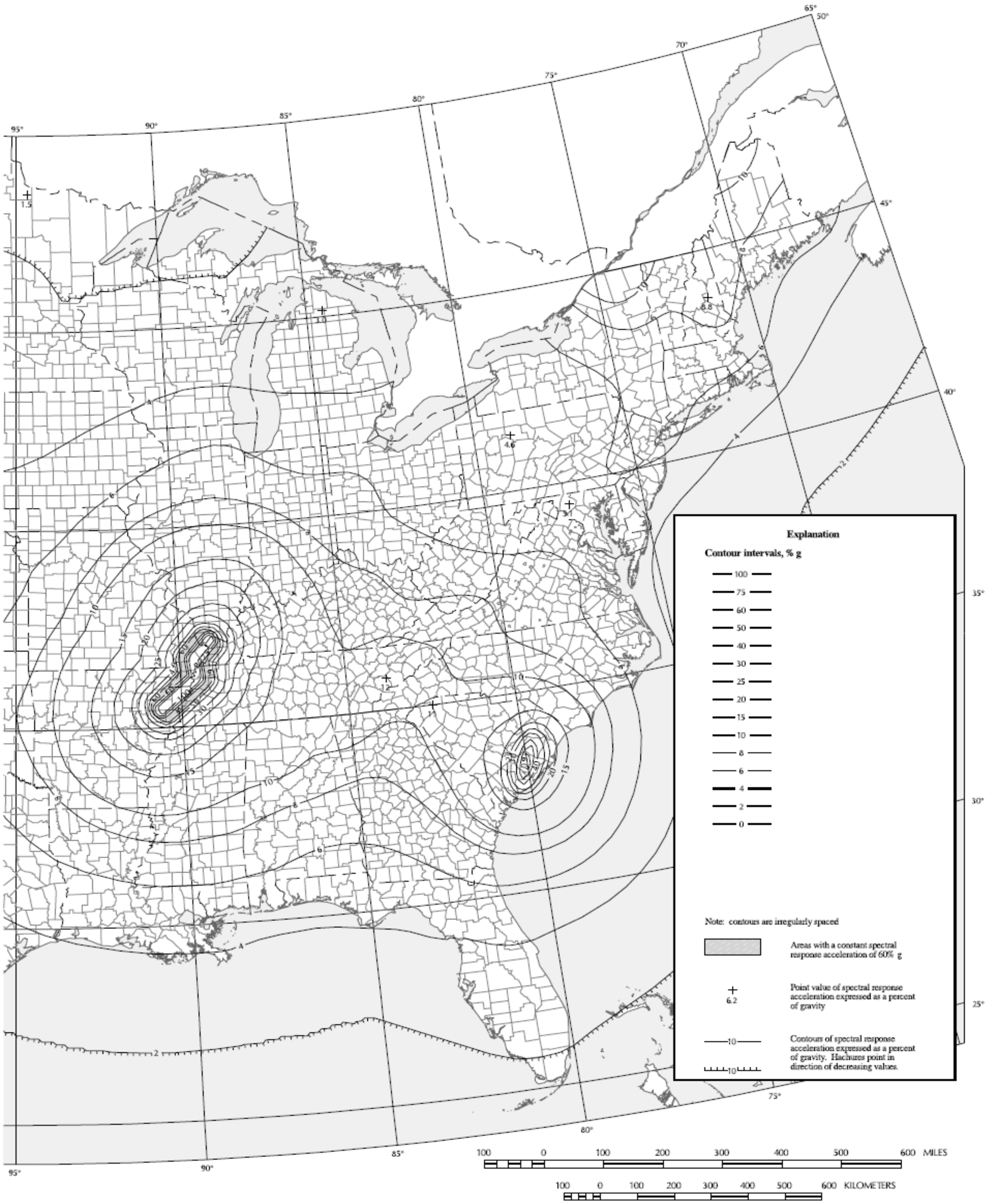


FIGURE 22-2 continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Appendix 8 -- Site Class

5. Determination of Site Classification - Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. The site soil shall be classified in accordance with Table 20.3-1 and Section 20.3 based on the upper 100 ft (30 m) of the site profile. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site. Site Classes A and B shall not be assigned to a site if there is more than 10 ft of soil between the rock surface and the bottom of the spread footing or mat foundation.

TABLE 20.3-1 SITE CLASSIFICATION

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: - Plasticity index PI > 20, - Moisture content $w \geq 40\%$, and - Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s 1 lb/ft^2 = 0.0479 kN/m^2

$\bar{v}_s$ is the average shear wave velocity in the upper 100 ft of the site profile and is calculated as:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad (20.4\text{-}1)$$

d_i is the thickness of any layer between 0 and 100 ft (30 m).
v_{si} is the shear wave velocity in ft/s (m/s).

$\tilde{N}$ Average Field Standard Penetration Resistance and $\tilde{N}_{ch}$, Average Standard Penetration Resistance for Cohesionless Soil Layers. $\tilde{N}$ and $\tilde{N}_{ch}$ shall be determined in accordance with the following formulas:

$$\tilde{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad (20.4\text{-}2)$$

$$\tilde{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \tag{20.4-3}$$

where N_i and d_i in Eq. 20.4-3 are for cohesionless soil layers only and $\sum_{i=1}^{m} d_i = d_s$ where d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m). N_i is the standard penetration resistance (ASTM D1586) not to exceed 100 blows/ft (328 blows/m) as directly measured in the field without corrections. Where refusal is met for a rock layer, N_i shall be taken as 100 blows/ft (328 blows/m).

$\bar{s}_u$ is the Average Undrained Shear Strength. $\bar{s}_u$ shall be determined in accordance with the following formula:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \tag{20.4-4}$$

where

$\sum_{i=1}^{k} d_i = d_c$ and

d_c = the total thickness of cohesive soil layers in the top 100 ft (30 m)

PI = the plasticity index as determined in accordance with ASTM D4318

w = the moisture content in percent as determined in accordance with ASTM D2216

s_{ui} = the undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa) as determined in accordance with ASTM D2166 or ASTM D2850

Appendix 9 -- Site Specific Procedure

Chapter 21

SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

21.1 SITE RESPONSE ANALYSIS

The requirements of Section 21.1 shall be satisfied where site response analysis is performed or required by Section 11.4.7. The analysis shall be documented in a report.

21.1.1 Base Ground Motions. A maximum considered earthquake (MCE) response spectrum shall be developed for bedrock, using the procedure of Sections 11.4.6 or 21.2. Unless a site-specific ground motion hazard analysis described in Section 21.2 is carried out, the MCE rock response spectrum shall be developed using the procedure of Section 11.4.6 assuming Site Class B. If bedrock consists of Site Class A, the spectrum shall be adjusted using the site coefficients in Section 11.4.3 unless other site coefficients can be justified. At least five recorded or simulated horizontal ground motion acceleration time histories shall be selected from events having magnitudes and fault distances that are consistent with those that control the MCE. Each selected time history shall be scaled so that its response spectrum is, on average, approximately at the level of the MCE rock response spectrum over the period range of significance to structural response.

21.1.2 Site Condition Modeling. A site response model based on low-strain shear wave velocities, nonlinear or equivalent linear shear stress-strain relationships, and unit weights shall be developed. Low-strain shear wave velocities shall be determined from field measurements at the site or from measurements from similar soils in the site vicinity. Nonlinear or equivalent linear shear stress-strain relationships and unit weights shall be selected on the basis of laboratory tests or published relationships for similar soils. The uncertainties in soil properties shall be estimated. Where very deep soil profiles make the development of a soil model to bedrock impractical, the model is permitted to be terminated where the soil stiffness is at least as great as the values used to define Site Class D in Chapter 20. In such cases, the MCE response spectrum and acceleration time histories of the base motion developed in Section 21.1.1 shall be adjusted upward using site coefficients in Section 11.4.3 consistent with the classification of the soils at the profile base.

21.1.3 Site Response Analysis and Computed Results. Base ground motion time histories shall be input to the soil profile as outcropping motions. Using appropriate computational techniques that treat nonlinear soil properties in a nonlinear or equivalent-linear manner, the response of the soil profile shall be determined and surface ground motion time histories shall be calculated. Ratios of 5 percent damped response spectra of surface ground motions to input base ground motions shall be calculated. The recommended surface MCE ground motion response spectrum shall not be lower than the MCE response spectrum of the base motion multiplied by the average surface-to-base response spectral ratios (calculated period by period) obtained from the site response analyses. The recommended surface ground motions that result from the analysis shall reflect consideration of sensitivity of response to uncertainty in soil properties, depth of soil model, and input motions.

21.2 GROUND MOTION HAZARD ANALYSIS

The requirements of Section 21.2 shall be satisfied where a ground motion hazard analysis is performed or required by Section 11.4.7. The ground motion hazard analysis shall account for the regional tectonic setting, geology, and seismicity, the expected recurrence rates and maximum magnitudes of earthquakes on known faults and source zones, the characteristics of ground motion attenuation, near source effects, if any, on ground motions, and the effects of subsurface site conditions on ground motions. The characteristics of subsurface site conditions shall be considered either using attenuation relations that represent regional and local geology or in accordance with Section 21.1. The analysis shall incorporate current seismic interpretations, including uncertainties for models and parameter values for seismic sources and ground motions. The analysis shall be documented in a report.

21.2.1 Probabilistic MCE. The probabilistic MCE spectral response accelerations shall be taken as the spectral response accelerations represented by a 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance within a 50-yr. period.

21.2.2 Deterministic MCE. The deterministic MCE response acceleration at each period shall be calculated as 150 percent of the largest median 5 percent damped spectral response acceleration computed at that period for characteristic earthquakes on all known active faults within the region. For the purposes of this standard, the ordinates of the deterministic MCE ground motion response spectrum shall not be taken lower than the corresponding ordinates of the response spectrum determined in accordance with Fig. 21.2-1, where F_a and F_v are determined using Tables 11.4-1 and 11.4-2, respectively, with the value of S_S taken as 1.5 and the value of S_1 taken as 0.6.

21.2.3 Site-Specific MCE. The site-specific MCE spectral response acceleration at any period, S_{aM}, shall be taken as the

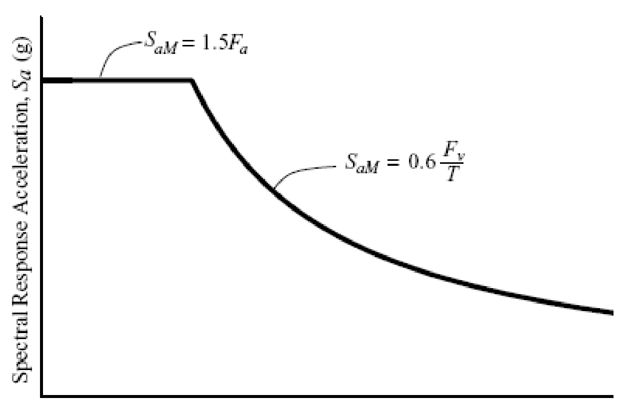


FIGURE 21.2-1 DETERMINISTIC LOWER LIMIT ON MCE RESPONSE SPECTRUM

lesser of the spectral response accelerations from the probabilistic MCE of Section 21.2.1 and the deterministic MCE of Section 21.2.2.

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 less than 80 percent of S_a determined in accordance with Section 11.4.5. For sites classified as Site Class F requiring site response analysis in accordance with Section 11.4.7, the design spectral response acceleration at any period shall not be taken less than 80 percent of S_a determined for Site Class E in accordance with Section 11.4.5.

21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as the spectral acceleration, S_a, obtained from the site-specific spectra at a period of 0.2 s, except that it shall not be taken less than 90 percent of the peak spectral acceleration, S_a, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the spectral acceleration, S_a, at a period of 1 s or two times the spectral acceleration, S_a, at a period of 2 sec. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80 percent of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.4 for S_{DS} and S_{D1}.

Note: Material below is from the 2003 Seismic Code Provisions

3.5 SITE CLASSIFICATION FOR SEISMIC DESIGN

Where the soil properties are not known in sufficient detail to determine the Site Class in accordance with Sec. 3.5.1, it shall be permitted to assume Site Class D unless the authority having jurisdiction determines that Site Class E or F could apply at the site or in the event that Site Class E or F is established by geotechnical data.

3.5.1 Site Class definitions. The Site Classes are defined as follows:

A Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/sec (1500 m/s)

B Rock with 2,500 ft/sec $< \bar{v}_s \leq$ 5,000 ft/sec (760 m/s $< \bar{v}_s \leq$ 1500 m/s)

C Very dense soil and soft rock with 1,200 ft/sec $< \bar{v}_s \leq$ 2,500 ft/sec (360 m/s $< \bar{v}_s \leq$ 760 m/s) or with either $\bar{N} > 50$ or $\bar{s}_u > 2{,}000$ psf (100 kPa)

D Stiff soil with 600 ft/sec $\leq \bar{v}_s \leq$ 1,200 ft/sec (180 m/s $\leq \bar{v}_s \leq$ 360 m/s) or with either $15 \leq \bar{N} \leq 50$ or 1,000 psf $\leq \bar{s}_u \leq$ 2,000 psf (50 kPa $\leq \bar{s}_u \leq$ 100 kPa)

E A soil profile with $\bar{v}_s <$ 600 ft/sec (180 m/s) or with either

$\bar{N} < 15$, $\bar{s}_u < 1{,}000$ psf, or any profile with more than 10 ft (3 m) of soft clay defined as soil with $PI > 20$, $w \geq$ 40 percent, and $s_u <$ 500 psf (25 kPa)

F Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

   Exception: For structures having fundamental periods of vibration less than or equal to 0.5 second, site-specific evaluations are not required to determine spectral accelerations for liquefiable soils. Rather, the Site Class may be determined in accordance with Sec. 3.5.2, assuming liquefaction does not occur, and the corresponding values of F_a and F_v determined from Tables 3.3-1 and 3.3-2.

2. Peat and/or highly organic clays ($H >$ 10 ft [3 m] of peat and/or highly organic clay, where H = thickness of soil)

3. Very high plasticity clays ($H > 25$ ft [8 m] with $PI > 75$)
4. Very thick, soft/medium stiff clays ($H > 120$ ft [36 m]) with $s_u < 1{,}000$ psf (50 kPa)

The parameters used to define the Site Class are based on the upper 100 ft (30 m) of the site profile. Profiles containing distinctly different soil and rock layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft (30 m). The symbol i then refers to any one of the layers between 1 and n.

where:

v_{si} = the shear wave velocity in ft/sec (m/s).

d_i = the *thickness* of any layer (between 0 and 100 ft [30 m]).

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \tag{3.5-1}$$

where $\sum_{i=1}^{n} d_i$ is equal to 100 ft (30 m).

N_i = the Standard Penetration Resistance determined in accordance with ASTM D 1586, as directly measured in the field without corrections, and shall not be taken greater than 100 blows/ft. Where refusal is met for a rock layer, N_i shall be taken as 100 blows/ft.

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \tag{3.5-2}$$

where N_i and d_i in Eq. 3.5-2 are for cohesionless soil, cohesive soil, and rock layers.

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \tag{3.5-3}$$

where N_i and d_i in Eq. 3.5-3 are for cohesionless soil layers only,

and $\sum_{i=1}^{m} d_i = d_s$

d_s = the total thickness of cohesionless soil layers in the top 100 ft (30 m).

s_{ui} = the undrained shear strength in psf (kPa), determined in accordance with ASTM D 2166 or D 2850, and shall not be taken greater than 5,000 psf (250 kPa).

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \tag{3.5-4}$$

where $\sum_{i=1}^{k} d_i = d_c$.

d_c = the total thickness of cohesive soil layers in the top 100 ft (30 m).

PI = the plasticity index, determined in accordance with ASTM D 4318.

w = the moisture content in percent, determined in accordance with ASTM D 2216.

3.5.2 Steps for classifying a site

Step 1: Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by: $s_u < 500$ psf (25 kPa), $w \geq 40$ percent, and $PI > 20$. If these criteria are satisfied, classify the site as Site Class E.

Step 3: Categorize the site using one of the following three methods with $\bar{v}_s$, $\bar{N}$ and $\bar{s}_u$ computed in all cases as specified in Sec. 3.5.1:

a. $\bar{v}_s$ for the top 100 ft (30 m) ($\bar{v}_s$ method)

b. $\bar{N}$ for the top 100 ft (30 m) ($\bar{N}$ method)

c. $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft (30 m) and average $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft (30 m) ($\bar{s}_u$ method)

Table 3.5-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$ [a]
E	< 600 fps (< 180 m/s)	< 15	< 1,000 psf (< 50 kPa)
D	600 to 1,200 fps (180 to 360 m/s)	15 to 50	1,000 to 2,000 psf (50 to 100 kPa)
C	> 1,200 to 2,500 fps (360 to 760 m/s)	> 50	> 2,000 (> 100 kPa)

[a] If the $\bar{s}_u$ method is used and the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

Assignment of Site Class B shall be based on the shear wave velocity for rock. For competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the site shall be assigned to Site Class C.

Assignment of Site Class A shall be supported by either shear wave velocity measurements on site or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft (30 m), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

Site Classes A and B shall not be used where there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

Appendix 10

Determination of the Seismic Design Category

The Seismic Design Category is a classification assigned to a structure based on its Occupancy Category (Appendix 3) and the severity of the design earthquake ground motion at the site. Seismic Design Category is an important parameter in the material design sections of the Code (Appendix 4).

Occupancy Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 g (Appendix 7) shall be assigned to Seismic Design Category E.

Occupancy Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_1 (Appendix 7), is greater than or equal to 0.75 g shall be assigned to Seismic Design Category F.

All other structures shall be assigned to a Seismic Design Category based on their Occupancy Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}.

Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective of the fundamental period of vibration of the structure, T (Appendix 5).

A geotechnical investigation report shall be provided for a structure assigned to Seismic Design Category C, D, E, or F.

TABLE 11.6-1 SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \le S_{DS} < 0.33$	B	B	C
$0.33 \le S_{DS} < 0.50$	C	C	D
$0.50 \le S_{DS}$	D	D	D

TABLE 11.6-2 SEISMIC DESIGN CATEGORY BASED ON 1-S PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \le S_{D1} < 0.133$	B	B	C
$0.133 \le S_{D1} < 0.20$	C	C	D
$0.20 \le S_{D1}$	D	D	D