CONSIDERATIONS OF SITE RESPONSE IN U. S. BUILDING CODE PROVISIONS

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Abstract: Current provisions to account for local site-response in U.S. Building Code Provisions were adopted in 1994. No significant changes have occurred in these provisions since that time. These provisions provide new unambiguous definitions of seismically distinct site classes in terms of shear wave velocity and new amplitude-dependent site coefficients as a function of shear velocity and site class. The provisions are based on strong-motion measurements of the 1989 Loma Prieta, CA earthquake, numerical modeling, and a rather extensive set of borehole geotechnical data. Subsequent earthquakes have provided a significant increase in the number of strong-motion recordings at base accelerations in the 0.2-0.5 g range and corresponding borehole shear wave velocity measurements. Analyses of these data and subsequent research by several researchers resulted in a consensus that changes in the 2003 edition of the NEHRP building code provisions as adopted by the US National Provision Update Committee was not warranted. This paper will summarize the current site-response provisions in US Building Codes and summarize the basis for this recommendation.

1. INTRODUCTION

Exaggerated damage on soft soil deposits from the 1985 Mexico City earthquake, the 1988 Spitak, Armenia earthquake and the 1989 Loma Prieta earthquake emphasized the need to revise site response provisions in building codes adopted for use in the United States. Subsequent efforts resulted in a new set of provisions to account for the amplification effects of local soil deposits. These new provisions were first adopted in the 1994 Edition of the NEHRP (National Earthquake Hazard Reduction Program) “Recommended Provisions for Seismic Regulations for New Buildings and other Structures” and subsequently adopted in the 1997 and 2000 editions (NEHRP, 1994, 1997, 2000). They have been adopted with no significant changes in the 1997 and 2000 editions of the Uniform Building Code (UBC) and the 2000 edition of the International Building Code (IBC, 2000). Recently, they also have been adopted for the proposed 2003 edition of the NEHRP provisions and the new ASCE 7 2003 edition. The IBC and ASCE 7 represent a consensus effort to standardize seismic code provisions in the United States.

Reviews by several investigators of the theoretical and empirical evidence for the site coefficients as adopted in the 2000 editions of US seismic code provisions indicated that evidence available in 2002 confirmed provisions as written and that no change proposals were warranted for the 2003 NEHRP edition. This manuscript summarizes procedures and commentary as adopted in the 2003 NEHRP provisions. The manuscript reproduces pertinent NEHRP code provision material for readers not familiar with the provisions. A brief summary is provided of recent results derived by other investigators and their implications for change.

2. SITE-SPECIFIC DESIGN SPECTRA IN US BUILDING CODE PROVISIONS

In general, site-specific design spectra are specified in US building code provisions for each period as the product of an input ground motion level and an appropriate site coefficient intended to account for the amplification effects of local site conditions. The site coefficients are specified for a short-period and a long-period band as a function of site conditions and input ground motion level. The site conditions are specified in terms of site classes. The site classes are defined in terms of a minimum thickness and an estimate of shear velocity to 30 meters of the near-surface material. The input ground motion levels are specified from seismic design maps termed "Maximum Considered Earthquake Maps" developed from national probabilistic seismic hazard maps for a uniform ground condition of firm to hard rock. Details of the procedure are presented in subsequent sections.

2.1 Site Class Definitions

The seismic response of near surface deposits depends strongly on the geological character of the ground with the amplification effects of soft soil deposits at some periods being significantly larger than incoming base rock motions, especially if local site response resonances develop. Site classes are used in US building code provisions to define categories of geologic units with distinct seismic response characteristics. The classes are defined in terms of shear velocity to a depth of 30 m, denoted by $\overline{v}_s$ (Borcherdt, 1992, 1994). For sites underlain by soils for which no measurements of $\overline{v}_s$ to 30 m are feasible corresponding limits in terms of standard penetration resistance ($\overline{N}_{ch}$) and undrained shear strength ($\overline{s}_u$) have been added to facilitate identification of the site classes.

The site classes in US building code provisions (http://www.bssconline.org/) are defined as follows:

- A Hard rock with measured shear wave velocity, $\overline{v}_s$ > 5,000 ft/sec (1500 m/s).
- B Rock with 2,500 ft/sec < $\overline{v}_s$ < 5,000 ft/sec (760 m/s < $\overline{v}_s$ < 1500 m/s),
- C Very dense soil and soft rock with 1,200 ft/sec < $\overline{v}_s$ < 2,500 ft/sec (360 m/s < $\overline{v}_s$ < 760 m/s) or with either N > 50 or $\overline{s}_u$ > 2,000 psf (100 kPa),
- D Stiff soil with 600 ft/sec < $\overline{v}_s$ < 1,200 ft/sec (180 m/s < $\overline{v}_s$ < 360 m/s) or with either 15 < N < 50 or 1,000 psf < $\overline{s}_u$ < 2,000 psf (50 kPa < $\overline{s}_u$ < 100 kPa)
- E A soil profile with $\overline{v}_s$ < 600 ft/sec (180 m/s) or with either N < 15, $\overline{s}_u$ < 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 $\overline{s}_u$ < 500 psf (25 kPa),
- F Soils requiring site-specific evaluations:
- a) 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 according to preceding classes, assuming liquefaction does not occur,
- b. Peat and/or highly organic clays (H > 10 ft [3 m] of peat and/or highly organic clay, where H = thickness of soil)
- c. Very high plasticity clays (H > 25 ft [8 m] with PI > 75)
- d. Very thick, soft/medium stiff clays (H > 120 ft [36 m]) with $\overline{s}_u$ < 1,000 psf (50 kPa).

The values for $\bar{v}_s$, $\bar{N}$ or $\bar{N}_{ch}$, and $\bar{s}_u$ for site classes E, D, and C are summarized in Table 1.

Table 1 Definition of site classes in terms of $\bar{v}_s$, $\bar{N}$ or $\bar{N}_{ch}$, and $\bar{s}_u$.

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
E	< 600 fps (< 180 m/s)	<15	<1000 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 psf (> 100 kPa)

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

2.2 Site Coefficients

Site response is characterized in current US building code provisions by average short-and long-period amplification factors Fa and Fv that are dependent on the type of local site condition and the amplitude of the incoming base motion. The site coefficients as specified in the provisions are tabulated as a function of site class and maximum considered spectral acceleration at short period (0.2 second, S_S) and at long period (1.0 second, S_1). The short and long period site coefficients as they appear in the 2003 NEHRP provisions (http://www.bssconline.org/) are tabulated in Tables 2 and 3.

Table 2 Values of Site Coefficient F_a

Site Class	**Mapped MCE Spectral Response Acceleration Parameter at 0.2 Second Period** [a]				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
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	—[b]	—[b]	—[b]	—[b]	—[b]

Notes:
[a] Use straight line interpolation for intermediate values of S_S.
[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 3 Values of Site Coefficient F_v.

Site Class	**Mapped MCE Spectral Response Acceleration Parameter at 1 Second Period** [a]				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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

Table 3 Values of Site Coefficient F_v.

Site Class	Mapped MCE Spectral Response Acceleration Parameter at 1 Second Period [a]				
	$S_1 \leq 0.1$	**$S_1 = 0.2$**	**$S_1 = 0.3$**	**$S_1 = 0.4$**	**$S_1 \geq 0.5$**
F	___[b]	___[b]	___[b]	___[b]	___[b]

Notes:
[a] Use straight line interpolation for intermediate values of S_1.
[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

2.3 Definition of Design Response Spectrum

The rigorous definition of the design response spectrum S_a as currently specified in recommended provisions for the 2003 edition of the NEHRP provisions may be written succinctly as:

$$S_a = \left\{ \begin{array}{ll} \left(0.6\dfrac{T}{T_0} + 0.4\right) S_{DS} & \text{for } T \leq T_0 = 0.2\left(\dfrac{S_{Dl}}{S_{DS}}\right) \\ S_{DS} = \dfrac{2}{3} S_{MS} = \dfrac{2}{3} F_a S_S & \text{for } T_0 \leq T < T_S = \dfrac{S_{D1}}{S_{DS}} \\ S_{D1} = \dfrac{2}{3} S_{M1} = \dfrac{2}{3}\dfrac{F_v S_1}{T} & \text{for } T_S \leq T \leq T_L \\ S_{D1}\left(\dfrac{T_L}{T^2}\right) & \text{for } T_L \leq T \end{array} \right\}, \tag{1}$$

where the notation is defined as follows:

S_a	The design spectral response acceleration for a given period T,
S_{aM}	The maximum considered earthquake spectral response acceleration at a given period T,
S_{DS}	The design, 5-percent-damped, spectral response acceleration parameter at short periods,
S_{D1}	The design, 5-percent-damped, spectral response acceleration parameter at a period of one second,
S_{MS}	The maximum considered earthquake, 5-percent-damped, spectral response acceleration parameter at short periods adjusted for site class effects,
S_{M1}	The maximum considered earthquake, 5-percent-damped, spectral response acceleration parameter at a period of one second adjusted for site class effects,
F_a	Short-period site coefficient specified in table 2,
F_v	Long-period site coefficient specified in table 3,
S_S	The mapped, maximum considered earthquake, 5-percent-damped, spectral response acceleration parameter at short periods as determined from the 0.2 sec Maximum Considered Earthquake maps included in the NEHRP provisions,
S_1	The mapped, maximum considered earthquake, 5-percent-damped, spectral response acceleration parameter at 1 second period as determined from the 1.0 sec Maximum Considered Earthquake maps included in the NEHRP provisions,

T = the fundamental period of the *structure* (sec),
T_0 = $0.2S_{D1}/S_{DS}$,
T_S = S_{D1}/S_{DS},
T_L = Region dependent transition period as shown on maps of the coterminous US, California, Alaska, and Hawaii in Figures 3.3-4 through 3.3-5 of the proposed 2003 NEHRP provisions.

Notation used in the definition of the design response spectral acceleration is illustrated in Figure 1. The dashed curve represents the design response spectral acceleration estimated for a base motion spectrum for a site-class B site (firm to hard rock). The solid curve represents the design response spectral acceleration for a site class D site (stiff clays and sandy soils), where the appropriate short and long period site coefficients Fa and Fv have been multiplied by the spectra estimated for rock to obtain the resulting site class D spectra for an input maximum considered spectral acceleration at 0.2 seconds of $S_S = 0.25g$ and at one second of $S_1 = 1.0g$. The proposed 2003 NEHRP edition includes a new constraint to reduce design motions with periods longer than T_L, but no modifications of the site coefficients are proposed (see Figure 1).

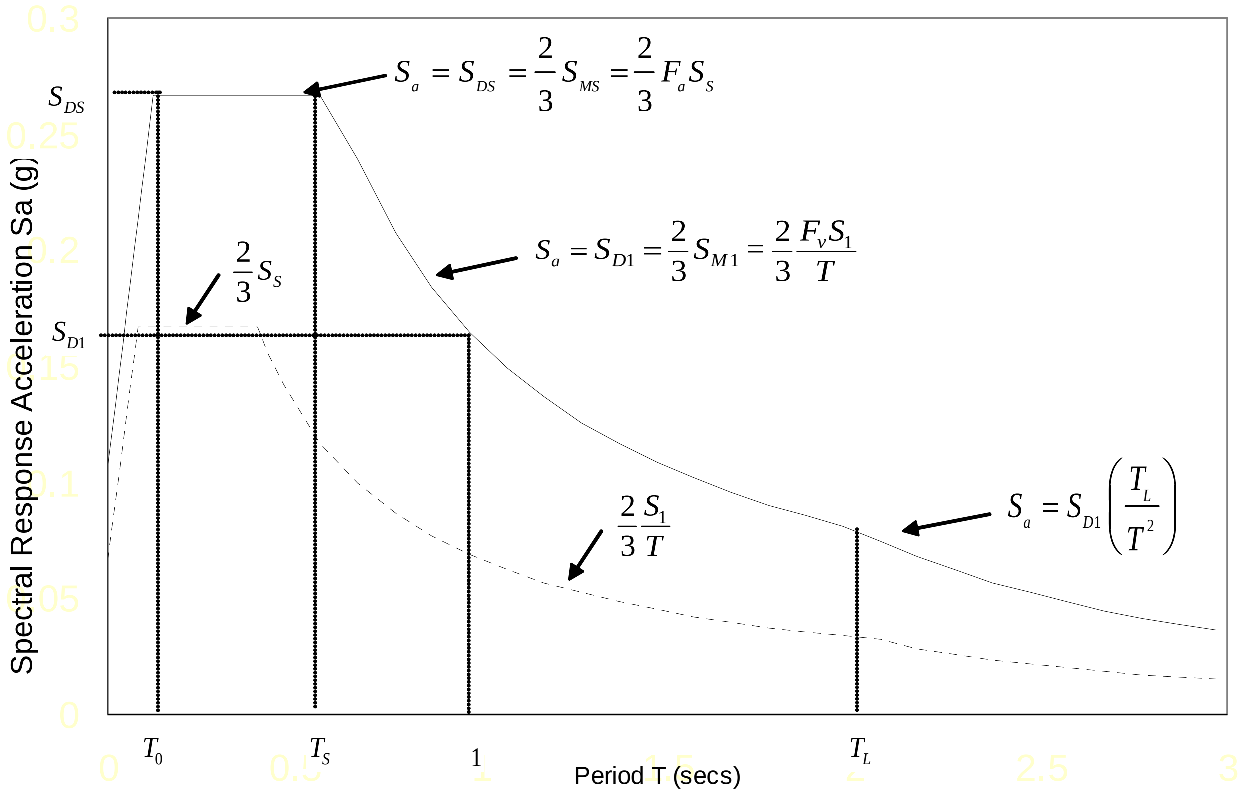


Figure 1 Illustration showing design response spectral acceleration versus period as defined in equation 1 (see text) for a base ground motion spectra (dashed curve) and a ground motion spectra modified from the base motion spectra to account for the local site effects. Parameters, as defined in equation 1, are illustrated.

2.4 Methodology for Estimating Design Response Spectrum

The methodology for estimation of design response spectral acceleration is easily understood in terms of equation 1, Figure 1, and Tables 1-3. The general steps in the methodology may be summarized as follows:

1) Classify the site according to the appropriate site class defined in Table 1 and section 2.1.

2) Determine the short-period and long-period site coefficients F_a and F_v appropriate for the determined site class from Tables 2 and 3.
3) Derive estimates of S_S and S_1 from the "Maximum Considered Earthquake" national design maps for 0.2 second and 1.0 second spectral acceleration for the site location of interest, *(The MCE maps are included in the building code provisions. They have been prepared for a uniform ground condition of site class B (firm to hard rock) from the national probabilistic seismic hazard maps showing 2% in 50 year probability of exceedance levels for spectral response acceleration with 5% damping for 0.2 and 1.0 second period motions (Frankel, and others, 1996;2003, http://eqhazmaps.usgs.gov/html/us2002.html) with deterministic upper bounds placed on the exceedance levels for some locations with short return periods (Leyendecker, and others, 2000))*
4) Derive estimates of the short- and long-period design spectral parameters S_{DS} and S_{D1} as defined in equation 1 and shown in Figure 1.
5) Derive estimates of T_0 and T_S as defined in equation 1 and T_L as shown on new maps in 2003 NEHRP provisions,
6) Compute values of spectral response acceleration function S_a for values of period T of interest in the intervals $[0,T_0]$, $[T_0,T_S]$, $[T_S,T_L]$ and $[T_L,\infty]$ using equation 1.

3. BASIS FOR SITE COEFFICIENTS AS SUMMARIZED FROM COMMENTARY FOR 2003 EDITION OF NEHRP PROVISONS

The basis for the site coefficients as provided in the commentary for the proposed 2003 edition of the NEHRP provisions is restated in summary here as explanatory information for the provisions (http://www.bssconline.org/ReformattedProv.htm).

Strong-motion recordings obtained on a variety of geologic deposits during the Loma Prieta earthquake of October 17, 1989 provided an important empirical basis for the development of the site coefficients Fa and Fv. Average amplification factors derived from these data with respect to "firm to hard rock" for short-period (0.1-0.5 sec), intermediate-period (0.5-1.4 sec), mid-period (0.4-2.0 sec), and long-period (1.5-5.0 sec) bands show that a short- and mid-period factor (the mid-period factor was later denoted the long-period factor in the NEHRP Provisions) are sufficient to characterize the response of the local site conditions (Borcherdt, 1994). This important result is consistent with the two-factor approach to response spectrum construction summarized in Figure 1. Empirical regression curves fit to these amplification data as a function of mean shear wave velocity at a site are shown in Figure 2.

The curves in Figure 2 provide empirical estimates of the site coefficients Fa and Fv as a function of mean shear wave velocity for input peak ground accelerations on rock (Borcherdt, 1994; Borcherdt and Glassmoyer, 1994). The empirical amplification factors predicted by these curves are in good agreement with those obtained from empirical analyses of Loma Prieta data for soft soils by Joyner et al. (1994). These short- and long-period amplification factors for low peak ground (rock) acceleration levels (~ 0.1 g) provided the basis for the values in the left-hand columns of Tables 2 (3.3-1) and 3 (3.3-2). Note that in Tables 2 and 3, peak ground (rock) acceleration of 0.1g corresponds approximately to a response spectral acceleration on rock at 0.2-second period (Ss) equal to 0.25g (Table 2) and to a response spectral acceleration on rock at 1.0-second period (S1) equal to 0.1g (Table 3).

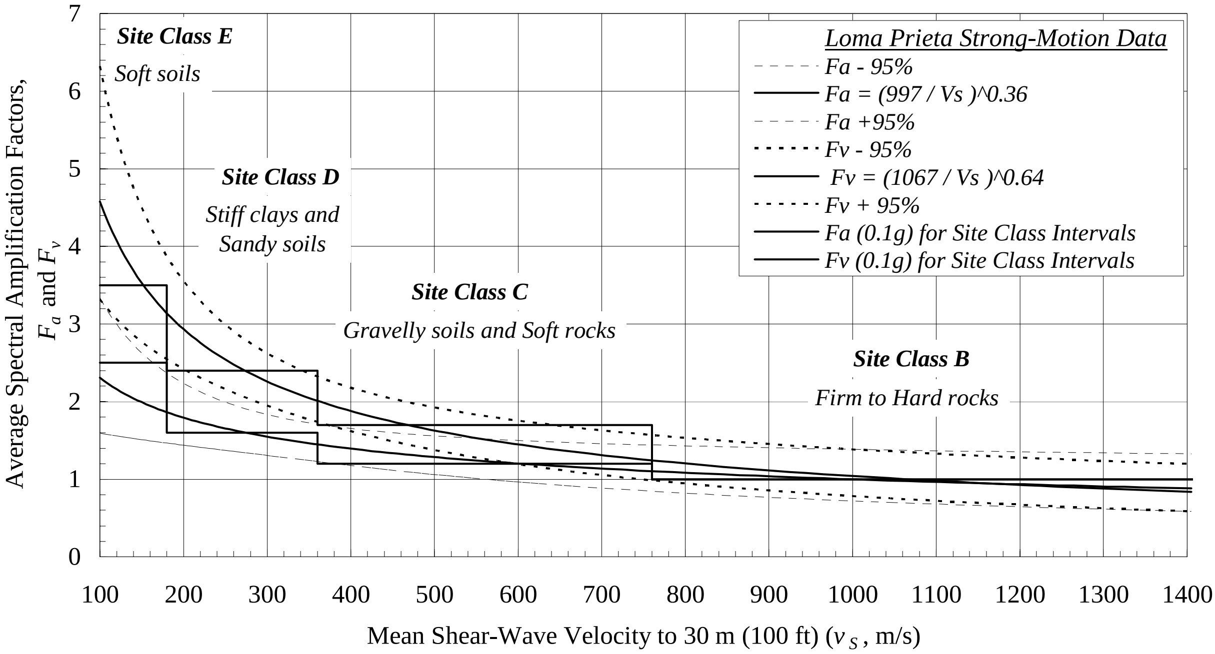


Figure 2 Short-period Fa and long-period Fv site coefficients with respect to site class B (firm to hard rocks) inferred as a continuous function of shear-wave velocity from empirical regression curves derived using Loma Prieta strong-motion recordings. The 95 percent confidence intervals for the ordinate to the true population regression line and the corresponding site coefficients in Tables 2 and 3 for 0.1g acceleration are plotted. The curves show that a two factor approach with short- and long-period site coefficients are needed to characterize the response of near surface deposits (modified from Borcherdt 1994).

The values of Fa and Fv obtained directly from the analysis of ground motion records from the Loma Prieta earthquake were used to calibrate numerical one-dimensional site response analytical techniques, including equivalent linear as well as nonlinear programs. The equivalent linear program SHAKE (Schnabel et al. 1972), which had been shown in previous studies to provide reasonable predictions of soil amplification during earthquakes (e.g., Seed and Idriss 1982), was used extensively for this calibration. Seed et al. (1994) showed that the one-dimensional model provided a good first-order approximation to the observed site response in Loma Prieta, especially at soft clay sites. After calibration, these equivalent linear and nonlinear one-dimensional site response techniques were used to extrapolate the values of Fa and Fv to larger rock accelerations of as much as 0.4g or 0.5g. These results provided the basis for the values of Fa and Fv shown in the right-most four columns of Tables 2 and 3.

Graphs and equations that provide a framework for extrapolation of Loma Prieta results to larger input ground motion levels continuously as a function of site conditions (shear-wave velocity) are shown in Figures 3a and 3b. Site coefficients in Tables 2 and 3 are superimposed on each figure. These simple curves were developed to reproduce the site coefficients for site classes E and B and provide approximate estimates of the coefficients for the other site classes at various ground acceleration levels. The equations describing the curves indicate that the amplification at a site is proportional to the shear velocity ratio (impedance ratio) with an exponent that varies with the input ground motion level (derivation details are provided in Borcherdt, 1994). The equations and graphs provide a simple framework for inference of Fa and Fv values as a continuous function of shear velocity at various input acceleration levels for sites requiring special investigations.

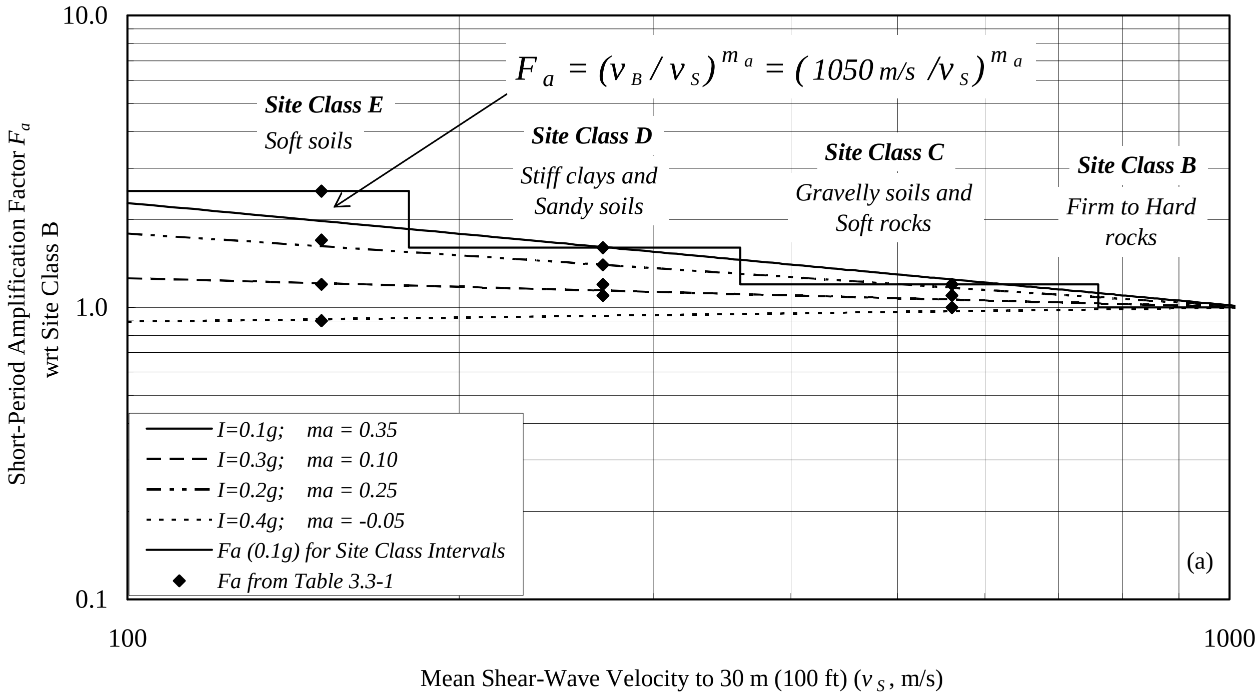
4. IMPLICATIONS OF RECENT ANALYSES FOR CHANGES IN SITE-COEFFICIENT PROVISIONS

Empirical and numerical estimates of F_a and F_v conducted on the basis of data collected since the Loma Prieta earthquake have been reported by a number of researchers, including Crouse and McGuire, 1996; Dobry et al., 1999; Silva et al., 2000; Joyner and Boore, 2000; Rodriquez-Marek et al., 2001; Stewart et al., 2001; and Borcherdt, 2002. In particular, The Northridge earthquake provided the largest set of strong-motion recordings exceeding 0.2g yet obtained in the United States. These data provided an important basis to develop empirical estimates of site coefficients, F_a and F_v for comparison with those specified in current U.S. building code provisions for Site Classes C and D, but not E, due to a limited number of "soft-soil" sites in the area.

Recent estimates of site coefficients as derived by the various investigators are plotted in Figures 4 and 5 as a function of base acceleration level from a detailed comparison by Borcherdt (2002). Figures 4a and 4b show results derived for site coefficient F_a for site class D and C sites.

Similarly, Figures 5a and 5b show results derived for site coefficient F_v for site class D and C sites. Superimposed on each figure are the best fitting regression curves and corresponding 95% confidence limits for the ordinate to the true population regression line as derived from the Northridge strong-motion recordings (Borcherdt, 2002). Also, superimposed are the site coefficients as presented in Tables 2 and 3.

The estimates, as derived by various investigators, vary depending on the database, the reference ground motion, the site-classification method, and the procedure used to infer the resultant site factors. The regressions of the site coefficients on base acceleration as derived from the Northridge recordings (Figures 4 and 5) show that F_a and F_v as specified in current code provisions at the higher levels of base acceleration (≥ 0.3 g) are within the 95 percent confidence bands for the ordinates to the true population regression line. Hence, in a rigorous statistical sense this result implies that for the higher levels of base acceleration no changes in the present code provisions are justified at the 95 percent confidence level. For lower levels of base acceleration (< 0.3 g) these regressions suggest that the short-period factors could be increased at the 95 percent confidence level by percentages up to 13 percent; however, considerations of the base normalization velocity for reference sites indicates that such an increase is not warranted. The analyses show that no change in the slopes or the regression coefficients that specify the dependence of the site coefficients on base acceleration can be justified at the 95 percent confidence level. Consensus based on review of these results by the appropriate provision update committees indicated a change in the provisions was not warranted.



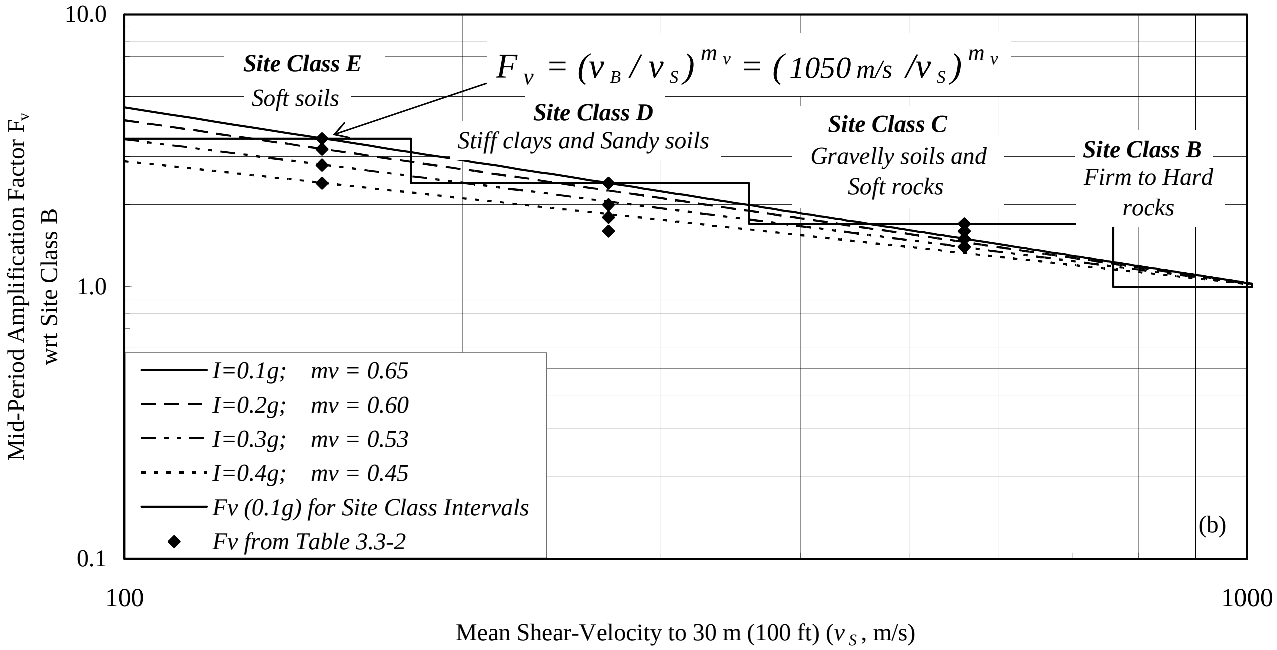
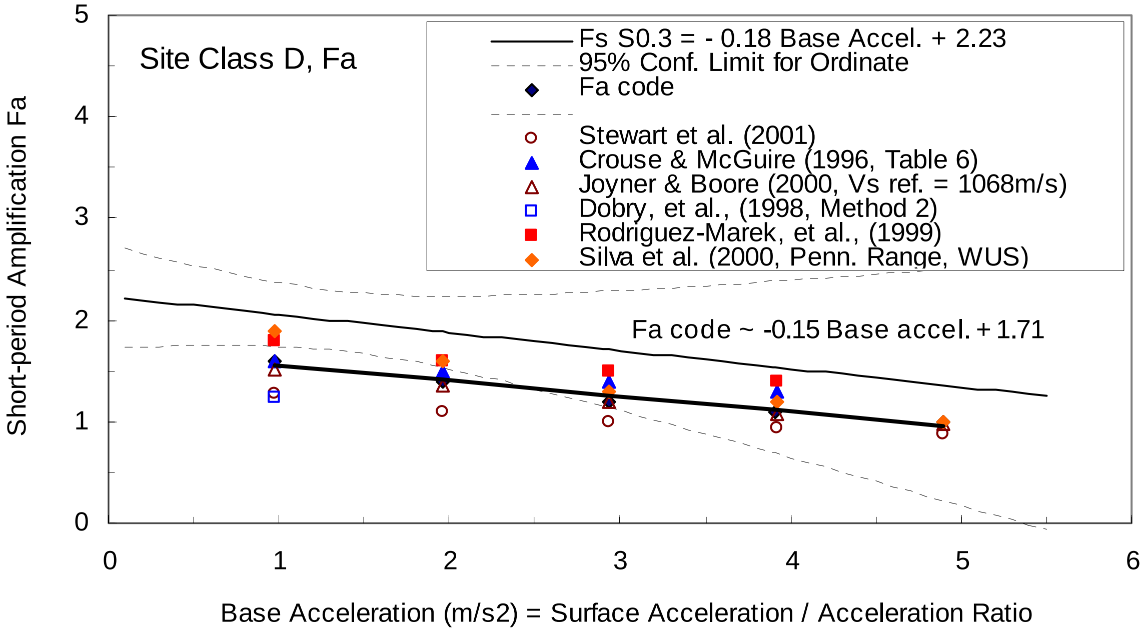


Figure 3 Graphs and equations that provide a simple framework for inference of (a) Fa and (b) Fv values as a continuous function of shear velocity at various input acceleration levels. Site coefficients in Table 2 and 3 are superimposed. These simple curves were developed to reproduce the site coefficients for site classes E and B and provide approximate estimates of the coefficients for the other site classes at various ground acceleration levels (from Borcherdt 1994).



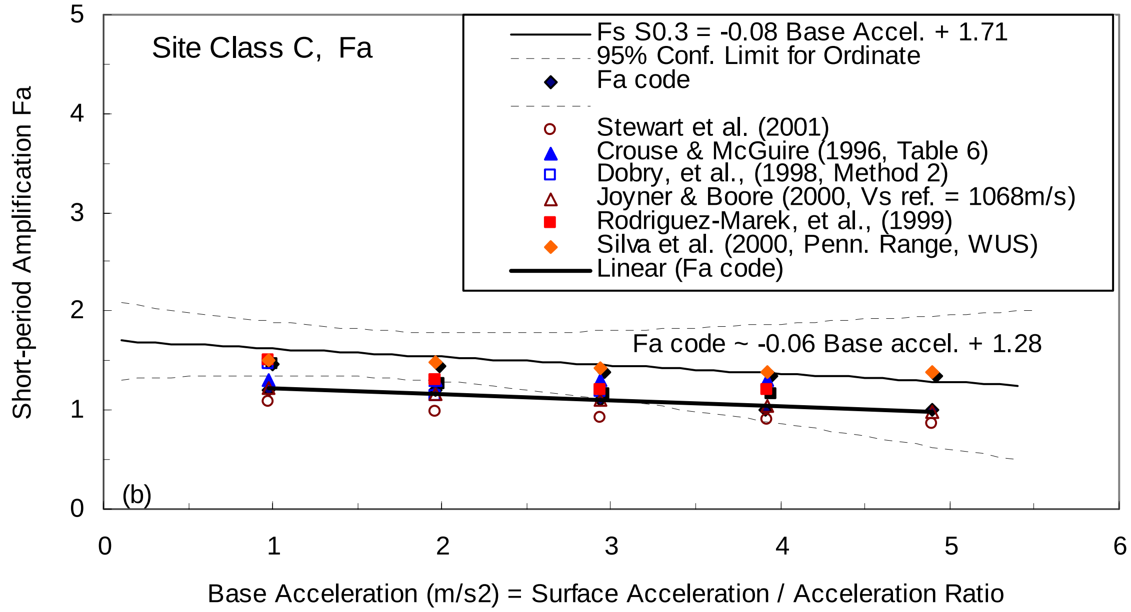


Figure 4 Estimates of site coefficient Fa for site class D (a) and C (b) sites as a function of base acceleration level and regression curves and corresponding 95 % confidence intervals for the ordinate to the true population regression line as derived from recordings of the Northridge earthquake (from Borcherdt, 2002).

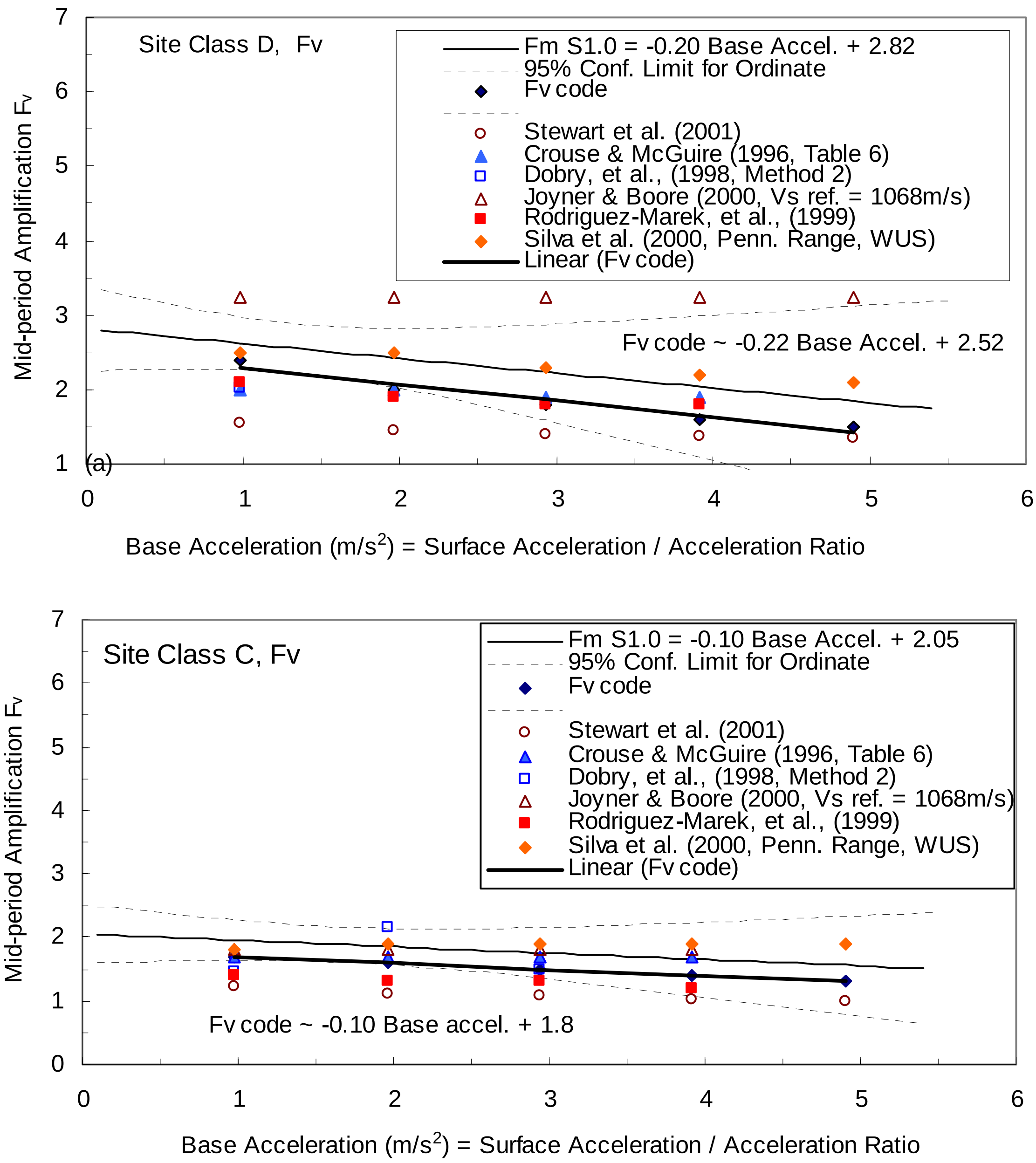


Figure 5 Estimates of site coefficient Fv for site class D (a) and C (b) sites as a function of base acceleration level and regression curves and corresponding 95 % confidence intervals for the ordinate to the true population regression line as derived from recordings of the Northridge earthquake (from Borcherdt, 2002).

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