MCEER
MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
A National Center of Excellence in Advanced Technology Applications
ISSN 1520-295X

Site Factors and Site Categories in Seismic Codes

by

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Technical Report MCEER-99-0010

April 23, 1999

This research was conducted at Rensselaer Polytechnic Institute and Geomatrix Consultants, Inc. and was supported by the Federal Highway Administration under contract number DTFH61-92-C-00106.

NOTICE

50272 – 101

REPORT DOCUMENTATION PAGE	1. Report No. MCEER-99-0010	2.	3. Recipient's Accession No.

4. Title and Subtitle Site Factors and Site Categories in Seismic Codes	5. Report Date April 23, 1999 6.
7. Authors Ricardo Dobry; Ricardo Ramos and Maurice S. Power	8. Performing Organization Report No. 10. Project/Task/Work Unit No. 106-E-2.9 and 112-D-4.1
9.Performing Organization Name and Address Department of Civil and Environmental Engineering Rensselaer Polytechnic Institute Troy, NY 12180-3590	11. Contract(C)or Grant (G) No. (C)DTFH61-92-C-00106 DTFH61-92-C-00112 (G)
12. Sponsoring Organization Name and Address Multidisciplinary Center for Earthquake Engineering Research (MCEER) State University of New York at Buffalo Red Jacket Quadrangle, Buffalo, NY 14261	13. Type of Report & Period Covered Technical report 14.

15. Supplementary Notes

This research was conducted at Rensselaer Polytechnic Institute and Geomatrix Consultants, Inc. and was supported by the Federal Highway Administration under contract number DTFH61-92-C-00106

16. Abstract (limit 200 words)

The overall objective of the task reported was to provide a detailed evaluation and assessment of the site coefficients contained in the 1994 NEHRP provisions for the seismic design of buildings in light of data provided by recent strong motion records obtained during the 1994 Northridge and 1995 Kobe earthquakes. It was found that site coefficients back-calculated from recordings of the Northridge earthquake validated the 1994 NEHRP values quite well. These site coefficients reflect a broad consensus of the geotechnical engineering and earth science communities and constitute a significant improvement over provisions contained in older codes and specifications. The authors recommend that the current AASHTO "Standard Specifications for Highway Bridges" and AASHTO "LRFD Bridge Design Specifications" be upgraded to be consistent with the 1994 and 1997 NEHRP and 1997 UBC provisions for site categories and coefficients.

17. Document Analysis a. Descriptors

Earthquake engineering. Site characteristics. Site coefficients. Seismic design. Amplification. Soil conditions. Seismic codes. Building codes. Northridge, California earthquake, January 17, 1994. Kobe, Japan earthquake, January 17, 1995. Hanshin-Awaji, Japan earthquake, January 17, 1995. Hyogo-ken Nanbu, Japan earthquake, January 17, 1995. Liquefaction.

b. Identifiers/Open-Ended Terms

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
Release unlimited.	Unclassified	108
	20. Security Class (This Page)	22. Price
	Unclassified	

(see ANSI_Z39.18)

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Publication Date: July 19, 1999
Submittal Date: April 27, 1998

Technical Report MCEER-99-0010

Task Numbers 106-E-2.9 and 112-D-4.1

FHWA Contract Number DTFH61-92-C-00106
and
FHWA Contract Number DTFH61-92-C-00112

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Preface

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

MCEER's research is conducted under the sponsorship of two major federal agencies, the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is also derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

The Center's FHWA-sponsored Highway Project develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to:

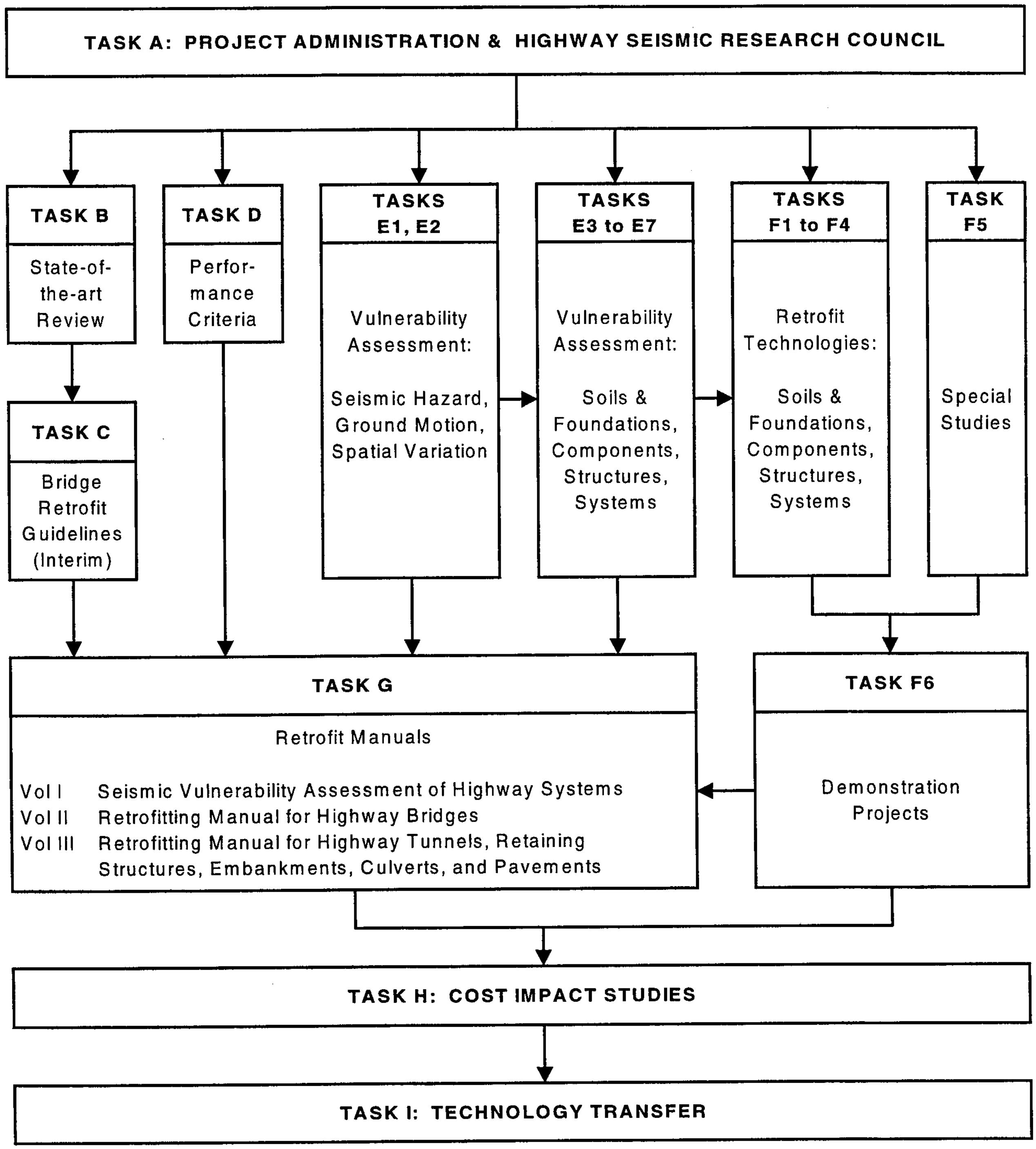
- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway systems and structures.

Highway Project research focuses on two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the existing highway structures project, and was performed within Task 106-E-2.9, "Evaluation of Site Coefficients Based on Northridge and Kobe Data" and under the new highway structures project under Task 112-D-4.1, "Site Response Effects."

The overall objective of this task was to provide a detailed evaluation and assessment of the site coefficients contained in the 1994 NEHRP provisions for the seismic design of buildings, in light of data provided by recent strong motion records obtained during the 1994 Northridge and 1995 Kobe earthquakes. It was found that site coefficients back-calculated from recordings of the

Northridge earthquake validated the 1994 NEHRP values quite well. These site coefficients reflect a broad consensus of the geotechnical engineering and earth science communities and constitute a significant improvement over provisions contained in older codes and specifications. The authors recommend that the current AASHTO "Standard Specifications for Highway Bridges" and AASHTO "LRFD Bridge Design Specifications" be updated to be consistent with the 1994 and 1997 NEHRP and 1997 UBC provisions for site categories and coefficients.

SEISMIC VULNERABILITY OF EXISTING HIGHWAY CONSTRUCTION
FHWA Contract DTFH61-92-C-00106



ABSTRACT

The report discusses some of the evidence on amplification of earthquake motions due to local soils which culminated in the new definitions of site categories and site coefficients, F_a and F_v, incorporated, first in the 1994 NEHRP Recommended Provisions for Seismic Regulations for New Buildings, and more recently in the 1997 NEHRP and 1997 Uniform Building Code (UBC). These site categories and site coefficients are described and compared to previous code provisions. Preliminary results of recent studies are discussed including averages and ranges of site coefficients calculated from recordings of the 1994 Northridge earthquake, which generally validate the 1994 NEHRP values. The possibility of performing similar calculations of site coefficients from available recordings of the 1995 Kobe, Japan earthquake is also discussed. Use of the low period site coefficient, F_a, in conjunction with de-aggregated measures of the seismic hazard on rock are suggested for evaluation of soil liquefaction in seismic codes. Areas needing further research are suggested. Finally, it is recommended that the seismic design provisions contained in the 1996 AASHTO Standard Specifications for Highway Bridges be updated to be consistent with the 1994 and 1997 NEHRP and 1997 UBC provisions for site categories and site coefficients.

ACKNOWLEDGEMENT

The authors are grateful to Dr. Walter Silva of Pacific Engineering & Analysis, for generating and providing the simulated rock response spectra for the 1994 Northridge earthquake used in Section 6.1.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SECTION 1
INTRODUCTION

Site effects associated mainly with the types and spatial distribution of soils, and also to a certain extent with the ground surface topography, play a very significant role in determining the potential for damage to engineering facilities during earthquakes. This is true for buildings as well as for bridges and other highway facilities, as shown by many earthquakes, including most recently: the 1989 Loma Prieta and 1994 Northridge events in California, and the 1995 Kobe earthquake in Japan.

While in some cases damage is due to liquefaction and associated ground failure and large ground displacements, in many others the effect is caused by amplification of the strong ground motions on softer soils as compared to the motions on rock or stiffer soils. This amplification played a significant role in the damage to highway structures in the San Francisco Bay Area during the 1989 Loma Prieta earthquake, including the collapse of the Cypress Viaduct in Oakland (EERI, 1989; Housner, 1990). The evidence from the 1994 Northridge earthquake also indicates that motions at several of the collapsed bridges may have been significantly amplified by the local soil conditions (Housner, 1994, Housner and Thiel, 1995).

SECTION 2
BASIC CONSIDERATIONS

Figure 2-1 presents horizontal elastic response spectra recorded on soft clay and rock sites during the Loma Prieta earthquake, located close to each other 70 km north of the epicenter. Figure 2-2 includes the profile of the soft clay site. While the peak acceleration on rock is approximately 0.1g, it was amplified three times to about 0.3g by the soil site; the spectral ordinates at low periods were also amplified by a factor or 2 or 3. At higher periods, the amplification is even greater, and at a period $T \approx 0.6$ sec, the rock spectrum was amplified four to five times. This behavior was typical at soft soil sites far from the epicenter in this earthquake, with the soil amplifying the rock spectrum as much as six times for the period range between 0.5 and 1.5 seconds (Housner, 1990; Chang, 1991).

A useful tool to study this amplification phenomenon is the curve of Ratio of Response Spectra (RRS) versus T, illustrated with results of a 1D site response analysis in Fig. 2-3. In Fig. 2-3, RRS < 1.5 for low periods less than 0.5 sec, but $RRS = RRS_{max} \approx 3.5$ at the predominant site period of the soft clay deposit, $T \approx 1.4$ sec. An even more extreme case of this type of amplification has been observed in the very soft clay deposits of Mexico City (Fig. 2-4) at periods of the order of 2 or 3 seconds, with RRS_{max} ranging from about 3 to 20. Fortunately, both the extreme softness of the soil as well as other characteristics of the Mexico City clay inducing these very high amplifications are rather unusual.

Useful insight on the factors controlling the value of RRS_{max} and the amplification phenomenon at soft sites revealed by Figs.2-1- 2-4 is provided by the model of Fig. 2-5 and the results in Fig. 2-6 (Roesset, 1977). Both the horizontal acceleration on rock at point B, a_B, and the corresponding soil acceleration at point A, a_A, are caused by vertically propagating, harmonic shear waves of frequency f (cps). Therefore, both a_A and a_B are amplitudes of harmonic (sinusoidal) accelograms of frequency f. The amplification ratio a_A / a_B is a function of the ratio of frequencies $f / (V_s / 4h)$, of the soil material damping ratio β_s, and of the rock/soil impedance ratio, $I = \gamma_r V_r / \gamma_s V_r$. In Fig. 2-5, γ_r, γ_s are the unit weights and V_r, V_s are the shear wave velocities of rock and soil. The maximum amplification $(a_A / a_B)_{max}$ corresponding to resonance in shear of the soil layer, occurs at about the natural frequency of the layer, $f \approx V_s / 4h$, and is approximately equal to:

$$\left[\frac{a_A}{a_B}\right]_{max} \approx \frac{1}{(1/I)+(\pi/2)\beta_s} \qquad (2\text{-}1)$$

In Fig. 2-6, $V_s / 4h = 1.88$ cps, $I = 6.7$, and for $\beta_s = 0.05$, $(a_A / a_B)_{max} \approx 4.4$. A plot such as Fig. 2-6 provides the transfer function of the site, which is constant and independent of the rock motion for the assumed linear soil and vertically propagating shear waves, and can be properly estimated by dividing Fourier Spectra of recorded horizontal accelerations on nearby soil and rock sites. On the other hand, plots of Ratio of Response Spectra such as in Fig. 2-3, more appropriate for engineering evaluations involving response spectra and for determination of site coefficients in seismic codes, are not independent of the rock motion. However, analyses and comparisons at actual soft clay sites on much stiffer rock or soil, suggest that: (i) both RRS_{max} and $(a_A / a_B)_{max}$ occur at about the same frequency, (ii) Equation 2-1 often predicts reasonably well the value of RRS_{max} (Dobry, 1991), and (iii) average values of RRS and (a_A / a_B) for the same period range are within 30% of each other (Joyner et al., 1994).

Therefore, based on Eq. 2-1 it could be expected that the value of RRS_{max} at soft clay sites is controlled by two main factors: the impedance ratio $I = (\gamma_r / \gamma_s)(V_r / V_s)$, and the internal damping β_s of the soil. Some main reasons why the Mexico City clay exhibits such high amplification (Fig. 2-4) are the low values of V_s, γ_s and β_s of this soil. For most soft clay sites such as those typically encountered in the US, the values of β_s and γ_s are higher. Also, the ratio $\gamma_r / \gamma_s \approx 1.1$ to 1.4 is quite constant in many sites, while β_s depends mostly on the intensity of the rock motions due to soil nonlinearity, as well as on the plasticity index of the clay (Vucetic and Dobry, 1991; Dobry et al., 1994). Thus, for a given type of rock outcrop, I is about proportional to $1/V_s$, and for a narrow range of plasticity indices, it should be expected that RRS_{max} at a site would depend mainly on V_s and on the intensity of the rock motions. This predicts that RRS_{max} should depend mainly on V_s for a specific clay of very high plasticity and small soil nonlinearity (and correspondingly small values of β_s), as confirmed by the Mexico City data in Fig. 2-4.

Seismic code provisions based on the theoretical framework described above, that is on calculations of the site period and of RSS_{max}, may be appropriate for a specific area consisting mainly of soft clays of known depth on much stiffer soil or rock, and for expected earthquakes which induce soil resonance without much soil nonlinearity. This is the case of the Mexico City seismic code (Simón and Suárez, 1994).

However, US codes must consider a much wider variety of site conditions and earthquake motions for which a direct application of the model of Fig. 2-5 is either not relevant or impractical. Specifically, at stiffer soil sites, while amplification of ground motions is typically observed with RRS > 1, often there is no clear peak and no value of RRS_{max} in the plot of RRS versus T. In many soft clay sites there is often the additional complication of stiffer soil deposits between the soft clay and the rock (Fig. 2-2). Also, in soft clay sites the values of site period and RRS_{max} will typically depend on the earthquake. Therefore, a more practical approach for the evaluation of site coefficients for seismic codes is the calculation of an average value of RRS at a given period for a number of stations having generally similar soil conditions, as done for the Loma Prieta earthquake in Fig. 2-7 for soft clay sites and in Fig. 2-8 for stiffer alluvium (Joyner et al., 1994).

Based on the assumption that the energy of the wave is preserved, Joyner et al. (1981) and Atkinson and Boore (1997) have suggested that the amplification RRS at a given period should be more or less proportional to $(V_s)^{-0.5}$, where V_s is again the shear wave velocity of the soil at shallow depth. Empirical calculations from earthquake records by Boore et al. (1994, 1997), Midorikawa et. al. (1994) and Borcherdt (1994a,b), using either RRS or Ratio of Fourier Spectra, have indicated that the amplification is approximately proportional to $(\bar{V}_s)^{-0.4}$ at low periods, and approximately proportional to $(\bar{V}_s)^{-0.6}$ at longer periods, where $\bar{V}_s$ = average shear wave velocity of the soil in the top 30 m (100 ft). Figure 2-9 presents the curves obtained by Borcherdt (1994a, b) from Ratios of Fourier Spectra of records on soil and rock in the 1989 Loma Prieta Earthquake, for a wide variety of soil sites. In Fig. 2-9, the short period amplification factor, F_a, was obtained by averaging the ratios of Fourier Spectra in the period range, T = 0.1 to 0.5 sec, and the long period amplification factor, F_v, was obtained from the period range 0.4 to 2 seconds. For the Loma Prieta earthquake, corresponding to a maximum rock acceleration of about 0.1g, Figs. 2-7 through 2-9 indicate values of F_a and F_v decreasing to 1.0 as V_s approaches a value of about 1000 m/sec or greater corresponding to the reference rock sites. These values and trends of F_a and F_v are consistent with the corresponding RRS for similar period ranges obtained by Joyner et al. (1994) in Figs. 2-7 and 2-8, also for the Loma Prieta earthquake.

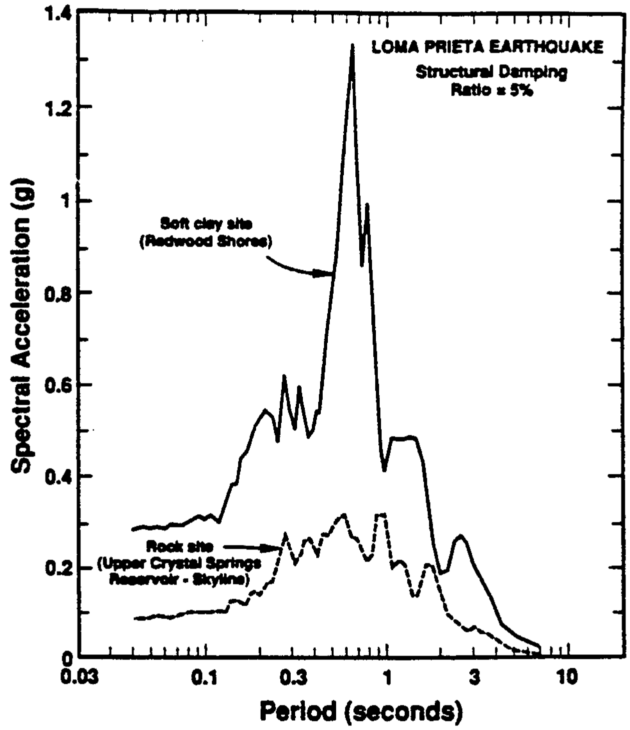


Figure 2-1 Recorded response spectra at a soft clay site (Redwood Shores) and at a rock site (Upper Crystal Springs Reservoir-Skyline), both located south of San Francisco, 1989 Loma Prieta earthquake (Chang, 1991; Dobry, 1991)

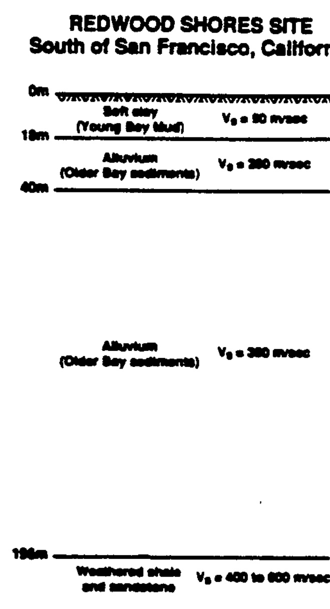


Figure 2-2 Dynamic soil profile at Redwood Shores Site (Dobry, 1991)

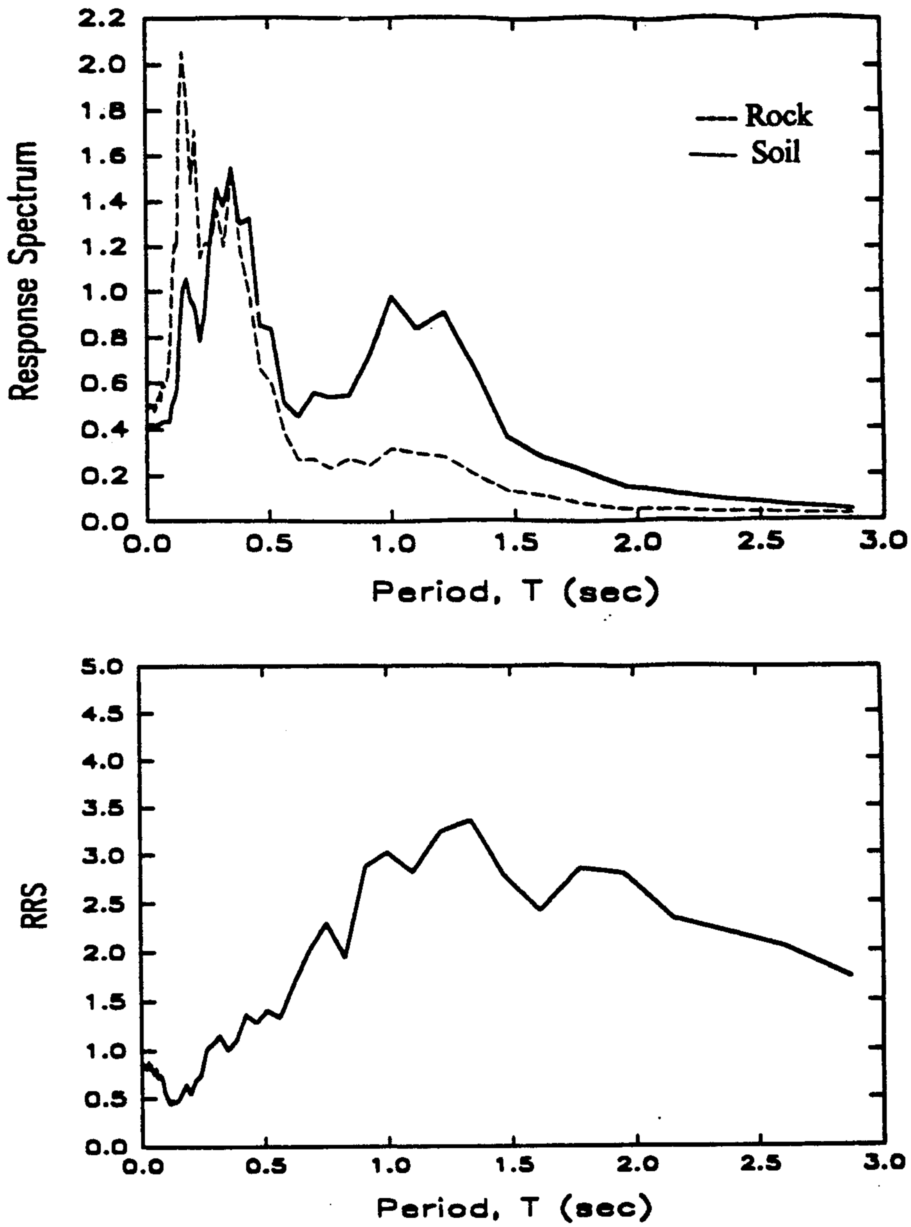


Figure 2-3 Determination of Ratio of Response Spectra (RRS) between soil and rock

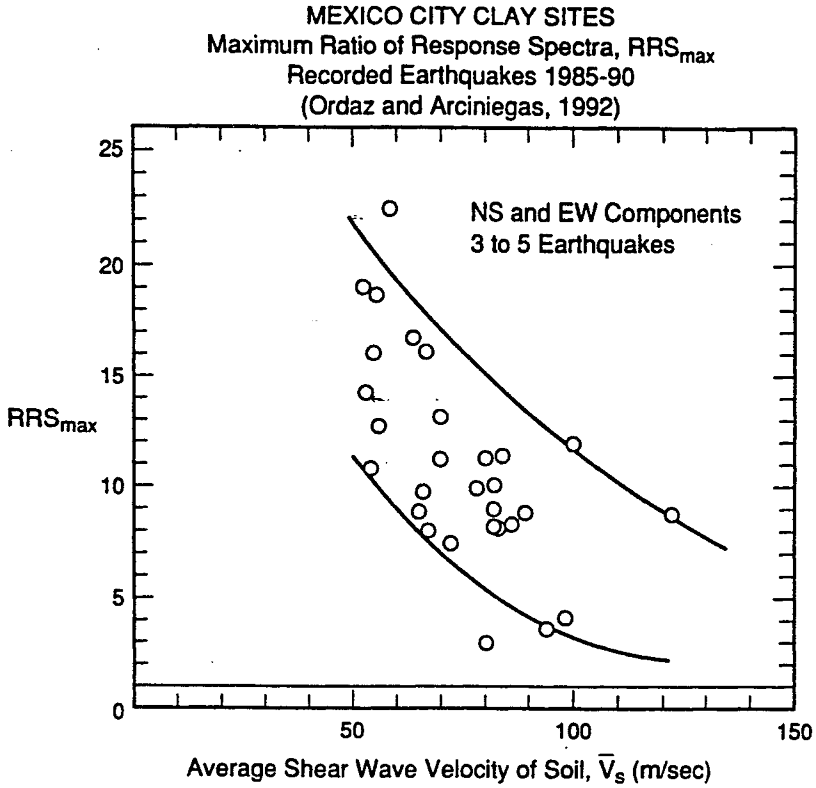


Figure 2-4 Maximum Ratio of Response Spectra, $(RRS)_{max}$, versus $\overline{V}_S$, recorded on Mexico City clay sites during five earthquakes between 1985 and 1990 (Ordaz and Arciniegas, 1992)

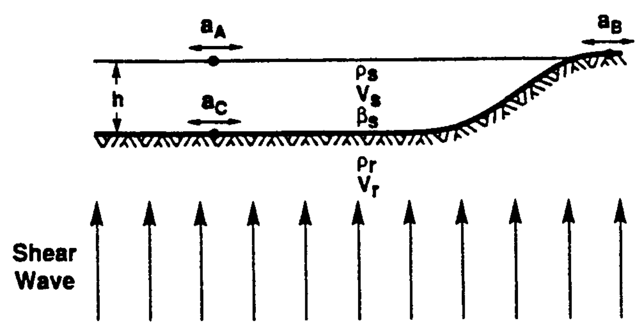


Figure 2-5 Uniform soil layer on elastic rock subjected to vertical shear waves

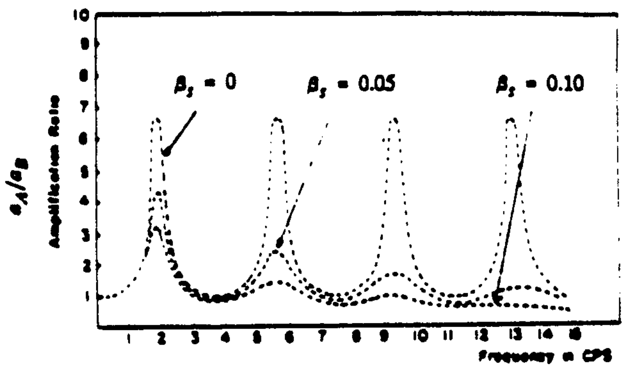


Figure 2-6 Amplification ratio soil/rock for *h* – 100 ft (30.5m), V_s = 1.88 cps, and IR = 6.7 (Roesset, 1977).

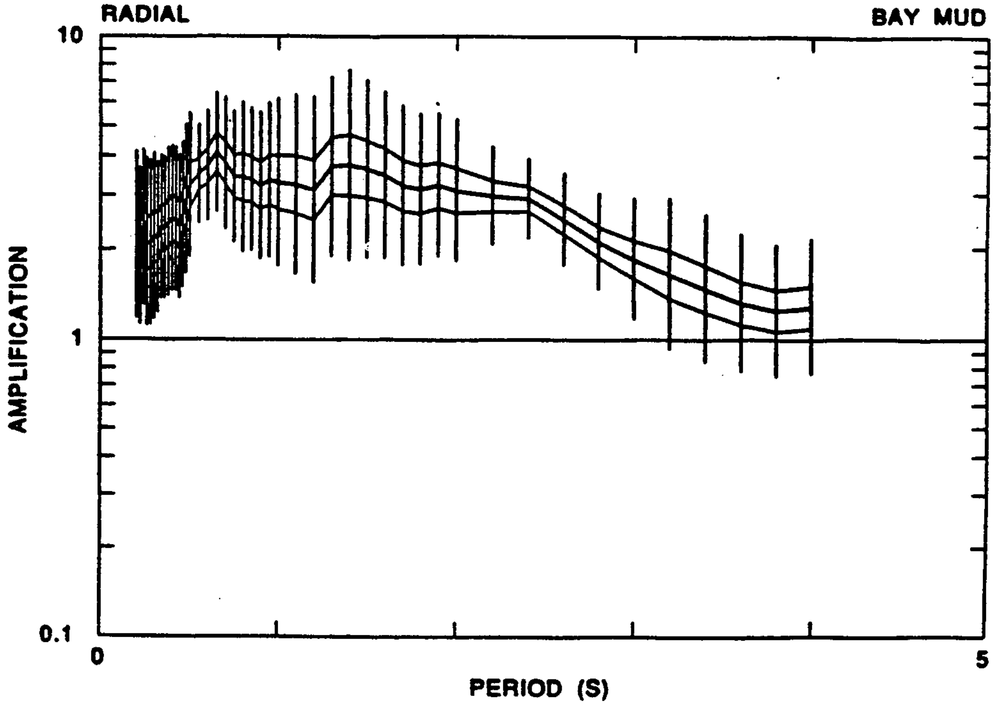


Figure 2-7 Calculation of average RRS curves from records of 1989 Loma Prieta earthquake on soft sites (Joyner et al., 1994)

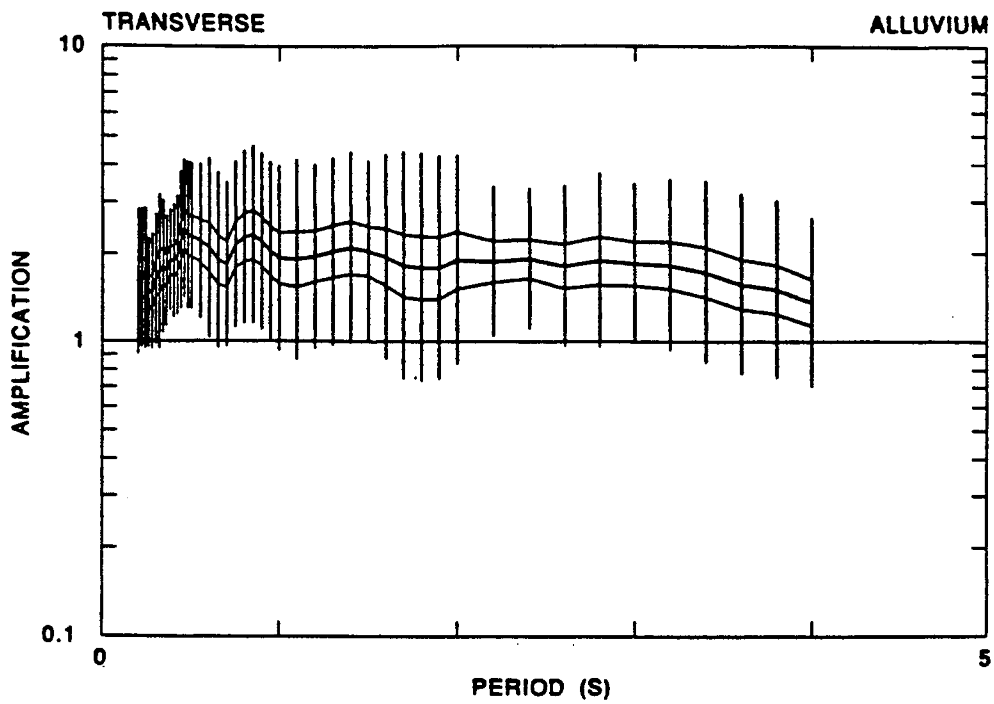


Figure 2-8 Calculation of average RRS curves from records of 1989 Loma Prieta earthquake on stiffer alluvium sites (Joyner et al., 1994)

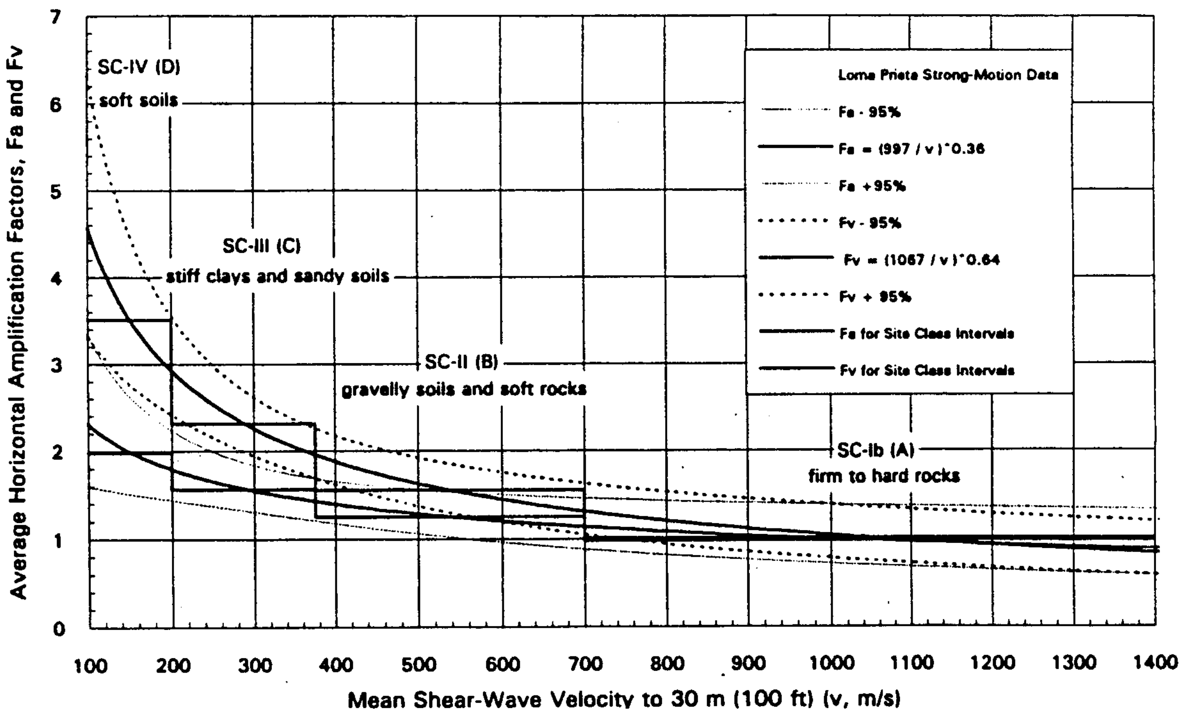


Figure 2-9 Short-period F_a and mid-period F_v amplification factors with respect to "firm to hard" rock plotted as a continuous function of mean shear wave velocity using the regression equations derived from the strong-motion recordings of the Loma Prieta earthquake. The 95 percent confidence intervals for the ordinate to the true population regressions line and the amplification factors for the simplified site classes also are shown (Borcherdt, 1994)

SECTION 3
SEISMIC CODE SPECIFICATIONS PRIOR TO 1994

The next section of this report discusses the new site categories and site coefficients for seismic design of buildings, incorporated first into the 1994 and 1997 NEHRP Provisions for Buildings, and later in the 1997 Uniform Building Code. These new site categories and coefficients are based largely on the basic considerations and observations presented in the previous section.

The rest of this section considers the basis for the regulations on local site conditions contained in building seismic codes prior to 1994, which are still current in the 1996 AASHTO Standard Specifications for Highway Bridges.

Seed and coworkers (1976a), and more recently Idriss (1990a, b) studied the relation between peak acceleration recorded on soil and that obtained on a nearby rock outcrop. Seed and his coworkers had at their disposal mostly records on stiff and deep cohesionless soils, with few on soft clay sites, and found very little difference between rock and soil accelerations. On the other hand, Fig. 3-1 includes the more recent curve developed by Idriss for soft sites based on data from a number of earthquakes including 1985 Mexico City and 1989 Loma Prieta, at low accelerations, and using site response calculations to extrapolate to larger rock accelerations. For low rock accelerations of the order of 0.05g to 0.10g, the corresponding soft soil accelerations are 1.5 to 4 times greater than the rock accelerations. This amplification factor decreases as the rock acceleration increases, and it becomes approximately unity for a rock acceleration of 0.4g, with a tendency for deamplification to occur at larger rock accelerations. This amplification at low rock motion intensities, and lack of amplification or even deamplification at high rock motion intensities, is directly related to the nonlinear stress-strain behavior of the soil as the rock acceleration increases.

A second step in the development of seismic regulations prior to 1994 was the study of the shape of the response spectrum and its correlation with the site conditions. Average spectral shapes for various soil conditions were developed by Seed et al. (1976a, b), on the basis of a statistical study of more than 100 records from twenty-one, mostly California earthquakes available at the time. These spectral shapes are shown in Fig. 3-2. As they are anchored at $T = 0$ to a value of 1.0, their use requires knowing the peak acceleration on rock or soil, which is typically obtained from seismic hazard maps. The spectral shapes in Fig. 3-2 are fairly constant and independent of site conditions at low periods but very different at longer periods, $T > 0.5$ sec. Therefore, based on this and other similar studies and code provision development activities such as ATC-3 (Applied Technology Council, 1978), simplified spectral shapes and associated site coefficients S such as shown in Fig. 3-3 were incorporated in building codes such as NEHRP and UBC and in bridge codes such as AASHTO. Figure 3-3 also includes the descriptions of the corresponding soil profile types S1 to S4, which are a mixture of qualitative and quantitative definitions, including both the type and stiffness of the soil as well as its depth. These descriptions of S1 to S4 are subject to interpretation and do not always define clearly one soil profile type at a given location. The corresponding amplification factor, applicable only at long periods, varies from 1.0 for rock and shallow stiff soils (S2) to 2.0 for thick soft sites (S4).

In AASHTO 1996, NEHRP 1991 and 1994, and UBC 1997, the rock acceleration at zero period is defined by a coefficient variously called A, Z, or A_a and A_v, obtained from a map of the US, and representing the "effective peak acceleration" needed to construct the spectrum through specifications such as those of Fig. 3-3, either to define a lateral force coefficient to compute the lateral force, or to design the structure using modal superposition[4]. The value of A in AASHTO is of the order of 0.4g in California and of the order of 0.1g in parts of the East. While in principle the spectral shapes and long period site coefficients S of Figs. 3-2 and 3-3, could have been applied in conjunction with amplification curves for the peak acceleration such as shown in Fig. 3-1, thus amplifying also the low period spectra and introducing the effect of soil nonlinearity, this was generally not done. The reason is that the studies by Seed et al (1976a) and ATC-3 (Applied Technology Council, 1978) had found little effect of site conditions on the acceleration.

As a result, in most seismic regulations including NEHRP before 1994 as well as the current AASHTO, the soil acceleration is assumed to be equal or close to the rock acceleration. The result seems to be about right on the average for soft clay sites in high-seismic parts of California, as soil presumably does not amplify or amplifies very little the rock peak acceleration for the level of about 0.4g used there in the codes (e.g. Fig. 3-1). However, it is not right at all in the East and other parts of the U.S. where a level of rock acceleration of 0.1g or 0.2g is appropriate, and where an amplification of the peak acceleration of as much as 2 or 3 and its effect on the spectrum should be considered.

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[4]In the maps attached to the 1994 NEHRP Provisions, two "effective peak accelerations" A_a and A_v instead of one, are defined to accommodate parts of the US where the intensity of rock motions at short and long period ranges for uniform seismic hazard on rock, are determined by different earthquakes (often a low magnitude, nearby earthquake controlling the short periods, and a larger magnitude, distant earthquake determining the rock spectral level at long periods). These two accelerations are A_a at short periods and A_v at long periods. In the maps attached to NEHRP 1991 and 1994, for San Francisco, $A_a = A_v = 0.4g$; and for New York City, $A_a = A_v = 0.1g$. New maps attached to the 1997 NEHRP Provisions (NEHRP, 1997) use two parameters other than A_a and A_v to specify the seismic hazard on rock. These new 1997 NEHRP rock seismic hazard parameters are not discussed in this report. However, the specification of site categories and site coefficients F_a and F_v is essentially identical in the 1994 and 1997 NEHRP Provisions for comparable hazard on rock. The nomenclature used in this report follows that of 1994 NEHRP.

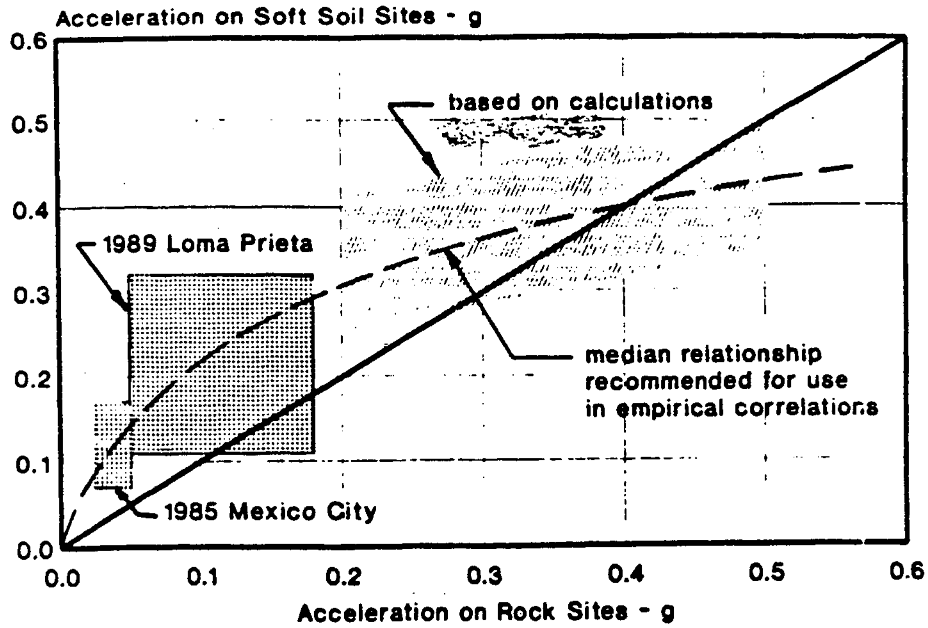


Figure 3-1 Relationships between maximum acceleration on rock and other local site conditions: Idriss (1990 a,b)

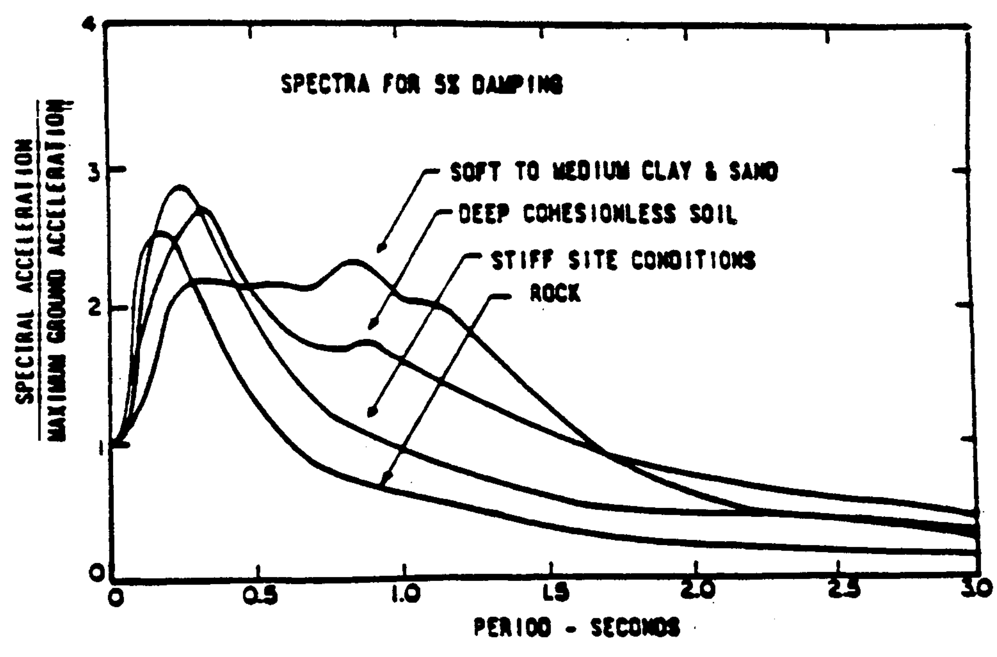
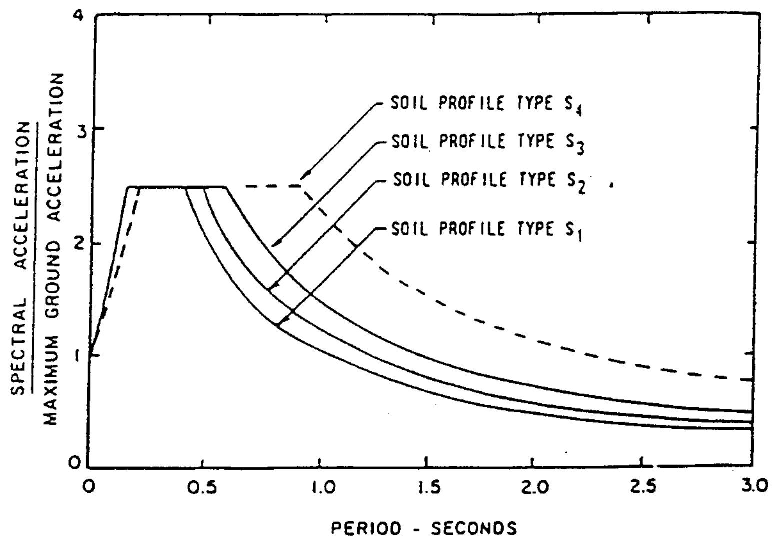


Figure 3-2 Average acceleration spectra for different site conditions (Seed et al., 1976a and 1976b)



Site Coefficient

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 stiff 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

Figure 3-3 Soil profile types, site coefficients and spectral shapes contained in seismic codes prior to NEHRP 1994 (Martin and Dobry, 1994)

SECTION 4
1994 AND 1997 NEHRP AND 1997 UBC PROVISIONS FOR NEW BUILDINGS

Figures 4-1 through 4-4 and Tables 4-1 through 4-4 summarize the new, modified local site response provisions for buildings incorporated into the 1994 and 1997 NEHRP recommended provisions, and also into the 1997 Uniform Building Code, as well as some additional results used for their development. These provisions were the result of an effort made for several years, mostly by the geotechnical engineering and earth science communities, and reflects a broad consensus of both communities based on both recorded evidence and theory.

The effort started in a workshop in Buffalo in October 1991 sponsored by NCEER and organized by Robert Whitman (Whitman, 1992), continued through the activity of a committee chaired by Maurice Power, and culminated in a workshop supported by NCEER, SEAOC, BSSC, NSF, and USGS and hosted by Geoffrey Martin at the University of Southern California (USC) in November 1992 (Martin, 1994; NEHRP, 1994; Martin and Dobry, 1994). This workshop, attended by 65 invited geoscientists, geotechnical engineers, and structural engineers, developed the broad consensus reflected in Figs. 4-3 and 4-4 and Tables 4-1 through 4-4. The actual site categories and site coefficients adopted were based on merging three similar proposals presented to the workshop by Roger Borcherdt, Ricardo Dobry and Raymond Seed. The use of the average V_s of the top 30m of soil to characterize the site was proposed by Roger Borcherdt based on his research (Borcherdt, 1994a). Much of the evidence of soil amplification was based on Loma Prieta 1989 and other earthquakes associated with rather low rock accelerations of the order of 0.1g, with essentially no recorded information available at the time on amplification of greater accelerations on rock such as 0.4g or 0.5g. Therefore, extensive site response analyses were performed by a number of researchers using the equivalent linear technique (mostly program SHAKE; Schnabel, et al., 1972), as well as nonlinear programs, to obtain extrapolated values of the site coefficients F_a and F_v with due consideration to soil nonlinearity. Some of this analytical work is shown in Figs. 4-1 and 4-2, while more extensive summaries are included in Martin and Dobry (1994), and in the Commentary volume of the 1994 NEHRP Provisions (NEHRP, 1994).

In summary, the new approach uses two site factors, F_a and F_v, for the short-period range and long-period range, respectively, instead of a single factor, with the two factors depending on both the site category and the intensity of the rock motions (defined by A_a or A_v in the 1994 NEHRP Provisions), and with the site category (called Soil Profile Type) defined by the average V_s of the top 30m of soil. Therefore, both the old site categories and the spectral shapes of Fig. 3-3 become obsolete and are replaced by Figs. 4-3 and 4-4 and Tables 4-1 through 4-4. Table 4-1 includes the approximate correspondence between the new Soil Profile Types A to E and the old site categories S1 to S4. The variation of site factors F_a and F_v with level of rock acceleration A_a and A_v is plotted in Fig. 4-4 for stiff (C and D) and soft (E) sites.

In building and bridge seismic codes, typically a key step is the determination of a seismic coefficient used to determine the level of lateral design forces for the structure. While this seismic coefficient depends on the particular code, method of analysis, fundamental period of the structure, importance of the structure and ductility of the structural system, it is always proportional to a value representing the design level of acceleration on soil (C_a or C_v), taken as the product of the design acceleration on rock (A_a or A_v for 1994 NEHRP, Z for 1997 UBC, and A for AASHTO) times the corresponding soil factor (F_a, F_v, or S).

Table 4-4 reproduces the corresponding values of $C_a = F_a A_a$ and $C_v = F_v A_v$ included in 1994 NEHRP. The most striking feature of Table 4-4 is that the effect of soil can be as significant or even more significant than the level of seismicity of the area as measured by the map values A_a, A_v or Z, in determining the level of lateral forces for which the structure must be designed. The value of C_a of soft profile type E in Table 4-4 becomes essentially constant, $C_a \approx 0.36$, independent of A_a for $A_a \geq 0.2g$, while the value of $C_v \approx 0.35$ to 0.40 is about the same for high seismicity rock sites (B) in California where $A_v = 0.4g$, as for lower seismicity soft sites (E) in the East where $A_v = 0.1g$ or 0.2g (Table 4-4). This dramatic increase of the level of seismic rock motions by the soil in low seismicity areas, which tends to erase the traditional concept of seismic hazard focused on motions on rock or firm ground, is a direct consequence of the nonlinear response of soil illustrated by Figs. 3-1 and 4-2, which tends to amplify more small rock motions as compared to larger rock motions.

It is useful to review again in some detail the three main innovations contained in the 1994 NEHRP and 1997 UBC, as well as in the 1997 NEHRP Provisions

1. The site characterization (Soil Profile Type) is now based only on the top 30 m (100 ft) of soil (Tables 4-1 and 4-2), disregarding both the depth of soil to rock if greater than 30m, the soil properties below 30m, and the properties of the rock underlying the soil. The soil profile type is made solely and unambiguously dependent on one parameter (the average shear wave velocity $\overline{V}_s$ of the top 30m of soil). However, the use of more readily available soil properties such as the Standard Penetration Resistance ($\overline{N}$ or $\overline{N}_{ch}$ in Table 4-2), or undrained shear strength ($\overline{s}_u$ in Table 4-2), are also conservatively allowed to characterize the top 30m of soil. Previous code versions, while relying in qualitative descriptions of the soil and thus being more ambiguous, did require information on soil type and total soil thickness down to much greater depths (200 ft), as shown by Fig. 3-3. While these other parameters in addition to $\overline{V}_s$ of the top 30m certainly play a role in local site response, it was felt that a single-parameter characterization based on $\overline{V}_s$ was appropriate at this stage and should cover most cases of interest. As discussed earlier, theoretical considerations and studies of actual ground motions point out to the great significance of $\overline{V}_s$ of the shallower soil to site amplification; therefore, if one parameter is to be selected, this is a natural one from a scientific viewpoint. It is also clearly measurable in the field (by geophysical techniques), thus removing the ambiguity of definitions of site categories contained in previous codes. Finally, the restriction to the top 30m makes it much more feasible for geotechnical engineers and earth scientists to come up with the necessary information for the site from available data. Therefore, the soil profile types based on $\overline{V}_s$ are now unambiguous, practical to use, and scientifically sound in that site amplification of ground motions for many or most soil sites and earthquakes are expected to be determined in first approximation by the value of $\overline{V}_s$. Also, it has provided

researchers with an unambiguous and measurable parameter for empirical studies of future earthquake data aimed at verifying, refining, or modifying Table 4-3, for example by including other factors in addition to $\overline{V}_s$ as more data becomes available. This is not a negligible consideration when considering the rapid worldwide increase taking place in available recordings and site information from recording stations.

2. A short period amplification site coefficient (F_a) is introduced, which did not exist before (Figs. 3-3 and 4-3). That is, the one-parameter model of local site amplification characterized by the coefficient S, is replaced by a two-parameter model characterized by F_a and F_v. Therefore, once the response spectrum on rock is specified (through the values of A_a and A_v in 1994 NEHRP, or Z in UBC, obtained from the corresponding seismic hazard map), the spectrum on soil is now calculated by using both F_a (which amplifies the short period part of the rock spectrum, in the neighborhood of T = 0.2 or 0.3 sec) and F_v (which amplifies the long period part of the rock spectrum, at periods in the neighborhood of T = 1 sec and above). In the old codes and provisions, essentially $F_a \approx 1$, with no soil amplification at short periods. Both F_a and F_v are unity for rock (Soil Profile Type B) and become greater as the soil becomes softer as measured by $\overline{V}_s$ and the Soil Profile Type evolves through C, D, and E. For the softer sites (Soil Profile Type E with $\overline{V}_s$ < 180 m/s), maximum values of $F_a = 2.5$ and $F_v = 3.5$ are specified in Table 4-3. In all cases, $F_a \leq F_v$ in the Table, reflecting the generalized experience about soil amplification that brought about the concept of spectral shapes and normalized response spectra contained in codes prior to NEHRP 1994 (Figs. 3-2 and 3-3). While $F_a \approx 1$ or even $F_a < 1$ seems to be about right in soft soils subjected to very intense rock motion, such as characterized by $A_a > 0.4$, due to soil nonlinearity, large amplifications of short period rock motions have been observed when the rock motions are less intense, like in the 1989 Loma Prieta earthquake. The need for $F_a > 1$ at short periods for less intense rock motions is also predicted by site response analyses.

3. Finally, and consistent with the analytical studies and the evidence from the field, the effect of soil nonlinearity is introduced by making both site coefficients F_a and F_v functions of the level of intensity of rock motions. That is, the two site coefficients F_a and F_v in Table 4-3 and Fig. 4-4 are now a function of: (i) the Soil Profile Type, and (b) the level of rock motion given by A_a or A_v in 1994 NEHRP (or by Z in UBC). This should be contrasted with the old codes and provisions, where the site coefficient S depended only on the site category and was unaffected by A_a or A_v. The main consequence of this change is the appearance of some large amplifications at both short and long periods on soft soils for those parts of the country where A_a and A_v are low, like the Eastern US (e.g., $F_a = 2.5$ and $F_v = 3.5$ for Soil Profile Type E and A_a or $A_v \leq 0.1$ in Table 4-3). Therefore, in these low seismicity areas of the country, the seismic forces for buildings on soft soil are significantly increased when compared to previous codes. In particular, the seismic forces for stiff structures or for the higher modes of taller buildings are now much higher at soft soil sites in these low seismicity areas compared to nearby rock sites. This effect of the change in the 1994 NEHRP provisions is exactly as intended based on the evidence. On

the other hand, for high seismicity areas in California where $A_a = A_v = 0.4$, $F_a \approx 1$, as before, more or less independent of site category, with the resulting shape of the spectra being very similar to the old spectral shapes of Fig. 3-3.

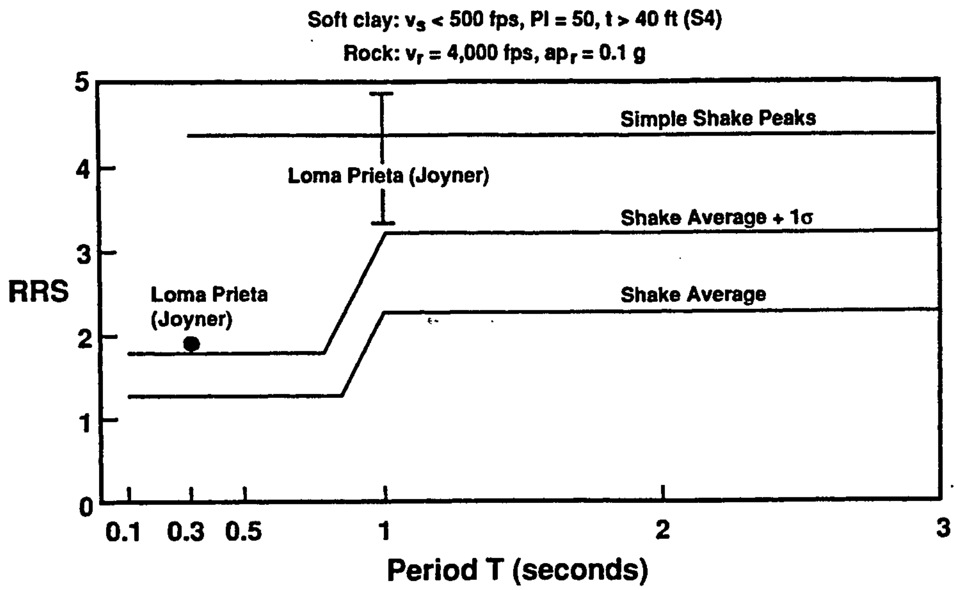
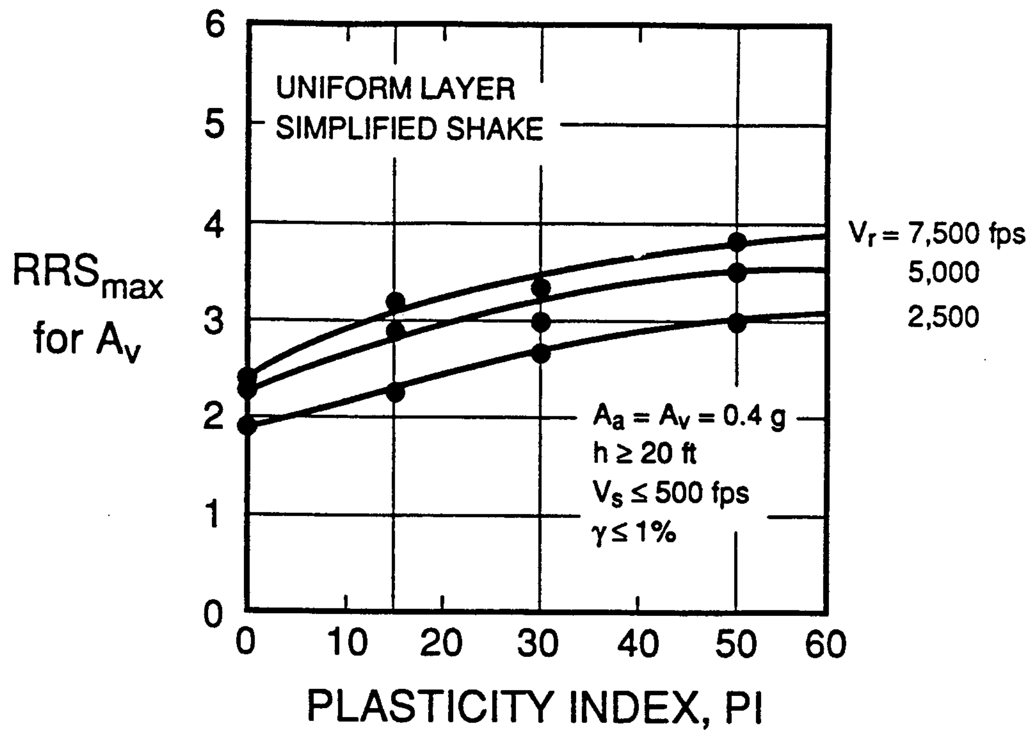


Figure 4-1 Values of RRS obtained from SHAKE analyses and from 1989 Loma Prieta (Dobry et al., 1994)



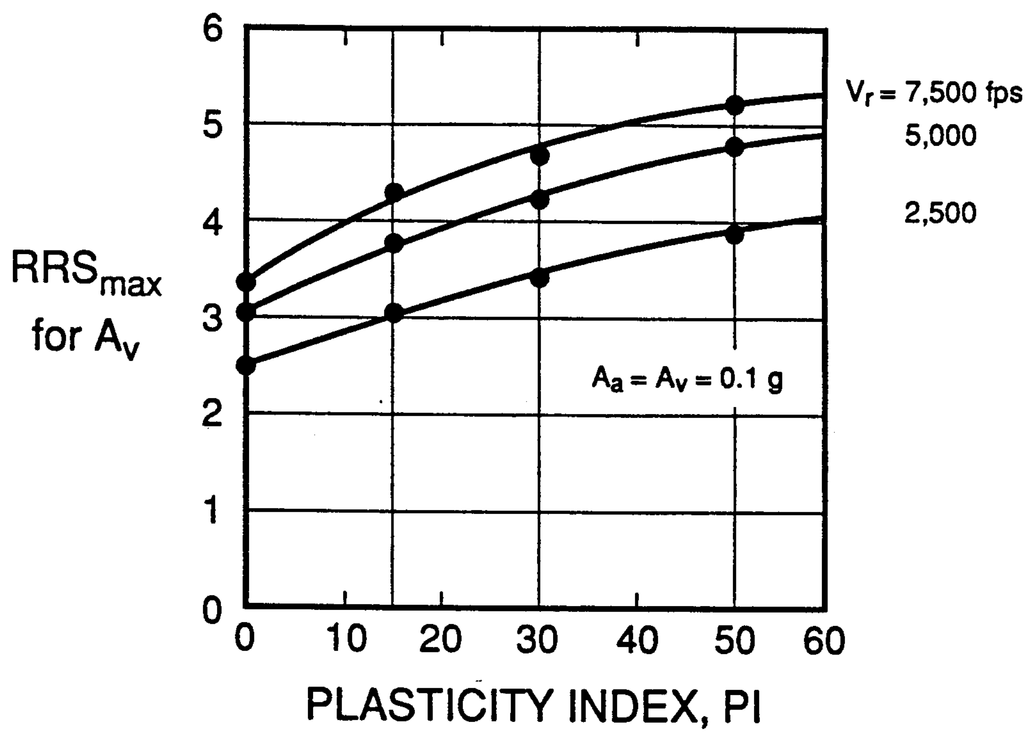


Figure 4-2 Variation of RRS_{max} of uniform layer of soft clay on rock from analyses (Dobry et al., 1994; NEHRP, 1994)

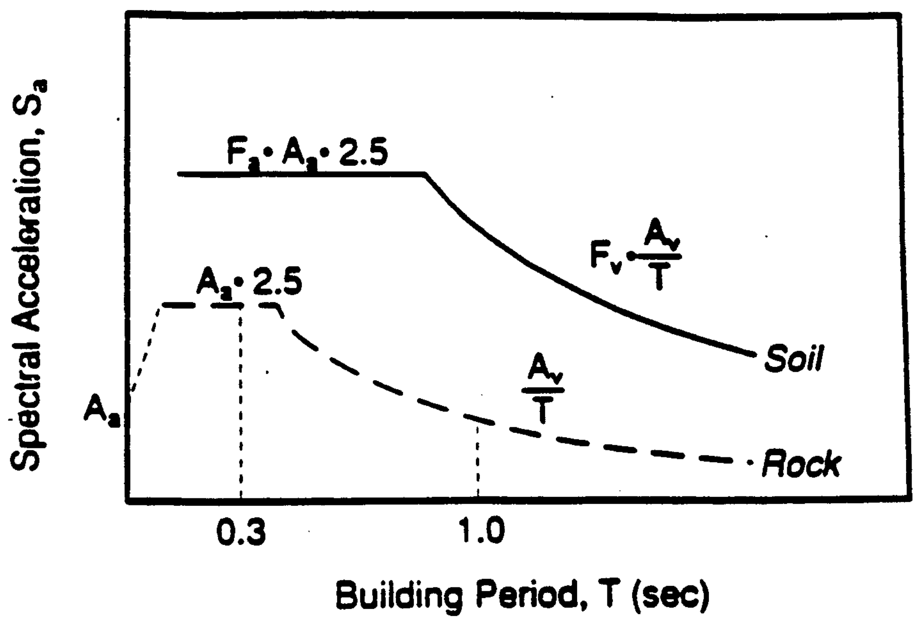


Figure 4-3 Two-factor approach to local site response incorporated into 1994 NEHRP and 1997 UBC (NEHRP, 1994)

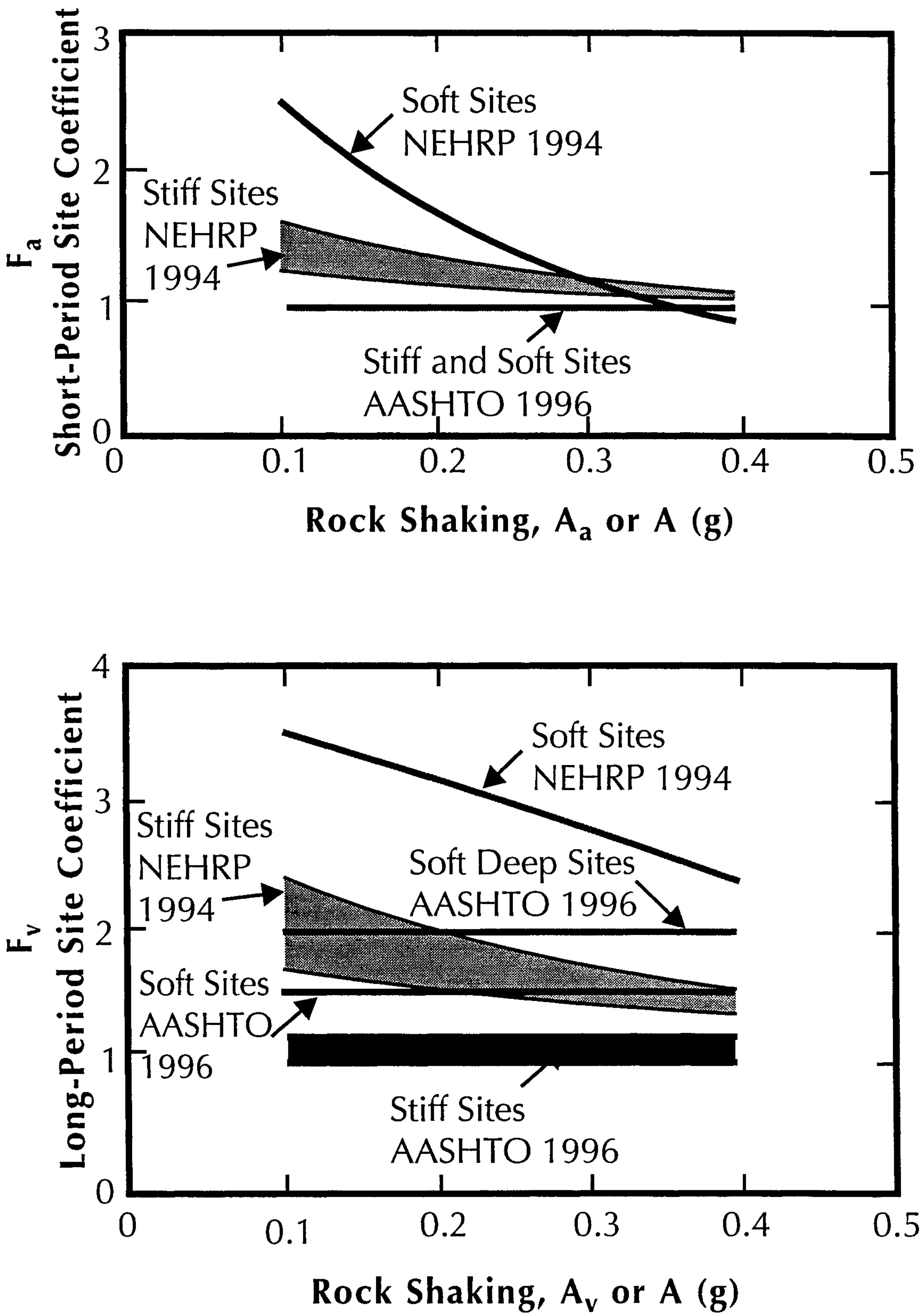


Figure 4-4 Influence of level of rock shaking on short and long period site coefficients at soft (E) and stiff (C and D) sites in new building codes (values from Table 3)

Table 4-1 Site Categories in New Building Codes (NEHRP 1994, UBC 1997)

Soil Profile Type	Description	$\overline{V}_S$ Top 30m (m/s)
A	Hard rock (S1)	>1500
B	Rock	760-1500
C	Very dense soil/soft Rock	360-760
D	Stiff soil (S2)	180-360
E	Soft soil (S3)	<180
F	Special soils (S4) Requiring site-specific evaluation	—

Table 4-2 Use of Geotechnical Parameters to Define Site Categories in New Building Codes (NEHRP, 1994)

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
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)

Site Classification

NOTE: 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).

Table 4-3 Site Coefficients for Short (F_a) and Long (F_v) Periods Contained in New Building Codes (NEHRP, 1994)

Values of F_a as a Function of Site Conditions and Shaking Intensity

Soil Profile Type	Shaking Intensity				
	$A_a \le 0.1$	$A_a = 0.2$	$A_a = 0.3$	$A_a = 0.4$	$A_a \ge 0.5^a$
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	*b*
F	*b*	*b*	*b*	*b*	*b*

NOTE: Use straight line interpolation for intermediate values of A_a.

[a] Values for $A_a > 0.4$ are applicable to the provisions for seismically isolated structures in Sec. 2.6 and certain other structures (e.g., see Table 2.2.4.3).

[b] Site specific geotechnical investigation and dynamic site response analyses shall be performed.

Values of F_v as a Function of Site Conditions and Shaking Intensity

Soil Profile Type	Shaking Intensity				
	$A_v \le 0.1$	$A_v = 0.2$	$A_v = 0.3$	$A_v = 0.4$	$A_v \ge 0.50^a$
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	*b*
F	*b*	*b*	*b*	*b*	*b*

NOTE: Use straight line interpolation for intermediate values of A_v.

[a] Values for $A_v > 0.4$ are applicable to the provisions for seismically isolated structures in Sec. 2.6 and certain other structures (e.g., see Table 2.2.4.3).

[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 4-4 Seismic coefficients on rock and soil for short (C_a) and long (C_v) periods contained in new building codes (NEHRP, 1994)

Seismic Coefficient $C_a = F_a A_a$

Soil Profile Type	$A_a < 0.05$	$A_a = 0.05$	$A_a = 0.10$	$A_a = 0.20$	$A_a = 0.30$	$A_a = 0.40$
A	A_a	0.04	0.08	0.16	0.24	0.32
B	A_a	0.05	0.10	0.20	0.30	0.40
C	A_a	0.06	0.12	0.24	0.33	0.40
D	A_a	0.08	0.16	0.28	0.36	0.44
E	A_a	0.13	0.25	0.34	0.36	0.36

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value of C_a

Seismic Coefficient $C_v = F_v A_v$

Soil Profile Type	$A_v < 0.05$	$A_v = 0.05$	$A_v = 0.10$	$A_v = 0.20$	$A_v = 0.30$	$A_v = 0.40$
A	A_v	0.04	0.08	0.16	0.24	0.32
B	A_v	0.05	0.10	0.20	0.30	0.40
C	A_v	0.09	0.17	0.32	0.45	0.56
D	A_v	0.12	0.24	0.40	0.54	0.64
E	A_v	0.18	0.35	0.64	0.84	0.96

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value of C_v

SECTION 5
NEW (NEHRP, UBC) AND OLD (AASHTO) SITE FACTORS

It is interesting to compare these new site coefficients F_a and F_v contained in 1994 and 1997 NEHRP and 1997 UBC, with the corresponding site coefficients, S, listed in Fig. 3-3 and incorporated into the current, 1996 AASHTO Provisions. Of course, as pointed out before, these values of S are also the same as included in prior versions of NEHRP and UBC before 1994 and 1997, respectively. The corresponding comparisons are plotted in Fig. 5-1 where the curves for F_a and F_v have been replotted from Fig. 4-4. In the upper panel of Fig, 5-1 a site factor of 1.0 has been indicated for AASHTO irrespective of site conditions and level of shaking, as the site factors S in AASHTO 1996 do not apply to the short period range. In the lower panel for Fig. 5-1, the values of F_v and S are compared for the long period range.

It is informative to revisit some of the differences illustrated by Fig. 5-1 between the older site factors included in the current AASHTO Provisions and the newer NEHRP site factors. As mentioned in an earlier section of this report, AASHTO site factors have their origin in the ATC-3 study (Applied Technology Council. 1978). These site factors developed in ATC-3 were based mainly on the analysis of response spectral shapes of predominantly California earthquake data by Seed et al. (1976 a, b) summarized in Fig. 3-2. As already discussed, in this approach it is assumed that site effects do not influence peak ground acceleration. Since site effects on peak acceleration are generally similar to those on short-period spectral values (see Fig. 3-2), this spectral shape approach will underestimate short-period site effects. The NEHRP site factors shown in the upper panel of Fig. 5-1 indicate that, for stiff sites, short-period site effects are present but are relatively modest even at low levels of shaking.

In summary, the site factors incorporated in AASHTO are, in effect, approximately equivalent to those in the current NEHRP provisions at higher levels of shaking, where site effects on peak ground acceleration and on short-period spectral values are very small. The main difference in the NEHRP and AASHTO site factors is at low levels of shaking, where earthquake data clearly show larger effects.

The previous discussion clearly reveals that the seismic design provisions contained in the 1996 AASHTO Standard Specifications for Highway Bridges have been superseded by recent developments including new data and analytical studies, which culminated in the broad consensus reached at the 1992 USC workshop and in the 1994 and 1997 NEHRP as well as the 1997 UBC Provisions for buildings. The differences between the 1996 AASHTO and 1994 NEHRP site coefficients are those illustrated in Fig. 5-1 and discussed in the previous paragraphs. While the NEHRP and UBC provisions may be refined in the future as more data becomes available, they constitute a significant advance from both scientific and practical viewpoints. Therefore, it is recommended that AASHTO be updated in a similar way.

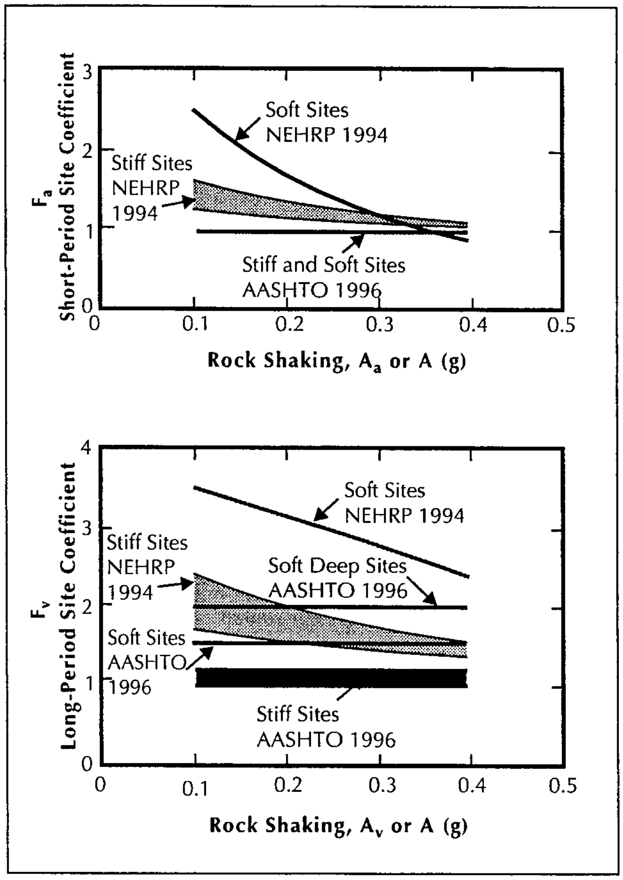


Figure 5-1 Comparison of Site Coefficients Contained in New Building Codes (NEHRP 1994 and UBC 1997) and in Specifications for Highway Bridges (AASHTO 1996)

SECTION 6
RECENT STUDIES

The effort which culminated in the development of the new site coefficients for buildings incorporated into the 1994 and 1997 NEHRP and 1997 UBC seismic provisions, involved a number of empirical and analytical studies as described in a previous section of this paper. These studies more or less summarized the situation prior to 1992 including evaluation of available strong-motion records, and much of this material is included in the Proceedings of the NCEER/SEAOC/BSSC Workshop held at the University of Southern California in November 1992 (Martin, 1994).

Both the adoption of the new site categories and site coefficients, and the fast growth in the number of records in the last few years especially from the 1994 Northridge, California and 1995 Kobe, Japan earthquakes, have stimulated the study of the subject after 1994, with much of this work still ongoing. Studies not yet considering the Northridge and Kobe records include Crouse and McGuire (1996), who conducted a statistical study of records prior to 1992, and generally confirmed the values of the site coefficients F_a and F_v proposed for NEHRP 1994. Midorikawa et al. (1994) and Boore et al. (1997) also conducted analyses of the spectral characteristics of strong motion records obtained prior to Northridge taking into account the soil conditions at the recording stations.

The occurrence of the 1994 Northridge and 1995 Kobe earthquakes accelerated this process even further. Part of the reason is simply the large number of strong-motion records measuring the response of a variety of soils generated by both earthquakes. Both the Northridge and Kobe earthquakes produced more than 200 records each (Borcherdt, 1997b; Sugito, 1995; Ejiri et al., 1996). While the Northridge epicentral area included few if any soft sites, it did have many stations on stiffer soils, and the Kobe earthquake had stations on both stiff and soft soils; both earthquakes produced for the first time a wealth of recordings on various site conditions very close to the source of a destructive event. Many studies of these records and of the exact site and topographic conditions at and near the recording stations are being done, with much of the effort oriented to the verification of the new NEHRP/UBC site coefficients, including their predicted effect of level of rock shaking (soil nonlinearity), influence of the soil and rock below a depth of 30m, and influence of the shape of the valley including basin edge effects and other 2D/3D factors. While studies of these data are still ongoing, it is useful to list here three key references which describe them and contain preliminary findings. These are the Proceedings volumes of three recent technical meetings: (i) the Proceedings of the International Workshop on Site Response held in Yokosuka, Japan in Jan. 1996 (Iai, 1996); (ii) the Proceedings of the North America - Japan Workshop on the Geotechnical Aspects of the Kobe, Loma Prieta and Northridge Earthquakes held in Osaka, Japan in February 1997 (Bardet, Idriss, Adachi, Hamada and Ishihara, 1997); and (iii) the Proceedings of the Northridge Earthquake Research Conference held in Los Angeles in August 1997 (Mahin, 1997).

Section 6-1 and 6-2 contain, respectively, comparisons of site coefficients obtained empirically by the authors from records of the Northridge earthquake, with F_a and F_v included in 1994 and 1997

NEHRP and UBC 1997, and an evaluation of the possibility of a similar analysis for the Kobe earthquake.

6.1 Recorded Ratios of Response Spectra in 1994 Northridge, California Earthquake

Borcherdt (1996, 1997a) conducted a study of the site coefficients in Table 4-3 using the 1994 Northridge records. For that purpose, he obtained Ratios of Fourier Spectra of the soil and corresponding rock records; as usually a rock record was not available at a very close distance from the soil station, he used a rock record of a similar azimuth after correcting for the ratio of distances from the stations to the seismic source. That is, he took advantage of the general similarity between Ratio of Response Spectra (RRS) and Ratio of Fourier Spectra (RFS), already mentioned. Similar to what was done in the studies prior to 1992 which culminated in the new 1994 NEHRP site coefficients, he defined F_a and F_v for each pair of soil-rock stations by taking averages of the RFS, in the period ranges 0.1 to 0.5 sec and 0.4 to 2.0 sec, respectively. That is, for any given pair of stations:

$$F_a(RFS) = \frac{R_{soil}}{R_{rock}} \frac{1}{0.4} \int_{0.1}^{0.5} \frac{FS_{soil}(T)}{FS_{rock}(T)} dT \tag{6-1}$$

where FS_{soil}, FS_{rock} = recorded Fourier Spectra on soil and rock at the same period T, and R_{soil}, R_{rock} = distances of soil and rock station to the zone of largest energy release on the fault at a depth of about 18km. The value of F_a (RFS) reported corresponds to the average of the radial and transverse components (Borcherdt, 1996). Similarly:

$$F_v(FRS) = \frac{R_{soil}}{R_{rock}} \frac{1}{1.6} \int_{0.4}^{2} \frac{FS_{soil}(T)}{FS_{rock}(T)} dT \tag{6-2}$$

As the soil recording stations were mostly on stiff sites, Borcherdt's study allowed verification mostly of F_a and F_v for Soil Profile Types C and D as defined in Tables 4-1, 4-2 and 4-3. In his study, Borcherdt was able to form 31 soil-rock pairs of stations for soil type C, and 20 soil-rock pairs of stations for soil type D, with rock stations being defined as those that would be classified as Soil Profile Type B in Tables 4-1 and 4-2. Borcherdt's calculated values for F_a (RFS) and F_v (RFS) are reproduced in columns 11 and 14 of Tables 6-4 and 6-5 for soil types C and D, for the subsets of pairs of soil-rock stations used in the authors study as described below.

The authors repeated Borcherdt's study for the Northridge earthquake using Ratio of Response Spectra (RRS) instead of Ratio of Fourier Spectra (RFS) to define F_a and F_v, but otherwise following Borcherdt's criteria for selecting the pairs of soil-rock stations, for correcting by the ratio of distances, and for classifying the station sites in the 1994 NEHRP system. That is, the authors used expressions similar to Eqs. 6-1 and 6-2 to compute F_a (RRS) and F_v (RSS), with the following two changes: (i) the Fourier Spectra of soil and rock, FS(T), inside the integrals are replaced by the Response Spectra, RS(T), calculated from the same records, and (ii) R_{soil} and R_{rock} now represent the hypocentral distances of the soil and rock stations instead of the distances to the zone of maximum energy release. This was done because the distances to zone of maximum energy release were not available to the authors; the authors verified by actual calculations that the difference between the ratios R_{soil}/R_{rock} using the two definitions of distance are not very significant and should not affect much the empirically calculated values of F_a and F_v.

The 47 recording stations selected for this study are listed in Table 6-1, and they correspond to recorded acceleration time histories provided in digital form by Geomatrix Consultants, Inc.. They are divided in 27 stations classified as C, 12 stations classified as D, and 8 stations classified as B and used as rock stations in the soil-rock pairs. All of these stations were used in the original Borcherdt study, and their locations are indicated in the map of Fig, 6-1; all 39 soil-rock pair combinations used by Borcherdt were preserved after checking the reasonableness of the selection in the map in terms of relative azimuths and distances between the two stations. In addition to Borcherdt's (1996) classification of the stations in Table 6-1 in terms of the NEHRP 1994 Soil Profile Type, three other sources of information were consulted to verify the site classifications at the stations. The first source was a list of site classifications at all recording stations provided by Geomatrix, using the Geomatrix classification system and designation included in Table 6-2; these designations were translated into the 1994 NEHRP Soil Profile Types using the key defined by the authors in the last column of Table 6-2. The second source of site classifications for the USC (University of Southern California) stations of Table 6-1 was the paper by Trifunac and Todorovska (1996), with their original designations and corresponding key used by the authors for the translation as summarized in Table 6-3. The original Geomatrix and Trifunac-Todorovska classifications are included in the last two columns of Table 6-1; and their translations into NEHRP 1994 Soil Profile Types are listed in columns 7 and 8 of Table 6-1. Finally, the third source was the Web page of Project ROSRINE (Resolution of Site Response Issues from the Northridge Earthquake, http://rccg03.usc.edu/rosrine/index.html), consulted in March 1998. In the ROSRINE project, selected sites affected by the 1994 Northridge earthquake including recording stations are studied to clarify their geotechnical characteristics. Drilling, sampling, in situ wave velocity measurements, and soils laboratory testing are being performed (Pyke, 1997; Schneider et al., 1997). This review of ROSRINE yielded information of in situ shear wave velocity measurements at Station # 27 in Table 5 (Pacoima: Kagel Canyon) that confirmed Borcherdt's classification of the site in the 1994 NEHRP System as a Soil Profile Type C.

Tables 6-4 through 6-8 and Figs. 6-2 through 6-4 summarize the results of this study of F_a and F_v for sites having soil profile types C and D, using Ratios of Response Spectra recorded in the 1994 Northridge earthquake in conjunction with the Borcherdt's soil-rock pairs listed in Tables 6-4 and 6-5. The main results for all soil-rock station pairs are listed in columns 9 and 12 of Tables 6-4 and 6-5, as F_a (RRS) and F_v (RRS) using Method 1. In this report, Method 1, used both by Borcherdt and by the authors, refers to the use of actual rock station records, corrected by distance as shown by the ratio R_{soil}/R_{rock} in Eqs. 6-1 and 6-2. The values of "corrected rock acceleration" at the corresponding soil station listed in column 7 of Tables 6-4 and 6-5 were also obtained using Method 1, that is, multiplying the actual recorded and averaged (between radial and transverse components) acceleration at the rock station, by the ratio of hypocentral distances R_{soil}/R_{rock}. Later in this section, an alternative Method 2 is also used, where the rock records at the locations of the soil stations are generated using an analytical model of the 1994 earthquake.

The average value of F_a (RRS) for the 27 sites type C computed at the bottom of Table 6-4 is $\overline{F}_a$ (RRS) = 1.28, compared with F_a = 1.1 to 1.2 specified by NEHRP 1994 in Table 4-3 for this range of rock accelerations ($A_a \approx (a_p)$ rock $\approx$ 0.1 to 0.3g). For the same 27 sites, the corresponding $\overline{F}_v$ (RRS) = 1.35, compared with F_v = 1.5 to 1.7 specified by NEHRP 1994 for A_v = 0.1 to 0.3g For the 12 sites of softer type D in Table 6-5, and for a narrower range of rock accelerations, $\overline{F}_a$ (RRS) = 1.25 (F_a = 1.5 to 1.6 in NEHRP), and $\overline{F}_v$ (RRS) = 1.50 (F_a = 2.2 to 2.4 in NEHRP). While the scatter of the individual computed values of F_a (RRS) and F_v (RRS) prevents any absolute definite conclusion, with standard deviations in the four cases in the range σ = 0.5 to 0.9 and many individual values of site coefficients less than 1.0, the four computed average values of $\overline{F}_a$ (RRS) and $\overline{F}_v$ (RRS) are all above 1.0, signaling amplification of rock motions by the soil as expected, and with the exception of $\overline{F}_v$ (RRS) for sites type D, are only slightly above or below the NEHRP range. Furthermore, $\overline{F}_v$ (RRS) $\geq$ $\overline{F}_a$ (RRS) for both soil types, consistent with the trend used in the code. Perhaps a better comparison is between the NEHRP code ranges and the ranges defined empirically between $\overline{RRS}$ (where $\overline{RRS}$ denotes either $\overline{F}_a$ (RRS) or $\overline{F}_v$ (RRS)), and $\overline{RRS}$ + 1σ, as done in the last two columns of Table 6-6. It can be seen that now the empirical and NEHRP 1994 code ranges, either overlap or the NEHRP coefficients are larger, with a definite trend for the long-period F_v (RRS) ranges to be above the corresponding short-period F_a (RRS) ranges in both soil types. Therefore, it can be concluded from Table 6-6 that the average values and ranges of F_a and F_v obtained empirically using Method 1 from Ratios of Response Spectra for soil profile types C and D using records of the 1994 Northridge earthquake, are generally consistent with the 1994 NEHRP site coefficients. (That is, the recorded site coefficients are about equal or smaller than the NEHRP values.) The exception is the discrepancy in the coefficients F_a of sites type C, where the ranges in Table 6-6 suggest that the NEHRP coefficients (1.1 to 1.2) may underestimate the true amplification at short periods for some earthquakes (1.28 to 1.80 in this case); additional research is needed on this issue including study of records from other events.

Figures 6-2 and 6-3 present the same data for sites C and D, over the whole period range between T = 0 and T = 2 seconds. For these figures, a statistical analysis was conducted of the Ratios of Response Spectra at each period T, before integrating between 0.1 and 0.5 seconds for F_a (RRS) and between 0.4 and 2 seconds for F_v (RRS). That is, in Fig. 6-2, at each period T, the average $\overline{RRS}$ and $\overline{RRS}$ + 1σ were calculated for all 27 soil-rock station pairs of sites type C, resulting in the two curves shown. Figure 6-3 includes the same information for the 12 pairs of softer soil type D. The NEHRP ranges for F_a and F_v have been superimposed on both plots. Figures 6-2 and 6-3 illustrate the scatter of the individual Northridge results in the period range of interest.

Table 6-7 breaks down the 27 values of F_a (RRS) and F_v (RRS) from Table 6-4 corresponding to soil profile type sites C in three smaller subsets corresponding to rock accelerations, (a_p) rock, centered around 0.10g, 0.15g, and 0.25g, to evaluate the effect of soil nonlinearity. No clear conclusion can be discerned from the data and from the comparisons with the corresponding NEHRP code values listed in the last column. This is not surprising considering the small ranges of variation of F_a and F_v specified by the code for these hard soil sites subjected to rock accelerations that didn't exceed 0.25g or 0.30g, combined with the large scatter of the values of F_a (RRS) and F_v (RRS) obtained from the individual Northridge earthquake records. No evaluation was possible for the effect of soil nonlinearity in the softer sites type D due to the even more limited range of variation of $(a_p)_{rock}$ in this case.

Tables 6-4 and 6-5 also include in column 8 the site amplification ratio, AR, obtained using Method 1 at a period, T = 0. That is, AR is a ratio of peak horizontal accelerations soil/rock, appropriately averaged between radial and transverse components and with the rock acceleration corrected by the ratio of hypocentral distances $R_{soil/rock}$. As expected, the individual values of AR are generally very similar to those of the short-period spectral ratio, F_a (RRS), with a few exceptions; this is confirmed by the similarity in the averages and standard deviations of AR and F_a (RRS) at the bottom of both tables. Figure 6-4 compares all values of AR and F_a (RRS) for all individual sites C and D. This plot confirms that, in Northridge, the amplification of peak rock acceleration and the amplification F_a at short periods was about the same (within about 30%), as expected. This agreement will be used in a later section of this report to discuss the evaluation of soil liquefaction in seismic codes.

In addition to Method 1, already discussed and summarized in Tables 6-6 and 6-7 and Figs. 6-2 through 6-4, an alternative Method 2 was used to obtain estimates of F_a and F_v from empirical Ratios of Response Spectra in the Northridge earthquake. The only difference between Methods 1 and 2 is in the definition of the rock response spectra. In Method 1, the average radial-transverse response spectrum on rock at each soil station was generated by Silva (1997) at the request of the authors using Wald and Heaton (1994) model of strong ground motions in the 1994 Northridge earthquake. This Wald-Silva calculation provided an alternative definition of the rock spectrum at the soil site that did not require any correction for distances, R_{soil}/R_{rock}, between the two stations.

The corresponding calculated values of F_a (RRS) obtained with Method 2 are listed in column 10 of Tables 6-4 and 6-5, while the F_v (RRS) are included in column 13 of the same tables. As shown in these tables, this could be done for all same sites used in Method 1, except for station #15 (Pasadena; 535 South Wilson Avenue). The results of Method 1 including comparisons with the 1994 NEHRP site coefficients, comparisons between Methods 1 and 2, and comparisons between rock records generated using Method 2 and those recorded, are presented at the bottom of Tables 6-4 and 6-5 and in Figs. 6-5 through 6-12.

Both the averages and ranges for F_a (RRS) and F_v (RRS) at the bottom of Tables 6-4 and 6-5, as well as Figs. 6-5 and 6-6, reveal a tendency for the site coefficients computed with Method 2 to be significantly greater than those using Method 1. The only exception is $\overline{F}_a$ for soil profile type D, where $\overline{F}_a$ (RRS) $\simeq$ 1.25 is obtained with both methods.

Another finding is a tendency for the Wald-Silva calculation to provide high values of the rock response spectra in the range of periods between about 1.6 and 2 seconds. This is shown in Fig. 6-7, which summarizes a statistical study for the 7 rock stations # 41 to # 47 in Table 6-1, of the ratios between the rock response spectra predicted by the Wald-Silva analysis and those actually recorded. While over the period range 0 to 1.6 sec, the two sets of spectra were about equal on the average for the 7 stations, for periods above 1.6 sec the calculated spectra are on the average 50% or 100% higher. The same trend is observed in Fig. 6-8, where the response spectra on rock at the locations of the 38 soil stations types C and D, are compared for Methods 1 and 2. This is not surprising, as all 38 rock spectra included in Fig. 6-8 for Method 1 are really the same recorded 7 rock spectra of Fig. 6-7, only now corrected by distance. Therefore, Figs. 6-7 and 6-8 pose a key question: are the records in the 7 rock stations selected biased toward low spectral values in the period range above 1.6 sec, due to chance, or they are not and the Wald-Silva calculation is too high for those 7 stations in this same period range? A comparison between rock spectra simulated by the same Wald-Silva procedure for other rock stations that recorded in 1994, and the corresponding recorded spectra, did not reveal any systematic bias of the procedure toward high spectra in this period range, with the calculated spectra being sometimes above, and sometimes below, the recorded spectra. However, in the 7 rock stations of interest, the Wald-Silva procedure did give systematically larger values between 1.6 and 2 seconds as indicated by Fig. 6-7. Therefore, the information available to the authors does not allow a clear answer to the question above. This points to a source of uncertainty that helps explain the scatter of individual values of F_v (RSS) calculated by both Methods 1 and 2. It also generally points out to the great importance of the way rock records are selected and processed when generating empirical site coefficients F_a and F_v from records on soil.

Figures 6-9 and 6-10 present comparisons between RRS ranges at different periods generated using Method 2, and the corresponding NEHRP specified F_a and F_v; these figures are the counterparts of Figs. 6-2 and 6-3 that used Method 1. Examination of Figs. 6-9 and 6-10 generally confirm the main finding already discussed based on the averages and range at the bottom of Tables 6-4 and 6-5: that the empirical values and ranges of RRS(T), as well as F_a (RRS) and F_v (RRS), are somewhat higher for Method 2 than for Method 1, and thus the empirical ranges for F_a and F_v in Figs. 6-9 and 6-10 tend to be systematically above the NEHRP values. Only for periods longer than 1.6 seconds the opposite trend develops, associated with the high rock response spectra generated by Model 2 in this period range. Finally, Figs. 6-11 and 6-12 present comparisons of the average curves of the empirical RRS (T) obtained by the two methods for sites C and D. The $\overline{\text{RRS}}$ (T) values for Method 2 are generally equal or higher than those of Method 1, except for periods over 1.6 seconds, where the opposite is true for soil sites C.

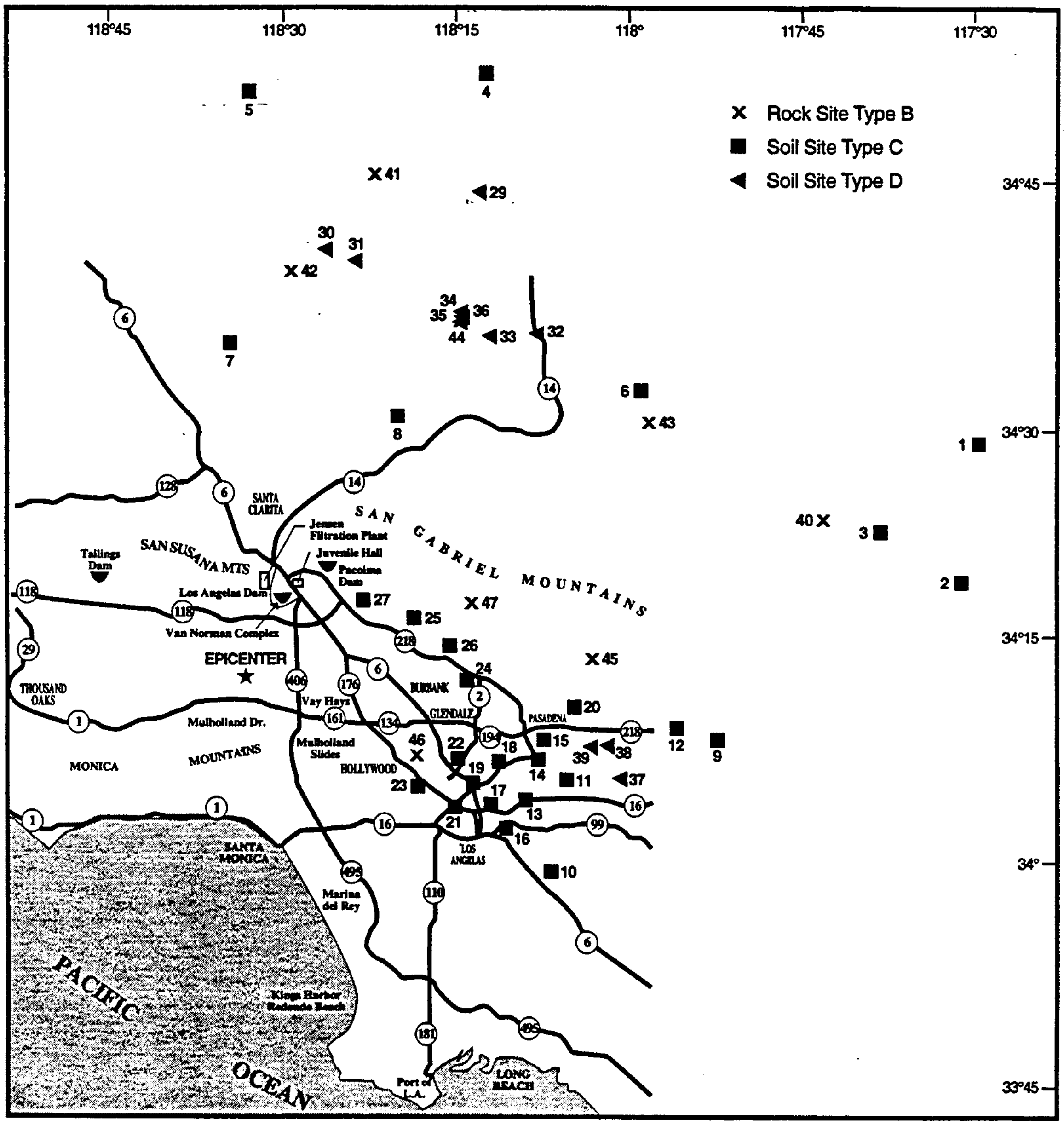
The authors tend to believe that the empirical values of RRS(T), F_a and F_v obtained in this study using Method 1 are more realistic for the set of stations selected than those using Method 2. The argument for this is that actually recorded rock motions were used in Method 1, and that the differences in azimuth and distance to the source between soil and rock stations in the selected soil-rock station pairs were generally mild.

However, as a further check and in an attempt to narrow the uncertainties in Method 1, the two subsets of stations marked with an asterisk (*) in column 1 of Tables 6-4 and 6-5 were analyzed. That is, for soil sites of profile type C, 16 soil-rock station pairs were kept and 11 pairs were discarded, and for soil profile type D, 7 pairs were kept and 5 were discarded. The criterion used to keep any pair was the closeness of the two corresponding soil and rock stations in the map of Fig. 18; any pair which did not look close enough was discarded. In addition, soil station # 21 (LA: Temple and Hope), which is reasonably close to rock station # 46 (LA: Griffith Observatory) in Fig. 6-1, was also discarded due to the uncertainty in the NEHRP classification in

Table 6-1 (soil profile type C by Borcherdt; soil profile type A or B by Geomatrix). It must be noted that the selection of soil-rock pairs with asterisks in Table 6-4 and 6-15 was done by the authors only once, and was based exclusively on the closeness of the two stations in the map, without any consideration to the values of F_a and F_v computed for those pairs.

The calculations of ranges between $\overline{RRS}$ and $\overline{RRS}$ + 1σ for F_a and F_v for these subsets of neighboring soil-rock station pairs are included in Table 6-8, which uses the same format of Table 6-6 for easy comparison. It is interesting that the averages, $\overline{F}_a$ (RRS) and $\overline{F}_v$ (RRS), of the subsets in Table 6-8 are either about equal or greater than those of the complete sets in Table 6-6. The difference is greater for soil profile type D, where $\overline{F}_a$ (RRS) increases from 1.25 to 1.46 and $\overline{F}_v$ (RRS) increases from 1.50 to 1.61. The agreement between empirical and NEHRP ranges is somewhat better in Table 6-8, where all empirical and NEHRP ranges overlap except for F_a of sites C. The comment made previously about F_a of sites C when discussing Table 6-6 is still applicable. In the authors' opinion, Table 6-8 contains the most reliable empirical estimates and verification of site coefficients from the Northridge earthquake presented in this study. Except for F_a of sites type C, the estimated range of F_a and F_v for sites D, and the estimated range of F_v for sites D, are quite consistent with the values specified by the 1994 NEHRP and 1997 UBC codes.

Figure 6-1 Locations of soil and rock stations used for study of site coefficients from records of 1994 Northridge earthquake



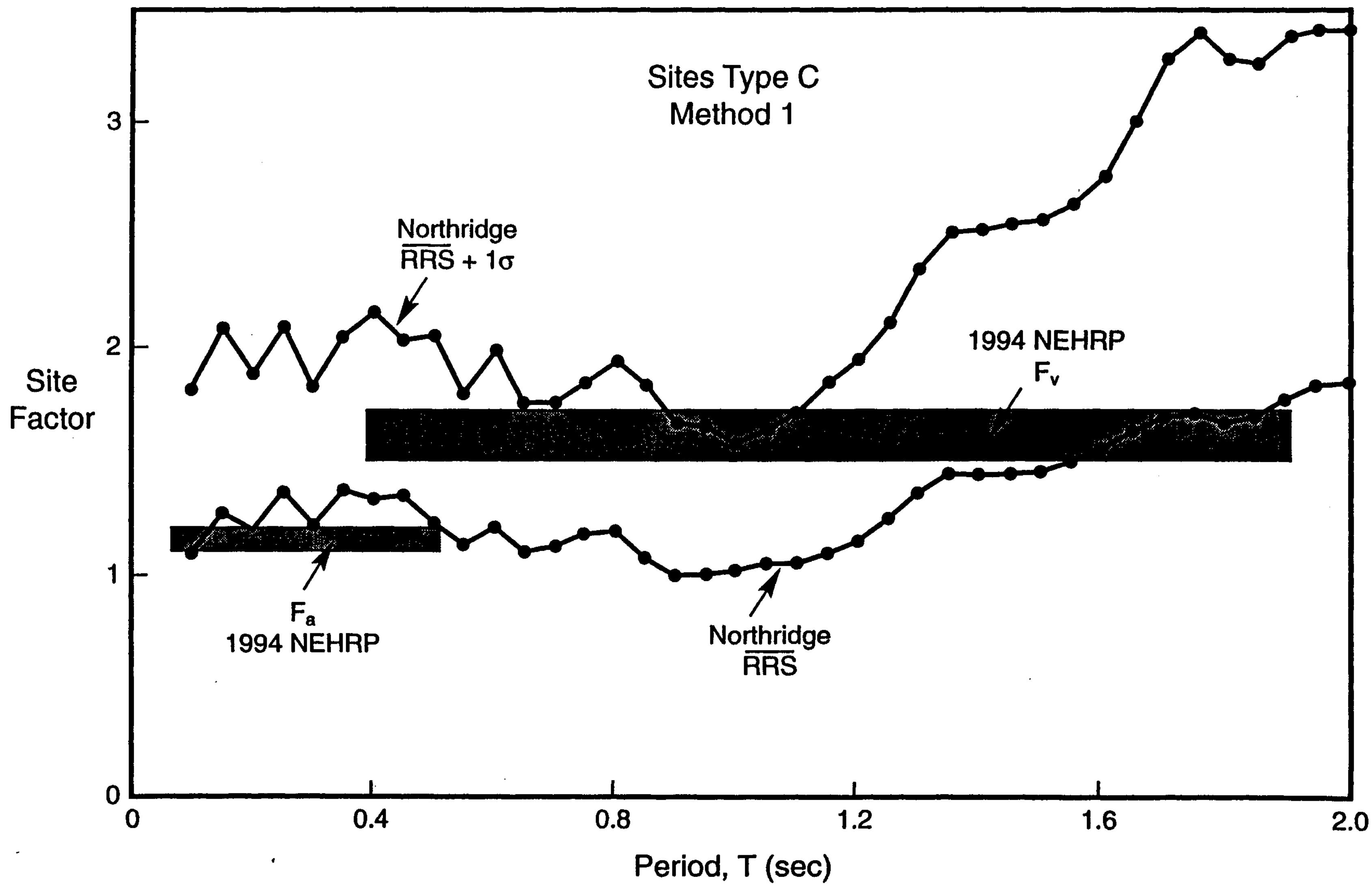


Figure 6-2 Comparison for sites of soil profile type C of RRS obtained from soil-rock station pairs of records (Method 1) in 1994 Northridge earthquake, with F_a and F_v from Table 3 (27 soil sites, $(a_p)_{rock} \simeq 0.04$ to $0.3g$).

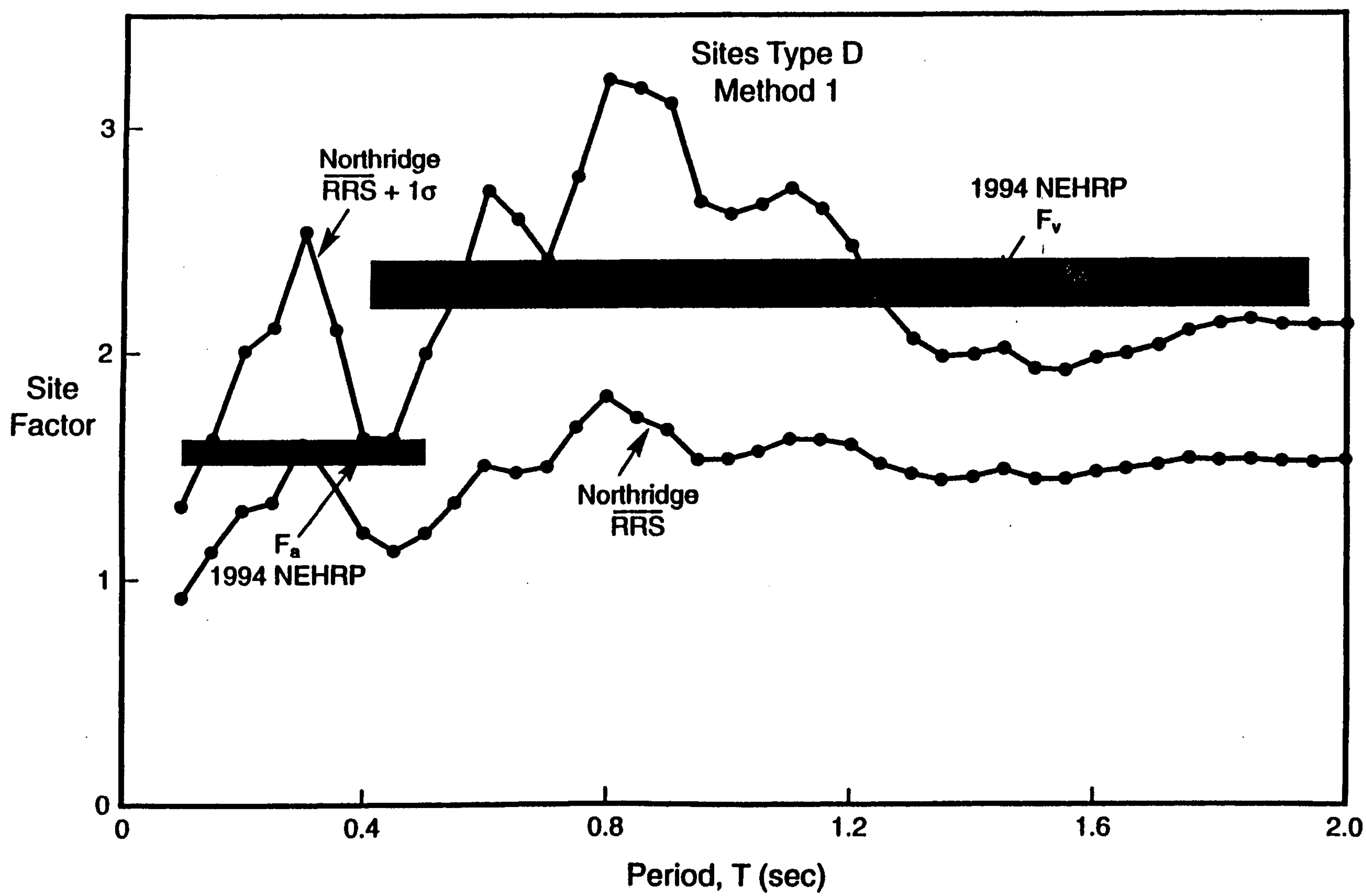


Figure 6-3 Comparison for sites of soil profile type D of RSS obtained from soil-rock station pairs of records (Method 1) in 1994 Northridge earthquake, with F_a and F_v in Table 3 (12 soil sites, $(a_p)_{rock} \simeq 0.04$ to 0.14g)

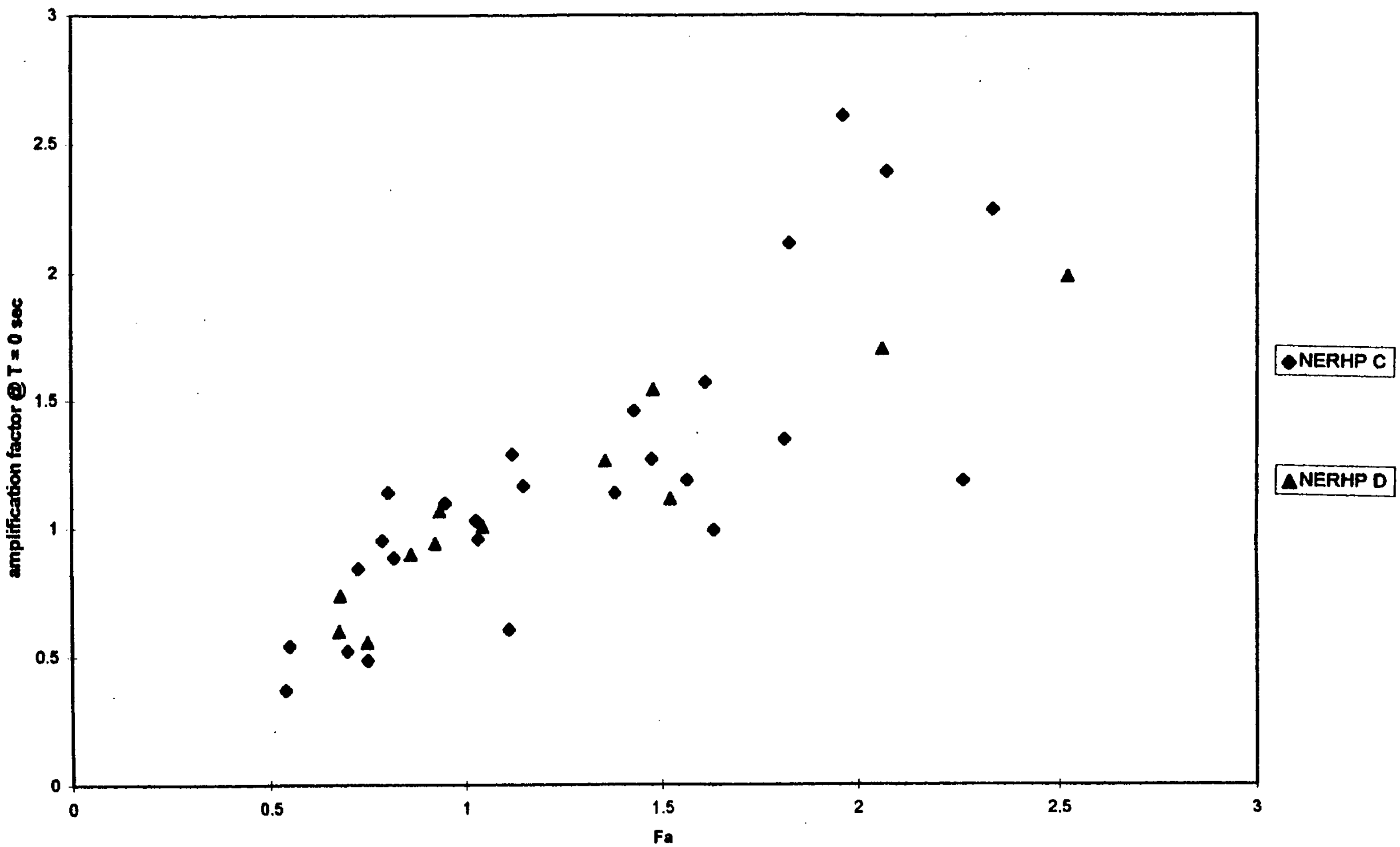


Figure 6-4 Comparison between amplification factors for period range 0.1 – 0.5 sec (F_a) and at a period T = 0 (amplification of peak acceleration), 1994 Northridge earthquake

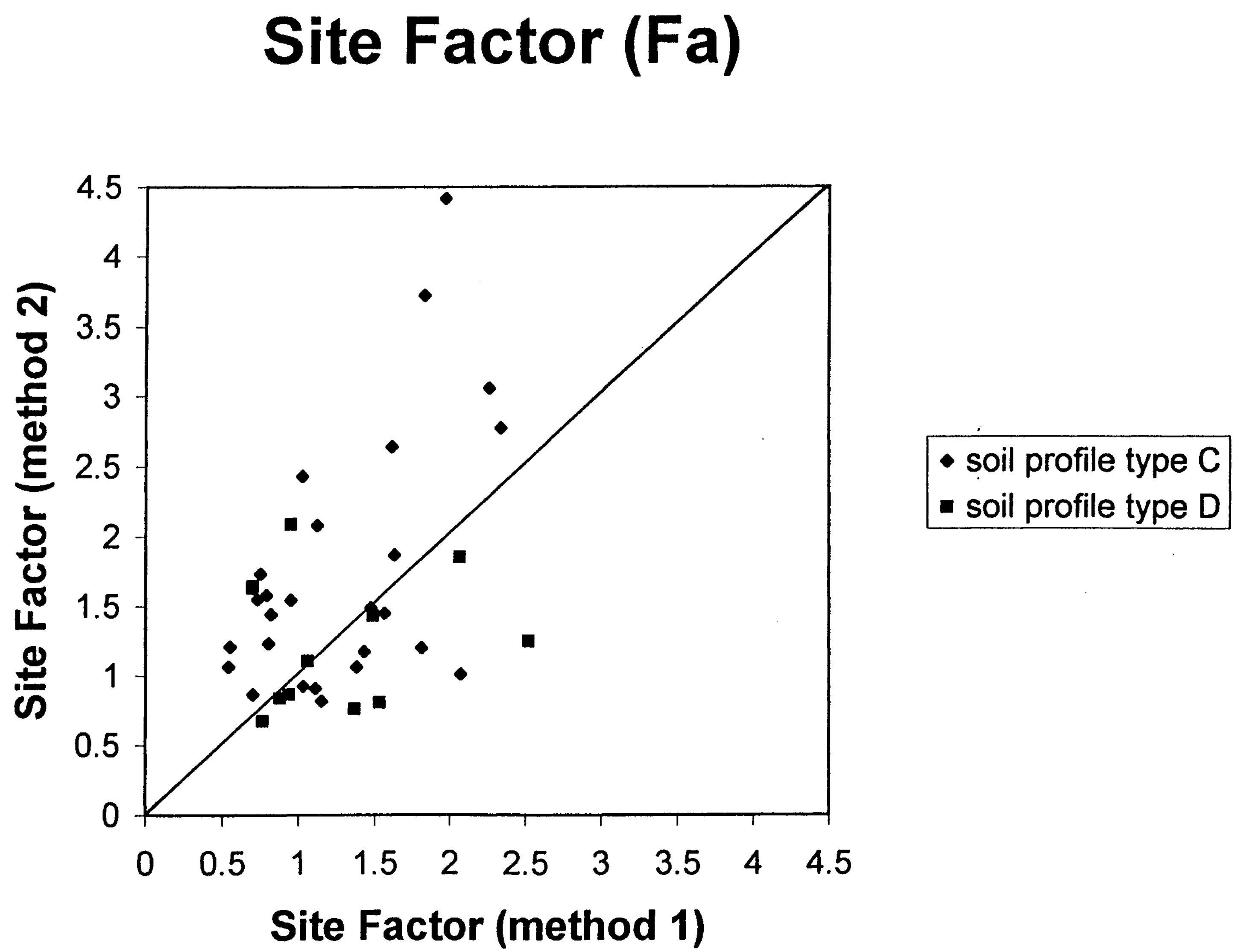


Figure 6-5 Comparison between empirical F_a obtained from $\overline{RRS}$ at Northridge earthquake using Methods 1 and 2

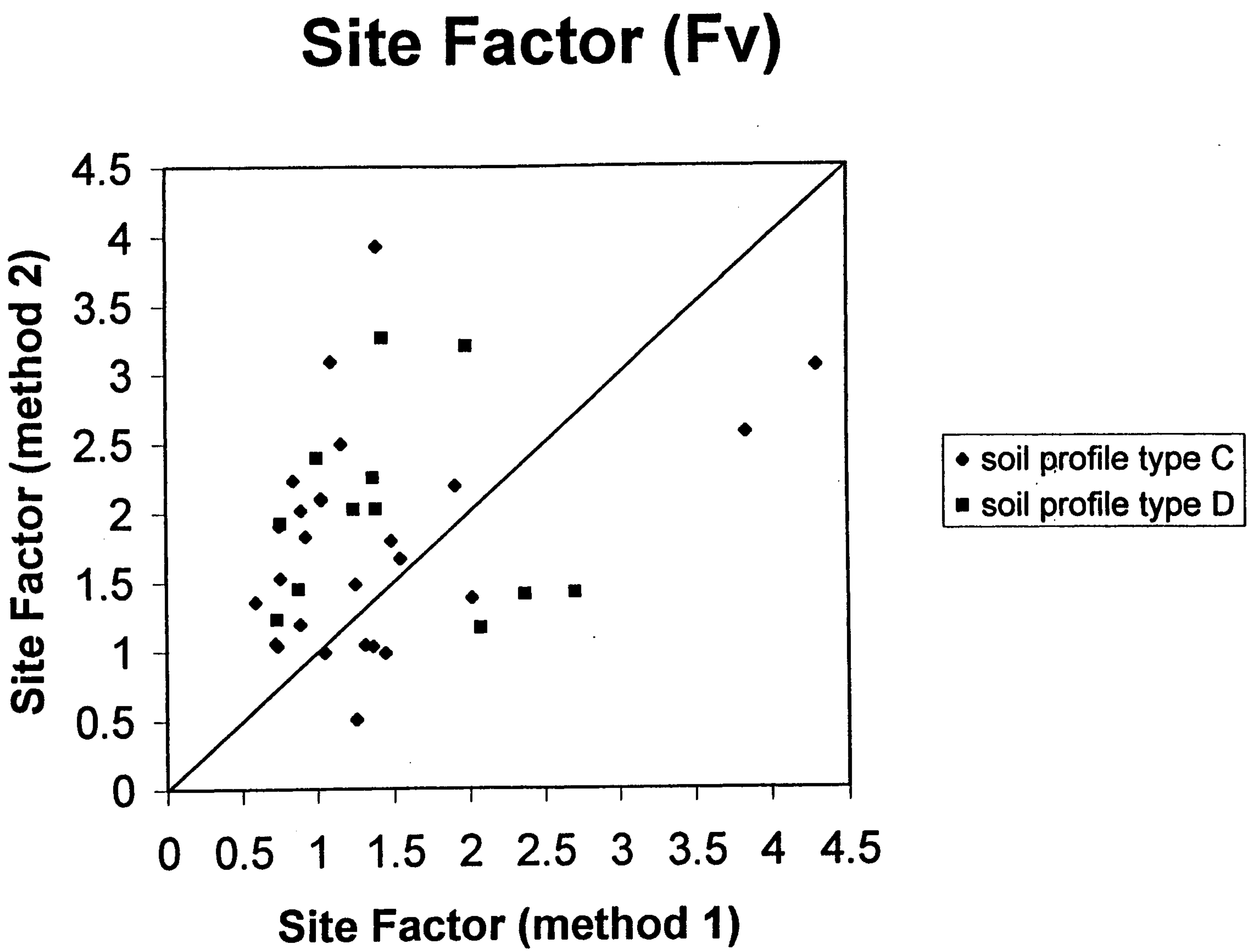


Figure 6-6 Comparison between empirical F_v obtained from $\overline{RRS}$ at Northridge earthquake using Methods 1 and 2

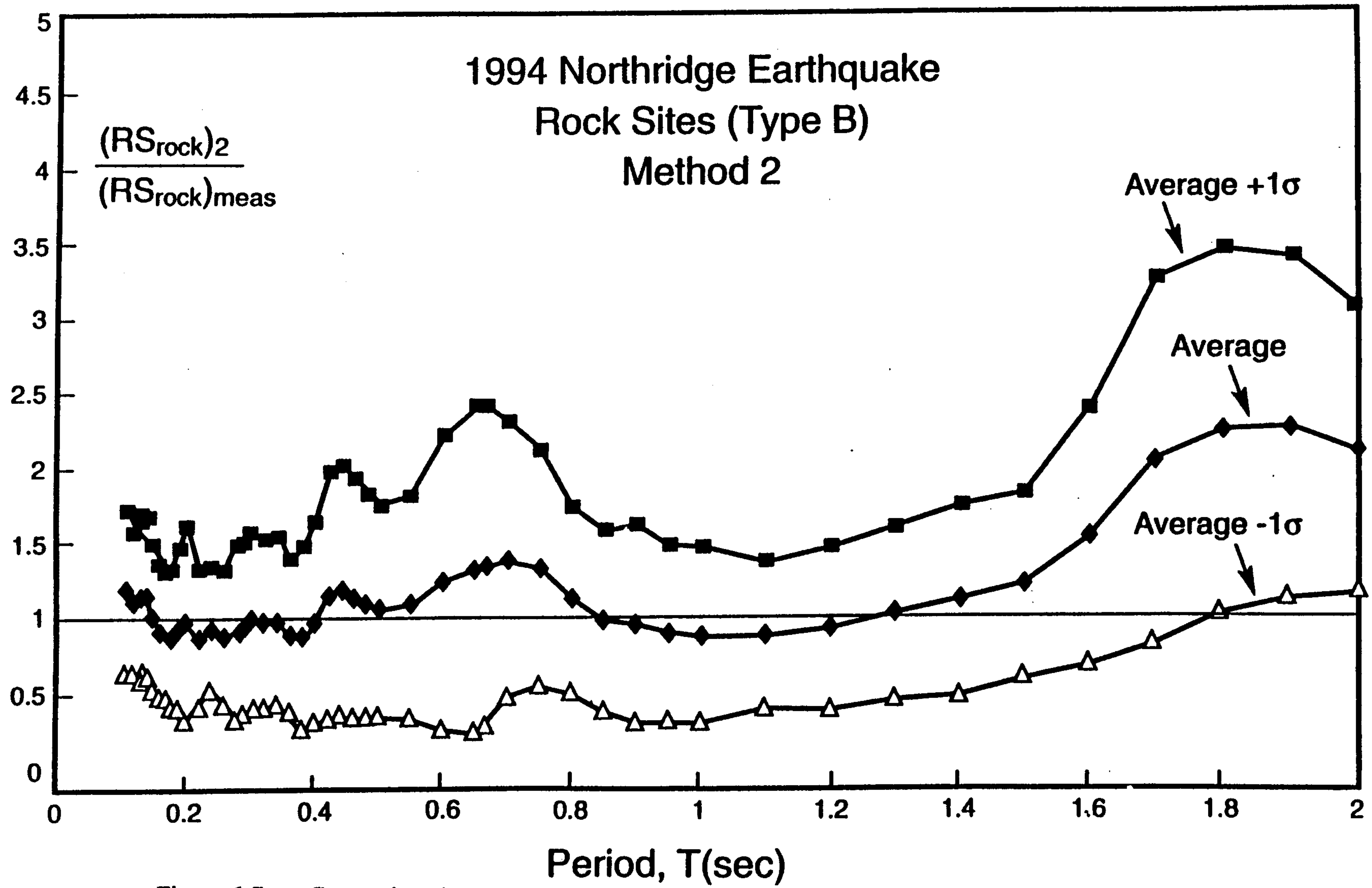


Figure 6-7 Comparison between response spectra calculated at rock stations that recorded in the 1994 Northridge earthquake using Wald and Silva model in Method 2, $(RS_{rock})_2$, and those recorded in the 1994 earthquake, $(RS_{rock})_{meas}$, (7 rock sites)

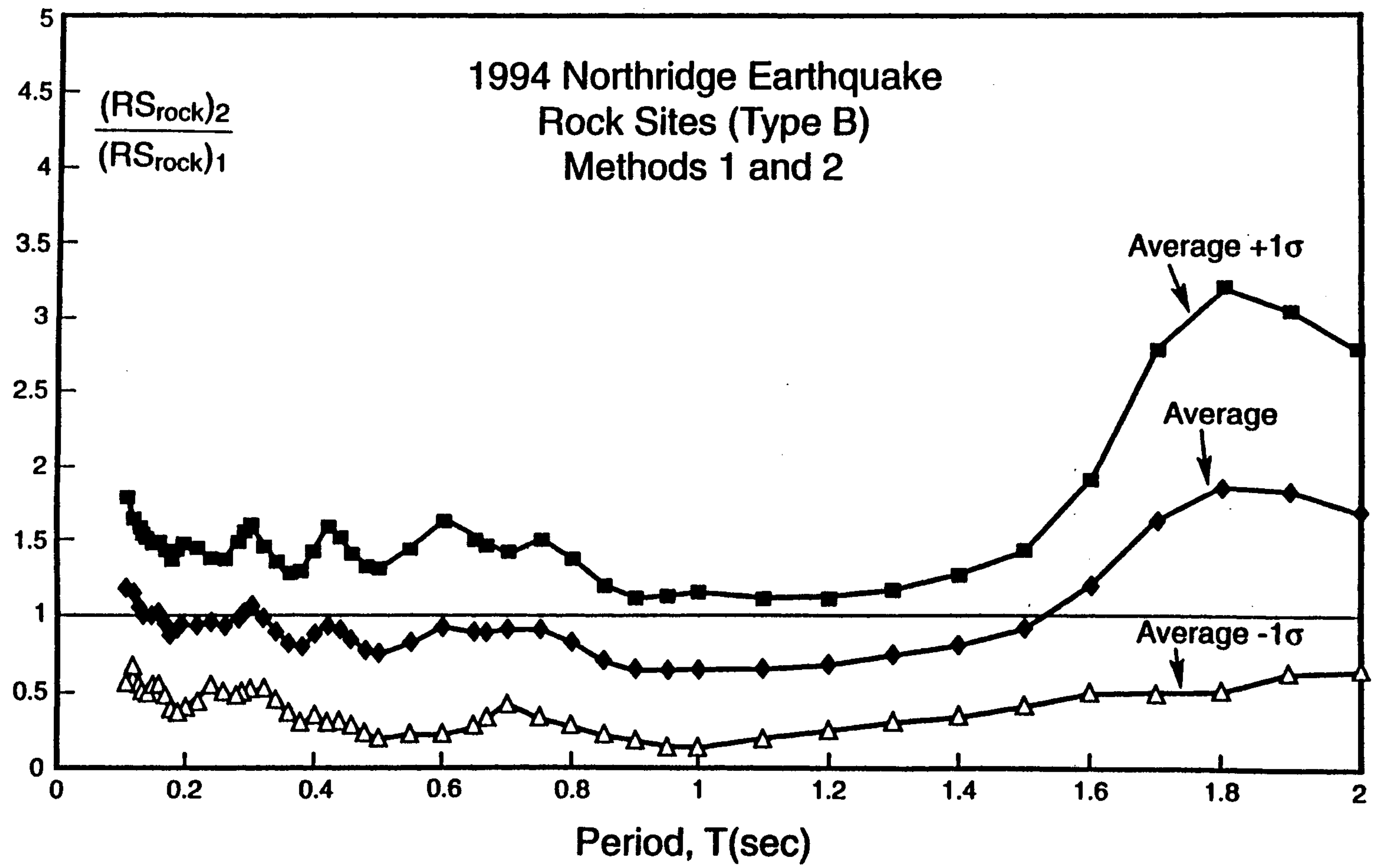


Figure 6-8 Comparison between response spectra on rock calculated at locations of soil stations Types C and D that recorded in the 1994 Northridge earthquake using Wald and Silva in Method 2, $(RS_{rock})_2$, and those obtained from recorded rock records after correcting by distance in Method 1, $(RS_{rock})_1$, (38 soil sites)

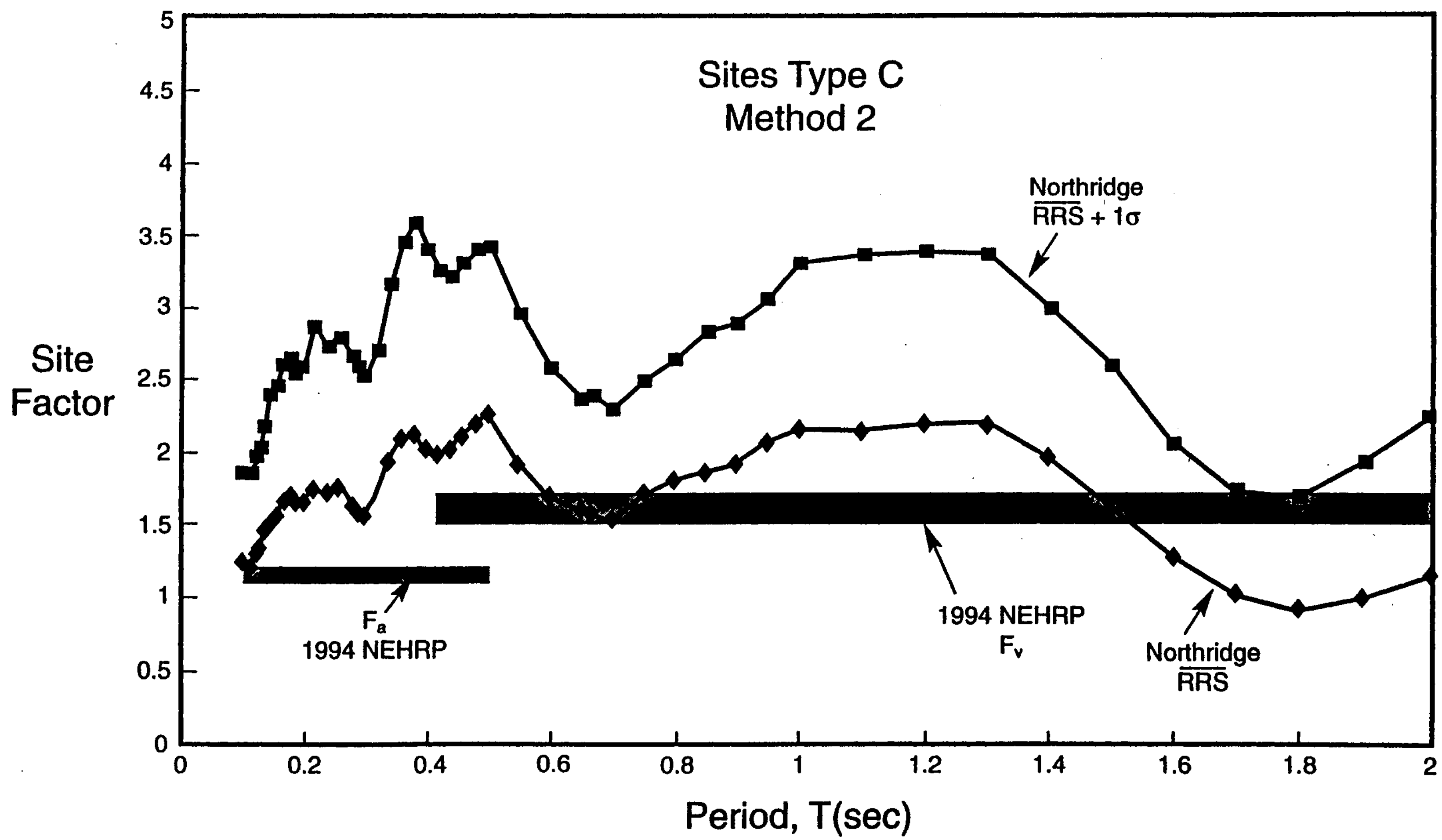


Figure 6-9 Comparison for sites of Soil Profile Type C of RRS obtained from soil stations records of the 1994 Northridge earthquake, using response spectra on rock at soil station locations calculated using Wald and Silva model (Method 2), with F_a and F_v in Table 3 (26 soil sites, $(a_p)_{rock} \simeq 0.04$ to 0.3g)

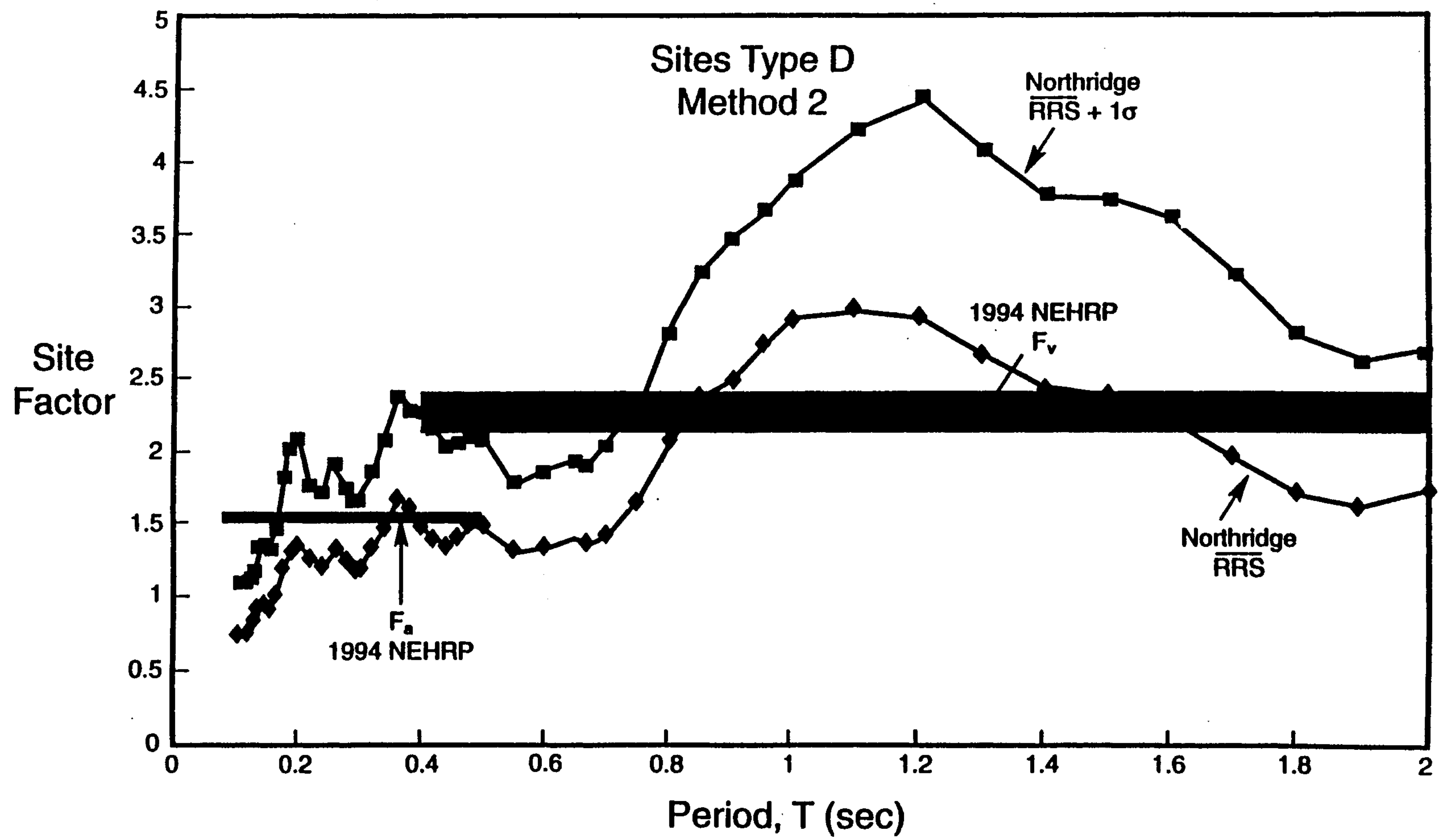


Figure 6-10 Comparison for sites of soil profile type D of RRS obtained from soil stations records of the 1994 Northridge earthquake, using response spectra on rock at soil site locations calculated using Wald and Silva model (Method 2), with F_a and F_v in Table 3 (12 soil sites, $(a_p)_{rock} \simeq 0.04$ to 0.14g)

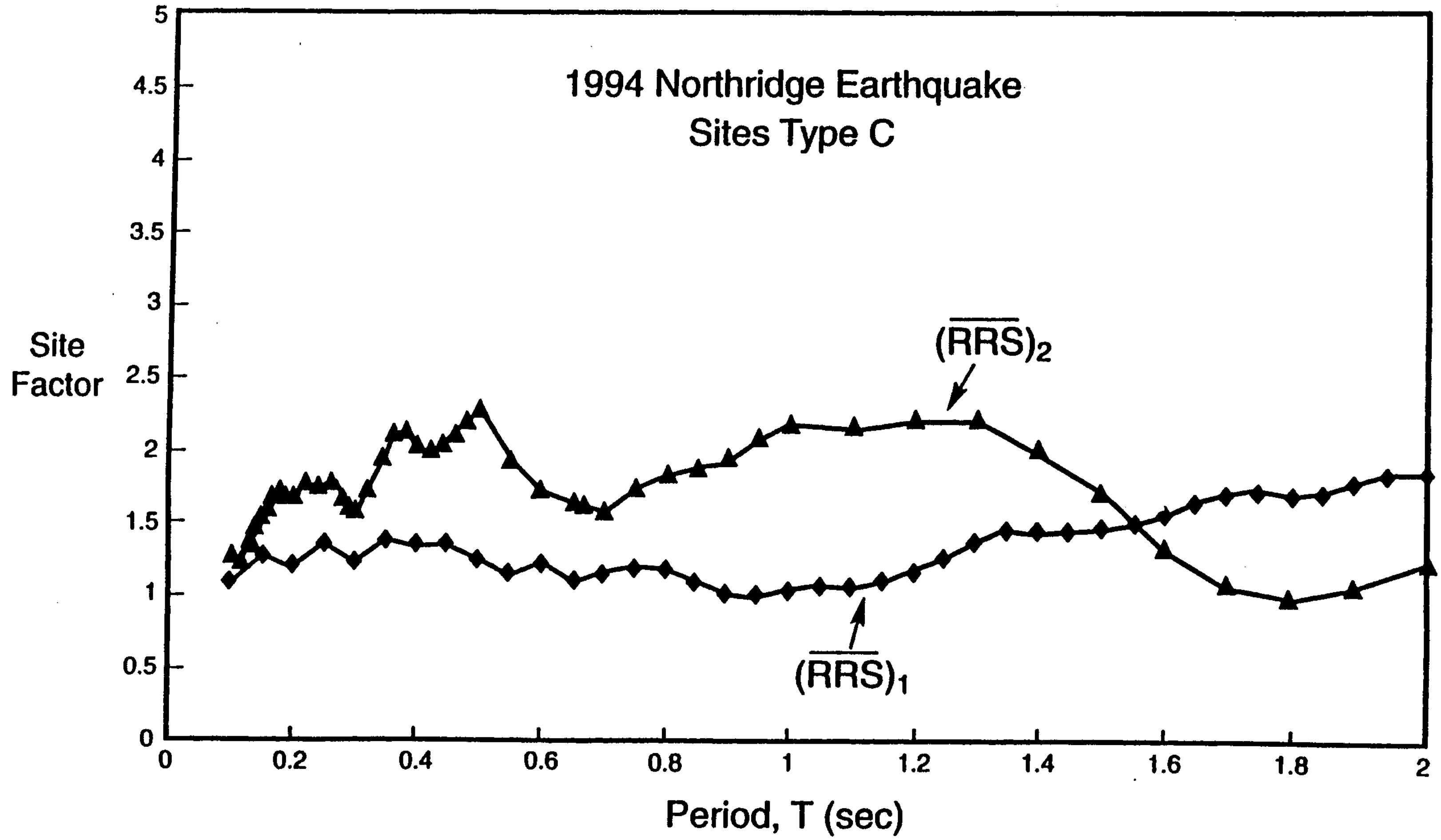


Figure 6-11 Comparison of average Ratio of Response Spectra for 1994 Northridge earthquake using rock spectra from rock records corrected for distance (Method 1), $(\overline{RRS})_1$, and using rock spectra calculated with Wald and Silva model (Method 2), $(\overline{RRS})_2$ (27 and 26 soil sites type C)

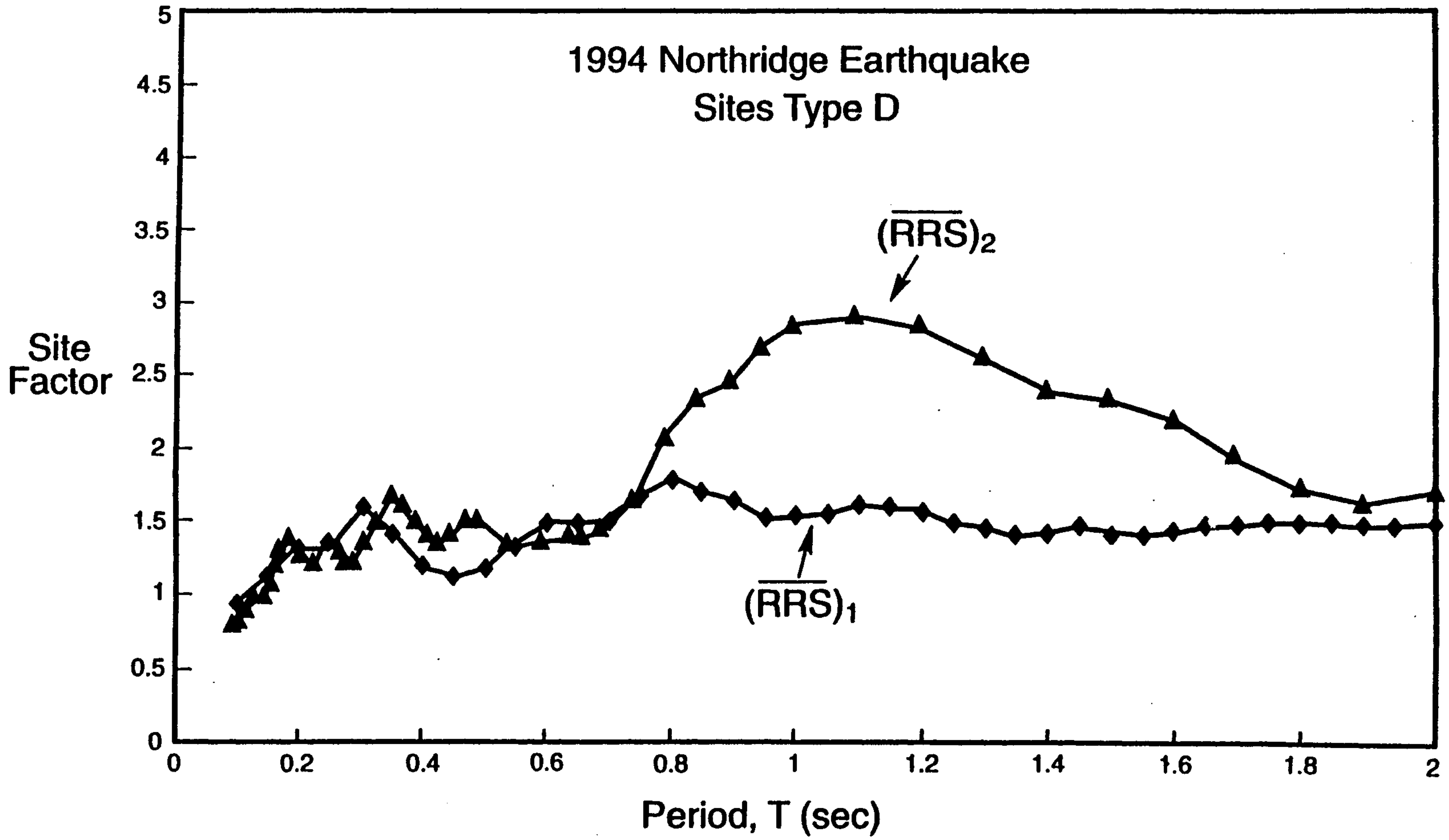


Figure 6-12 Comparison of average Ratio of Response Spectra for 1994 Northridge earthquake using rock spectra from rock records corrected for distance (Method 1), $(\overline{RRS})_1$, and using rock spectra calculated with Wald and Silva model (Method 2), $(\overline{RRS})_2$, (12 soil type D)

Table 6-1 Northridge 1994 Recording Stations Used in the Study and Corresponding Site Classification (Locations of Stations Are Indicated in Fig 6-1)

Station # Map Fig. 18	Station # Earthquake Spectra (1995)	Owner	Symbol Borcherdt (1996)	Name	NEHRP 1994 Soil Profile Type			Original Site Designation	
					Borcherdt (1996) And This Work	GEOMATRIX Translated	Trifunac Translated	GEOMATRIX (see Table6)	Trifunac (see Table 7)
1	23597	CDMG	PWR	PHELAN - WILSON RANCH ROAD	C	D or C		D	
2	23573	CDMG	WNR	WRIGHTWOOD - NIELSON RANCH	C	D or C		D	
3	23574	CDMG	WSV	WRIGHTWOOD - SWARTHOUT VALLEY	C	D or C		D	
4	24092	CDMG	ROS	ROSAMOND - AIRPORT	C	D or C		C	
5	24586	CDMG	NEE	NEENACH SACATARA CREEK	C	D or C		D	
6	5030	USGS	LRP	LITTLEROCK, LITTLEROCK POST OF	C		C		B
7	24607	CDMG	L12A	LAKE HUGHES ARRAY #12: ELIZABETH LAKE	C	D or C	C	D	B
8	24047	CDMG	VRP	VASQUEZ ROCKS PARK:	C	B or A		A	
9	65	USC	GMC	120 N. OAKBANK, GLENDORA, CA	C		C		B
10	11	USC	MTL	1105 BLUFF RD., MONTEBELLO, CA	C		D		C
11	19	USC	SGS	600 E. GRAND AVE., SAN GABRIEL, CA	C		C		B
12	67	USC	DUA	237 MEL CANYON RD., DUARTE, CA	C		C		B
13	24461	CDMG	ALF	ALHAMBRA - FREMONT SCHOOL	C	D or C		D	
14	24401	CDMG	SNM	SAN MARINO - SOUTHWESTERN ACADEMY	C	D or C		D	
15	5296	USGS	PSW	PASADENA, 535 SOUTH WILSON AVE	C		C		B
16	24400	CDMG	OBG	LOS ANGELES - OBREGON PARK	C	D or C	C	D	B
17	24605	CDMG	UHS	LA: 7-STORY UNIVERSITY HOSPITAL, BSMT	C	B or A		A	
18	32	USC	LF2	5921 N. FIGUEROA ST., LOS ANGELES, CA	C		C		B
19	33	USC	LDS	624 CYPRESS AVE., LOS ANGELES, CA	C		C		B
20	95	USC	SMV	1150 N.SIERRA MADRE VILLA AVE., PASADENA	C		C		B
21	24611	CDMG	LAT	LA: TEMPLE & HOPE	C	B or A		A	
22	34	USC	LF1	3036 FLETCHER DR., LOS ANGELES, CA	C		D		C
23	21	USC	LDH	607 N. WESTMORELAND AVE., LOS ANGELES, C	C		C		B
24	63	USC	GLF	3320 LAS PALMAS AVE., GLENDALE, CA	C		D		C
25	58	USC	SUN	10965 MT. GLEASON AVE., SUNLAND, CA	C		C		B
26	60	USC	LCA	4747 NEW YORK AVE., LA CRESCENTA, CA	C		D		C
27	24088	CDMG	PKC	PACOIMA: KAGEL CANYON (**)	C	C or B		B	
28	34093	CDMG	M58	MOJAVE - HWYS 14 & 58	D	D or C		C	
29	24475	CDMG	LNA	LANCASTER - FOX AIRFIELD GROUNDS	D	D or C		C	
30	24271	CDMG	LO1	LAKE HUGHES #1 - FIRE STATION #78	D	D or C	C	C	B
31	24575	CDMG	ELK	ELIZABETH LAKE	D	D or C		D	
32	24521	CDMG	P14	PALMDALE - HWY 14 & PALMDALE BLVD.	D	D or C		D	
33	24576	CDMG	AVY	ANEVERDE VALLEY CITY RANCH	D	D or C		D	
34	24055	CDMG	LV5	LEONA VALLEY #5 - RITTER RANCH	D	D or C		C	
35	24309	CDMG	LV6	LEONA VALLEY #6	D	D or C		D	
36	24306	CDMG	LV2	LEONA VALLEY #2	D	D or C		D	
37	66	USC	EMC	11338 FAIRVIEW AVE., EL MONTE, CA	D		D		C

Table 6-1 Northridge 1994 Recording Stations Used in the Study and Corresponding Site Classification (Locations of Stations Are Indicated in Fig 6-1) (cont'd)

38	93	USC	ARCC	180 CAMPUS DR., ARCADIA, CA	D		D		C
39	99	USC	ARC	855 ARCADIA AVE., ARCADIA, CA	D		D		C
40	23590	CDMG	WJF	WRIGHTWOOD - JACKSON FLAT	B	B or A		A	
41	24310	CDMG	ANB	ANTELOPE BUTTES	B	B or A	A or B	A	A
42	24469	CDMG	LO4	LAKE HUGHES # 4 - CAMP MENDENHALL	B	B or A	D	A	C
43	23595	CDMG	LBC	LITTLEROCK - BRAINARD CANYON	B	B or A		A	
44	24307	CDMG	LV3	LEONA VALLEY #3	B	B or A	A or B	A	A
45	24399	CDMG	MTW	MT. WILSON - CALTECH SEISMIC STATION	B	B or A	A or B	A	A
46	141	USGS	GPK	LOS ANGELES, GRIFFITH OBSERVAT	B	B or A	A or B	A	A
47	61	USC	BTS	BIG TUJUNGA STATION, ANGELES NATIONAL FO	B		C		B

(**) The classification of station # 27 (Pacoima: Kagel Canyon) as NEHRP 1994 soil profile type C was confirmed through examination of the measured shear wave velocity profile posted in April 1998 in the WEB page of project ROSRINE.

Table 6-2 Geomatrix (1996) Site Classification System and Key Used to Translate into NEHRP 1994 System

Geotechnical Subsurface Characteristics	Geomatrix Designation	Translation to NEHRP 1994 Soil Profile Type
Rock. Instrument is founded on rock material ($V_s > 600$ mps) or a very thin veneer (less than 5m) of soil overlying rock material.	A	B or A
Shallow (Stiff) Soil. Instrument is founded in/on a soil profile up to 20m thick overlying rock material, typically in a narrow canyon, near a valley edge, or on a hillside.	B	C or B
Deep Narrow Soil. Instrument is founded in/on a soil profile at least 20m thick overlying rock material in a narrow canyon or valley no more than several kilometers wide.	C	D or C
Deep Broad Soil. Instrument is founded in/on a soil profile at least 20m thick overlying rock material in a broad canyon or valley.	D	D or C
Soft Deep Soil. Instrument is founded in/on a deep soil profile that exhibits low average shear-wave velocity ($V_S < 150$ mps).	E	E or F

Table 6-3 Trifunac and Todorovska (1996) Site Classification System and Key Used to Translate into NEHRP 1994 System

Geotechnical Description	$\bar{V}_s$ (m/s)	Trifunac and Todorovska Designation	Translation to NEHRP 1997 Soil Profile Type
Hard site	>770	A	A or B
Hard site	360-770	B	C
Soft site	180-360	C	D

Table 6-4 Calculated site coefficients from records of soil stations type C (NEHRP 1994) listed in Table 6-1, Northridge 1994 Earthquake

Station #	Soil Station			Rock Station			AR	Fa (RRS) 0.1-0.5 s		Fa (RFS) Method 1	Fv (RRS) 0.4-2.0 s		Fv (RFS) method 1
Map Fig. 18	Name	Hyp Dist (km)	Average Max Acc (g)	Name	Hyp. Dist. (km)	Avg. Max. Acc. Corrected(g)	@ T=0sec	Method 1	Method 2	(Borcherdt 1996)	Method 1	Method 2	(Borcherdt 1996)
1 •	PWR	100.18	0.06	WJF	78.73	0.04	1.46	1.43	1.17	2.50	1.32	1.04	1.43
2 •	WNR	94.44	0.04	WJF	78.73	0.04	1.03	1.03	0.92	1.93	0.74	1.04	0.79
3 •	WSV	85.36	0.05	WJF	78.73	0.05	1.27	1.57	1.45	2.70	0.72	1.06	0.97
4 •	ROS	81.68	0.06	ANB	65.72	0.05	1.19	1.48	1.49	1.56	1.36	1.03	2.19
5 •	NEE	73.25	0.06	LO4	52.52	0.05	1.16	1.15	0.82	0.83	4.30	3.05	3.19
6 •	LRP	64.10	0.15	LBC	62.86	0.07	2.25	2.34	2.77	3.35	1.49	1.80	1.75
7 •	L12A	44.10	0.21	LO4	52.52	0.09	2.39	2.07	1.01	2.52	1.26	0.51	1.37
8	VRP	41.24	0.15	LV3	54.10	0.11	1.35	1.81	1.20	1.91	1.25	1.48	1.45
9 •	GMC	63.85	0.07	MTW	48.17	0.14	1.03	0.75	1.73	0.83	1.55	1.67	1.88
10	MTL	49.67	0.14	GPK	30.37	0.14	0.49	1.03	2.43	1.12	0.75	1.90	1.25
11	SGS	46.94	0.19	GPK	30.37	0.15	1.29	1.12	2.08	1.73	0.85	2.23	1.21
12 •	DUA	58.76	0.06	MTW	48.17	0.15	0.37	0.54	1.07	0.63	0.89	1.19	1.23
13	ALF	43.15	0.09	GPK	30.37	0.16	0.84	0.55	1.21	0.72	0.90	2.02	1.05
14	SNM	43.24	0.14	GPK	30.37	0.16	0.54	0.73	1.55	1.16	0.59	1.36	0.83
15	PSW	42.98	0.15	GPK	30.37	0.16	0.96	1.03		1.28	0.83		1.25
16	OBG	42.53	0.43	GPK	30.37	0.16	2.61	1.96	4.42	2.74	1.40	3.93	1.76
17 •	UHS	39.88	0.37	GPK	30.37	0.17	2.12	1.83	3.72	1.94	1.10	3.09	1.74
18 •	LF2	38.67	0.16	GPK	30.37	0.18	0.88	0.82	1.44	1.13	0.76	1.53	1.03
19 •	LDS	37.00	0.21	GPK	30.37	0.19	1.14	0.80	1.23	1.24	0.93	1.83	1.16
20 •	SMV	46.56	0.22	MTW	48.17	0.19	0.95	2.26	3.06	1.77	1.91	2.19	2.98
21	LAT	36.61	0.18	GPK	30.37	0.19	1.18	0.79	1.58	1.11	1.16	2.50	1.21
22 •	LF1	34.38	0.22	GPK	30.37	0.20	1.10	0.95	1.54	1.13	1.03	2.09	1.40
23 •	LDH	31.99	0.34	GPK	30.37	0.22	1.57	1.61	2.64	1.78	1.03	2.10	1.16
24	GLF	33.78	0.26	MTW	48.17	0.26	0.99	1.63	1.87	1.70	1.05	0.99	2.04
25	SUN	29.13	0.14	BTS	35.29	0.27	0.52	0.70	0.87	0.55	2.02	1.38	2.01
26 •	LCA	32.15	0.16	MTW	48.17	0.27	0.60	1.11	0.91	1.09	1.45	0.98	2.16
27	PKC	25.17	0.35	BTS	35.29	0.31	1.14	1.38	1.06	0.77	3.84	2.57	3.67
					27 sites								
					Average		1.20	1.28	1.74	1.55	1.35	1.79	1.64
					Standard deviation (σ)		0.57	0.53	0.92	0.73	0.86	0.79	0.72
					Avg.+ 1σ		1.77	1.80	2.66	2.27	2.21	2.58	2.35

• Station used in the calculation of Table 12

Table 6-5 Calculated Site Coefficients from Records of Soil Stations Type D (NEHRP 1994) Listed in Table 6-1, Northridge 1994 Earthquake

Station #	Soil Station			Rock Station			AR	Fa (RRS) 0.1-0.5 s		Fa (RFS) Method 1	Fv (RRS) 0.4-2.0 s		Fv (RFS) method 1
Map Fig. 18	Name	Hyp. Dist. (km)	Average Max Acc (g)	Name	Hyp. Dist. (km)	Avg. Max. Acc. Corrected(g)	@ T=0sec	Method 1	Method 2	(Borcherdt 1996)	Method 1	Method 2	(Borcherdt 1996)
28	M58	103.03	0.05	ANB	65.72	0.04	1.27	1.37	0.76	1.15	2.08	1.17	2.67
29	LNA	68.56	0.07	LV3	54.10	0.07	1.12	1.06	1.10	1.10	1.37	2.25	1.56
30 *	LO1	55.65	0.08	ANB	65.72	0.07	1.98	1.53	0.80	1.20	2.70	1.42	3.56
31 *	ELK	55.29	0.14	ANB	65.72	0.07	1.01	2.52	1.25	2.22	2.37	1.41	3.26
32 *	P14	58.60	0.07	LV3	54.10	0.08	0.94	0.94	0.87	1.24	1.39	2.02	1.42
33 *	AVY	54.93	0.05	LV3	54.10	0.08	0.56	0.77	0.67	0.86	0.73	1.23	0.97
34 *	LV5	54.55	0.13	LV3	54.10	0.08	1.70	1.49	1.43	1.47	1.99	3.20	1.89
35 *	LV6	54.76	0.14	LV3	54.10	0.08	1.54	2.07	1.85	2.22	1.24	2.02	1.69
36 *	LV2	54.01	0.08	LV3	54.10	0.08	0.90	0.88	0.84	1.07	0.88	1.45	0.92
37	EMC	53.00	0.14	GPK	30.37	0.13	1.07	0.95	2.09	1.17	1.43	3.26	1.63
38	ARCC	50.71	0.10	GPK	30.37	0.14	0.74	0.70	1.65	0.88	0.76	1.92	1.06
39	ARC	48.83	0.09	GPK	30.37	0.14	0.60	0.69	1.62	0.77	1.00	2.40	1.30
						Average	1.12	1.25	1.24	1.28	1.50	1.98	1.83
						Standard deviation (σ)	0.44	0.58	0.48	0.48	0.65	0.71	0.88
						Avg.+ 1σ	1.56	1.82	1.72	1.76	2.15	2.69	2.71

* Station used in the calculation of Table 12

Table 6-6 Comparison Between Ranges of Site Coefficients Calculated from 1994 Northridge Earthquake Records Using Method 1, and Ranges Specified in NEHRP 1994

Soil Profile Type	Range of Peak Rock Acceleration a_p, A_a, or A_v (g)	Type of Site Coefficient	Northridge Records				F_a or F_v NEHRP 1994
			No. of Records	$\overline{RRS}$	σ	Range between $\overline{RRS}$ and $\overline{RRS}+1\sigma$	
C	0.04 to 0.30	F_a	27	1.28	0.53	1.28 to 1.80	1.1 to 1.2
		F_v	27	1.35	0.86	1.35 to 2.21	1.5 to 1.7
D	0.04 to 0.14	F_a	12	1.25	0.58	1.25 to 1.82	1.5 to 1.6
		F_v	12	1.50	0.65	1.50 to 2.15	2.2 to 2.4

Table 6-7 Influence of Level of Rock Acceleration on Site Coefficients Calculated from 1994 Northridge Records using Method1, and those Specified in NEHRP 1994

Soil Profile Type	Peak Rock Acceleration a_p, A_a, or A_v (g)	Type of Site Coefficient	Northridge Records					F_a or F_v NEHRP 1994
			No. of Records	Range of (g)	$\overline{RRS}$	σ	Range between $\overline{RRS}$ and $\overline{RRS}+1\sigma$	
C	0.10	Fa	8	0.04-0.11	1.61	0.44	1.61-2.05	1.2
	0.15	Fa	9	0.14-0.17	1.06	0.52	1.06-1.58	1.2
	0.25	Fa	10	0.18-0.31	1.21	0.51	1.21-1.72	1.15
C	0.10	Fu	8	0.04-0.11	1.56	1.14	1.56-2.70	1.7
	0.15	Fu	9	0.14-0.17	0.98	0.31	0.98-1.29	1.65
	0.25	Fu	10	0.18-0.31	1.52	0.92	1.52-2.44	1.55
D	0.10	Fa	9	0.04-0.08	1.40	0.58	1.40-1.98	1.6
D	0.10	Fu	9	0.04-0.08	1.63	0.68	1.63-2.31	2.4

Table 6-8 Comparison Between Ranges of Site Coefficients in Northridge 1994 Using Method 1 for Selected Subset of Soil Stations, and Ranges Specified in NEHRP 1997

Soil Profile Type	Range of Peak Rock Acceleration a_p, A_a, or A_v (g)	Type of Site Coefficient	Northridge Records				F_a or F_v NEHRP 1994
			No. of Records	$\overline{RRS}$	σ	Range $\overline{RRS}$ to $\overline{RRS}+1\sigma$	
C	0.04 to 0.30	F_a	16	1.36	0.56	1.36 to 1.92	1.1 to 1.2
		F_v	16	1.37	0.85	1.37 to 2.22	1.5 to 1.7
D	0.07 to 0.08	F_a	7	1.46	0.06	1.46 to 2.12	1.5 to 1.6
		F_v	7	1.61	0.75	1.16 to 2.36	2.2 to 2.4

6.2 The 1995 Kobe, Japan Earthquake

As mentioned before, the 1995 Kobe earthquake produced a number of records on rocks and stiff and soft soils, some of them very close to the fault and corresponding to high accelerations in excess of 0.4g. This is very relevant to the verification of the site coefficients developed in NEHRP 1994 and 1997 and UBC 1997 for rock levels of shaking, A_a and A_v, equal or greater than 0.3g, especially those on soil profile types D and E in Table 4-3. For these high levels of shaking, very relevant to high seismic areas such as California, recordings on soft soils were scarce until Kobe, and thus the values of F_a and F_v contained in the 1994 NEHRP Provisions were largely based on analytical extrapolations of measurements at lower shaking levels (e.g., Figs. 3-1 and 4-2). These analytical extrapolations include the significant decrease in the values of site coefficients F_a and F_v in Table 4-3 as A_a and A_v increases, due to soil nonlinearity at soil profile types D and E.

Figure 6-13 shows the locations of ground recording stations in relation to the fault rupture in the 1995 Kobe earthquake, with indication of the recorded horizontal peak ground accelerations, a_p. As noted by Ejiri et al. (1996), values of a_p greater than 0.4g were recorded at 17 rock and soil stations near the fault rupture. (In Figs. 6-13 through 6-15, a_p is called PHGA = Peak Horizontal Ground Acceleration, and units of gal are used, with 1g = 1000gal). Figure 6-14 shows the attenuation of a_p with distance to the fault, where D is defined as the closest distance from the recording site to the ground surface projection of the fault rupture. In this figure, different symbols are used for rock, stiff, normal, and soft soil, where the definitions of these site categories correspond to those proposed by the Design Code for Bridges in Japan (1990), as listed in Table 6-9. Most of the rock and stiff sites which recorded a_p of 0.4g or greater are located at distances D < 10 km from the fault, and in fact the mean attenuation curve for these sites drops below 0.4g exactly at D = 10 km. Therefore, rock and soil stations at these very close distances, and especially stations located at normal and soft soil sites, are of great interest to the verification of soil nonlinearity at high levels of shaking.

Table 6-9 includes the key that in the authors' opinion should be used to translate the Japanese site classifications listed there to the NEHRP soil profile types of Tables 4-1 through 4-4. This key is based on both the word descriptions in Table 6-9 as well as the statements by Midorikawa and Kobayashi (1980) and Midorikawa (1980) cited by Goto et al. (1996), as understood by the authors, that the boundary between stiff and normal sites in Table 6-9 corresponds approximately to $\overline{V}_s$= 500 m/sec, while that between normal and soft sites it corresponds to $\overline{V}_s$ = 300 m/sec, where $\overline{V}_s$ is the average shear wave velocity at the top 30 meters of the profile at the site. Therefore, the "stiff ground" designation in Fig. 6-15, where Ejiri et al. lump together the rock and stiff sites of Table 6-9 and Fig. 6-14, would correspond to either sites A, B or C of NEHRP, while the "soft ground" designation in Fig. 6-15 would correspond to either sites C, D or E of NEHRP (or to sites F for sites that liquefied in Kobe, such as Port Island, which was very close to the fault and recorded very strong shaking at depth).

The four stations labeled "soft soil" in Fig. 6-14 within 10 km to the fault do plot lower than the average of the rest of the stations, suggesting deamplification of accelerations on soft sites at these very high levels of shaking. In fact, one could be tempted to take the three soft soil stations that recorded a_p of about 0.31g on the plot, and compare this value to a_p = 0.57 g, corresponding to the mean attenuation curve for rock and stiff soils at the same distance. The corresponding data point would be roughly consistent with the plot for acceleration of soft sites extrapolated analytically by Idriss (1990a, b) in Fig. 3-1. This would provide a very low acceleration ratio, AR = 0.31/0.57 = 0.54. However, there are several problems with this type of exercise. The first is that one of these soft soil stations is Port Island, that liquefied and thus corresponds to NEHRP site type F for which site coefficients are not specified; the exact site conditions at the other two to three soft soil stations of interest, or if any of them also experienced liquefaction, is not clear to the authors. A second problem is that one of these four stations labeled "soft soil" in Fig. 6-14, at D = 1 km, experienced a_p > 0.6 g . A third problem is the huge range of variation of a_p of rock and very stiff site stations at these locations very close to the fault, with a_p ranging from values below 0.3 g to about 0.8 g, that is a factor of almost three, which makes it very difficult to ascertain with any degree of precision the level of shaking on rock at the location of a specific soil station before dividing values of a_p or of response spectra of soil/rock, as done for Northridge in the previous section. Important factors contributing to this scatter of a_p are the sensitivity of a_p to the exact location of the station in relation to the fault rupture, that is the radiation pattern/directivity effect, and the fact that at least one of the rock stations (Kobe University) was not located at the ground surface but at some depth (Sugito, 1995; Ejiri et al., 1996; Somerville, 1996; Bardet et al., 1997). As a result of these difficulties, different authors have arrived to different conclusions about the amplification issue for the Kobe earthquake: Ejiri et al. (1996) notices deamplification of peak accelerations on soft soil, while Midorikawa at al. (1996) does not find any significant site effect.

In an effort to eliminate the very important effects of radiation pattern and directivity in the evaluation of a_p, Ejiri et al. (1996) defined an "equivalent hypocentral distance," X_{eq}, as a weighted average of the distances of the station from 600 square segments that ruptured on the fault plane. Figure 6-15 includes the corresponding attenuation relations for a_p for Kobe 1995 based on X_{eq}, where all the small values of X_{eq} correspond to Zone A in front of the fault rupture (Fig. 6-13). Two plots are included in Fig. 6-15: one for "stiff ground" that lumps together the sites labelled "rock" and "stiff" in Table 6-9 and Fig. 6-14 (A through C in NEHRP), and one for "soft ground", which lumps together the sites labelled "normal" and "soft" (C through E in NEHRP, once the cases of liquefaction are eliminated). Figure 6-15 indeed shows less scatter than Fig. 6-14, thus justifying the selection of the equivalent distance X_{eq}.

The authors selected the data in Fig. 6-15 for X_{eq} < 22.2 km , corresponding to a_p > 0.25 g in the curve labelled "PHGA Attenuation in A" on the figure for stiff ground. The corresponding values of X_{eq} and PHGA = a_p for the 7 stiff ground and 9 soft ground stations were digitized and listed in Tables 6-10 and 6-11 for stiff and soft grounds (the data point for Port Island in Fig. 6-15 was not included). It seems that the attenuation curve for Zone A and stiff ground in Fig. 6-15, corresponding to an equation fitted to values of X_{eq} both smaller and greater than 22.5 km, overestimates the value of a_p of most of the 9 stiff ground stations of interest, corresponding to small X_{eq}. Therefore, the authors fitted by least squares the linear equation between log a_p and log X_{eq} listed at the bottom of Table 6-11, to these 9 stiff ground stations at X_{eq} < 22.5 km. In Tables 6-10 and 6-11, values of a_p calculated with this expression are denoted as $(a_p)_{stiffeq}$.

The acceleration ratios, AR = $a_p/(a_p)_{stiffeq}$ for stiff ground sites are listed in the last column of Table 6-11. As expected, they fluctuate around 1.0 (range: 0.8 to 1.5), and their average at the bottom of the table is 1.05. In Table 6-10, a similar exercise is conducted in computing AR for the soft ground sites. The average is again 1.05 (range: 0.6 to 1.7), thus failing to reveal any general trend of deamplification of peak accelerations at these soft ground sites at levels of rock/stiff sites accelerations of the order of 0.6 g. A similar conclusion is obtained if one looks at the averages and ranges of a_p in the same tables, which are about 0.6 g and 0.3 to 0.8 g irrespective of site condition.

Further research of these and other records on rock and stiff and soft soils that correspond to very high levels of shaking in the 1995 Kobe earthquake is recommended. Careful evaluation of all relevant effects as well as better documentation of the geotechnical conditions at the stations and of the depth and other installation data for the instruments, are necessary before soil and rock accelerations and spectra can be compared for evaluating site effects and code site coefficients.

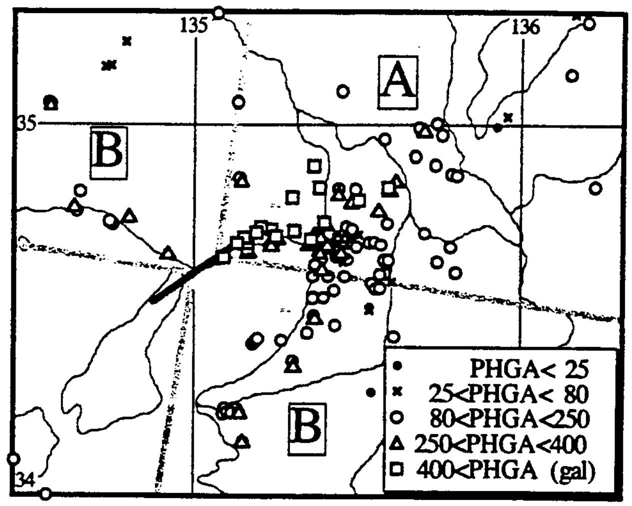


Figure 6-13 Distribution of peak ground horizontal accelerations with respect to the fault rupture in 1995 Kobe earthquake (Ejiri et al., 1996)

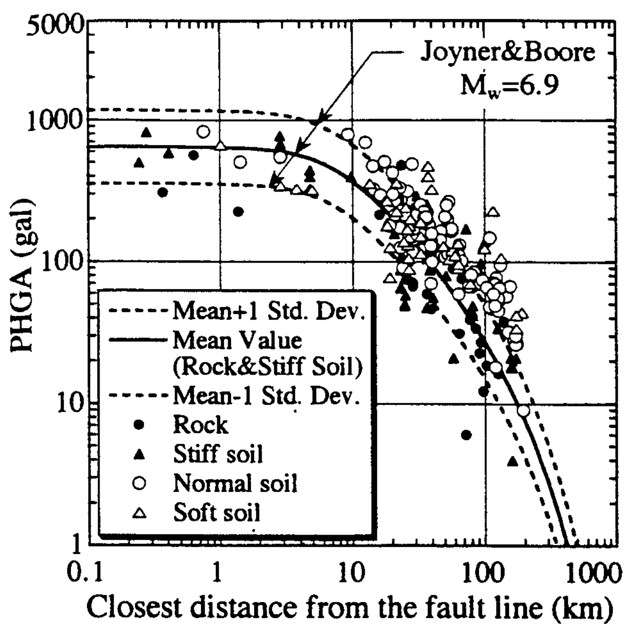


Figure 6-14 Attenuation of peak ground acceleration with distance for different site conditions in 1995 Kobe earthquake (Ejiri et al., 1996)

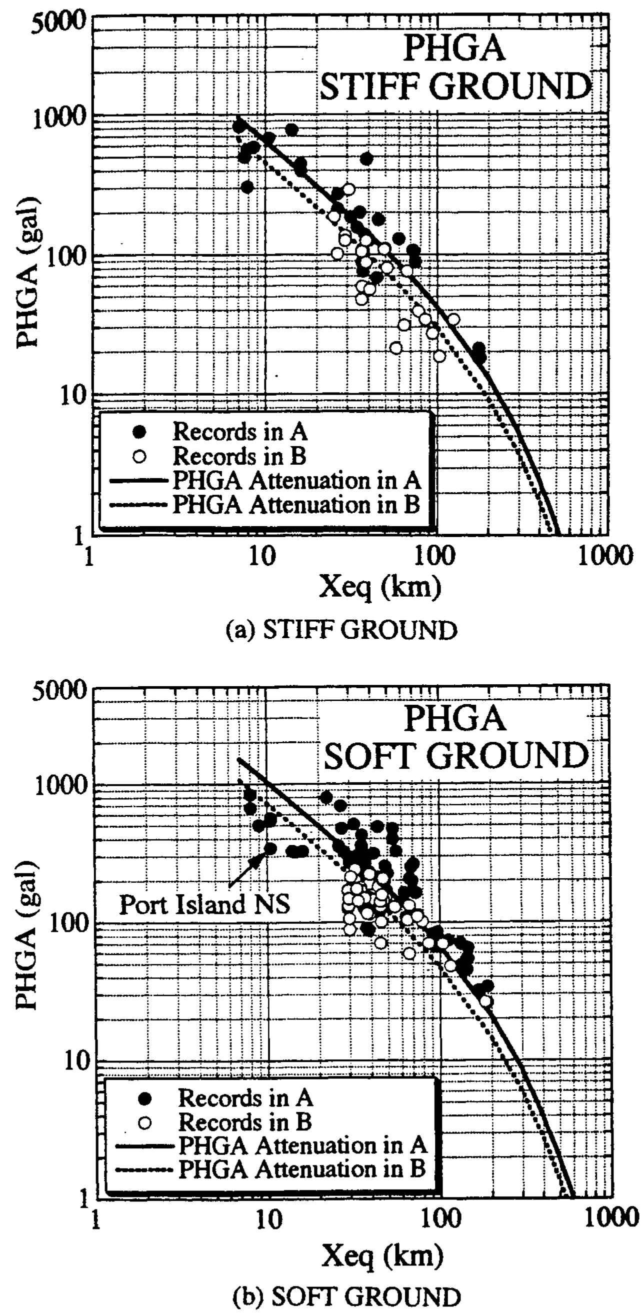


Figure 6-15 Attenuation of peak ground acceleration versus "Equivalent Hypocentral Distance," Xeq, 1995 Kobe earthquake

Table 6-9 Site Classification Used by Ejiri et al (1996) For the Study of Kobe Earthquake Records in Figs. 6-14 and 6-15

Geotechnical Subsurface Charateristics	Ejiri et al. Designation (Fig. 31)	Ejiri et al Designation (Fig. 32)	Approximate Translation To NEHRP 1994	
Tertiry or older rock, Or diluvium with H< 10 m	Rock	Stiff Ground	A or B	A,B Or C
Diluvium with $H \geq$ 10m, Or alluvium with H < 10 m	Stiff Soil			
Alluvium with H< 25 m Including soft layer with Thickness less than 5 m	Normal Soil	Soft Ground		C, D Or E (or F)
Other than the above, Usually soft alluvium Or reclaimed land	Soft Soil		D or E E or F	

Table 6-10 Estimated Soft Ground /Stiff Ground Acceleration Ratios (AR) From Soil Stations in Fig. 6-13 Close to the Fault in 1995 Kobe Earthquake, (modified after Ejiri et al, 1996)

Station #	X_{eq} (km)	a_p (g)	$(a_p)_{stiff\,eq}$ (*) (g)	$AR = \frac{a_p}{(a_p)_{stiff\,eq}}$
1	7.9	0.81	0.575	1.41
2	8.0	0.64	0.574	1.12
3	8.9	0.50	0.564	0.89
4	10.5	0.56	0.547	1.02
5	14.2	0.32	0.519	0.62
6	15.8	0.32	0.510	0.63
7	22.2	0.79	0.480	1.65
	Average =	0.56	Average =	1.05

(*) $(a_p)_{stiff\,eq}$ obtained from $\log a_p = 0.9181 - 0.17594 \log x_{eq}$

Table 6-11 Stiff Ground Soil Stations in Fig. 6-13 close to the fault in 1995 Kobe Earthquake (modified after Ejiri et al., 1996)

Station #	X_{eq} (km)	a_p (g)	$(a_p)_{stiff\ eq.}$(*) (g)	$AR = \frac{a_p}{(a_p)_{stiff\ eq.}}$
8	6.8	0.86	0.591	1.46
9	7.5	0.52	0.580	0.90
10	7.7	0.58	0.577	1.01
11	8.5	0.59	0.568	1.04
12	8.0	0.31	0.574	0.54
13	10.6	0.70	0.546	1.28
14	14.1	0.79	0.520	1.52
15	16.2	0.44	0.507	0.87
16	16.2	0.41	0.507	0.81

Average = 0.58 **Average = 1.05**

(*) $(a_p)_{stiff\ eq}$ Obtained from $\log a_p = 0.9181 - 0.17594 \log X_{eq}$

SECTION 7

LIQUEFACTION TRIGGERING EVALUATION IN SEISMIC CODES

In addition to modification of the ground shaking due to local site conditions previously discussed in this report, another important cause of earthquake damage of constructed facilities is liquefaction of water-saturated sands and other cohesionless soils and associated ground failure and ground displacement. In general, most problems arise after high excess pore water pressures and triggering of liquefaction occurs in the free field, and the state-of-practice including code specifications such as AASHTO 1996 are based on charts such as that of Fig. 7-1, which allow evaluating liquefaction triggering for a given set of local site conditions and ground shaking parameters.

These charts, calibrated by case histories of liquefaction and no liquefaction during earthquakes, were originally proposed by Seed and Idriss (1971), and further developed by Seed et al. (1975, 1985) and Seed and Idriss (1982) using as main soil parameter the corrected Standard Penetration Resistance (SPT), $(N_1)_{60}$, shown in Fig. 7-1. These charts have been recently updated and extended to include also the use of the static Cone Penetration Resistance (CPT) and of the shear wave velocity of the relevant liquefiable soil, Vs, as well as of the Becker Penetration Test (BPT) for gravelly soils (Robertson and Wride, 1998; Andrus and Stokoe, 1998; Harder and Boulanger, 1998; Youd et al., 1998). The use of all these charts requires two main ground shaking parameters: the earthquake magnitude, M, and the horizontal peak ground acceleration, (a_p), that would develop at the surface of a soil deposit similar to the one being evaluated in the absence of high pore water pressures and of liquefaction (Youd et al., 1998).

Guidelines are needed for the formulation of code provisions in future editions of AASHTO and other seismic codes on the subject of liquefaction triggering that incorporates: (i) the way seismic hazard on rock or firm soil is being mapped by USGS throughout the US; (ii) the new site categories and site coefficients already incorporated in NEHRP 1994 and 1997 and UBC 1997, and proposed for incorporation in future versions of AAHSTO; and (ii) the new developments in the state-of-practice of evaluating liquefaction.

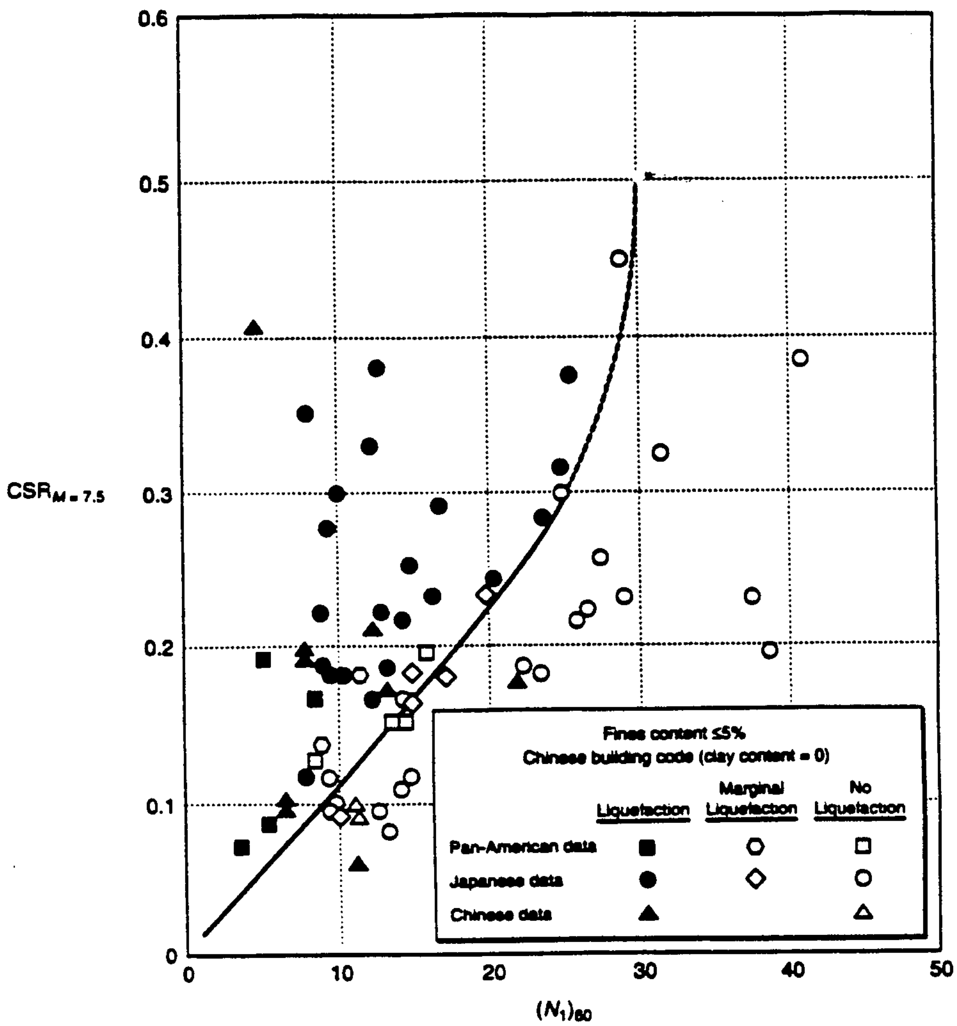
It is suggested that the charts based on SPT, CPT, V_s, and BPT, included in the publication by Youd et al. (1998) be included in future versions of AASHTO, as well as other relevant aspects needed for the evaluation (specifically, the magnitude scaling coefficients to use charts such as that of Fig. 7-1, developed for M = 7.5, in connection with other earthquake magnitudes).

Information on the current mapping of seismic hazard can be found in Frankel et al. (1997) and in the corresponding Web page (http://geohazards.cr.usgs.gov/eq/). All maps and map values are being done for hard sites at the boundary between NEHRP soil profile types B and C in Table 4-1, rather than on soil profile type B used for site coefficients F_a and F_v in Table 4-3. However, as done already in NEHRP (1997), it is suggested that this difference be conservatively ignored and that values of a_p provided in Tables such as Table 7-1, be assigned to soil profile type B. Probabilistic values of peak ground acceleration and of response spectra at various periods are being provided for these hard sites. For example, for San Francisco, $a_p = 0.8983$ g and for New York City, $a_p = 0.2413$ g, with both values corresponding to 2% probability of exceedance in 50 years.

It is proposed to use the values of F_a of Table 4-3 as the basis to convert the corresponding mapped a_p into the value of a_p on the soil surface needed for liquefaction evaluations. That is, as discussed in previous sections, the short-period site coefficient F_a is used at zero period, assuming $AR = F_a$. In this formulation, the values of A_a are identified with a_p on soil profile type B, in g's. In the definition of the site category, all soil layers down to a depth of 30 m should be used, including both liquefiable and nonliquefiable layers and ignoring the possibility of liquefaction. For example, for New York City and a site classified as E in Table 4-3, the value of a_p to be used to enter the liquefaction charts for a 2% probability of liquefaction in 50 years would be approximately, $a_p = (0.2413)(1.5) = 0.36$ g.

Another important aspect in the liquefaction evaluation is the selection of the earthquake magnitude, M, associated with a_p, which may be a problem in probabilistic formulations of the seismic hazard that consider contributions to the hazard of different earthquake sources and magnitudes. To address this problem, USGS provides plots and tables in which the probabilistic hazard in terms of response spectra, peak acceleration and other parameters, is de-aggregated, as illustrated by Table 7-1 and Fig. 7-2. The height of each bar or the numbers in the tables in these magnitude-distance (M-D) plots and tables, represents the percent contribution of that M-D combination to the total hazard. Conservatively, it is suggested to conduct the liquefaction evaluation with the corresponding probabilistic value of a_p already calculated (e.g., $a_p = 0.36$ g for the hypothetical site and probability above in New York City), and with the largest value of M which is significant for a_p at that location. One possibility would be to neglect the earthquake magnitudes that contribute, say, less than about 20% to the hazard. Looking at Table 7-1, this would give M = 8.0 for San Francisco and M between 6.5 and 7.0 for New York City.

Figure 7-1 Liquefaction evaluation chart for clean sands and earthquake magnitude, M = 7.5 (Seed et al., 1975; reproduced by Kramer, 1996)



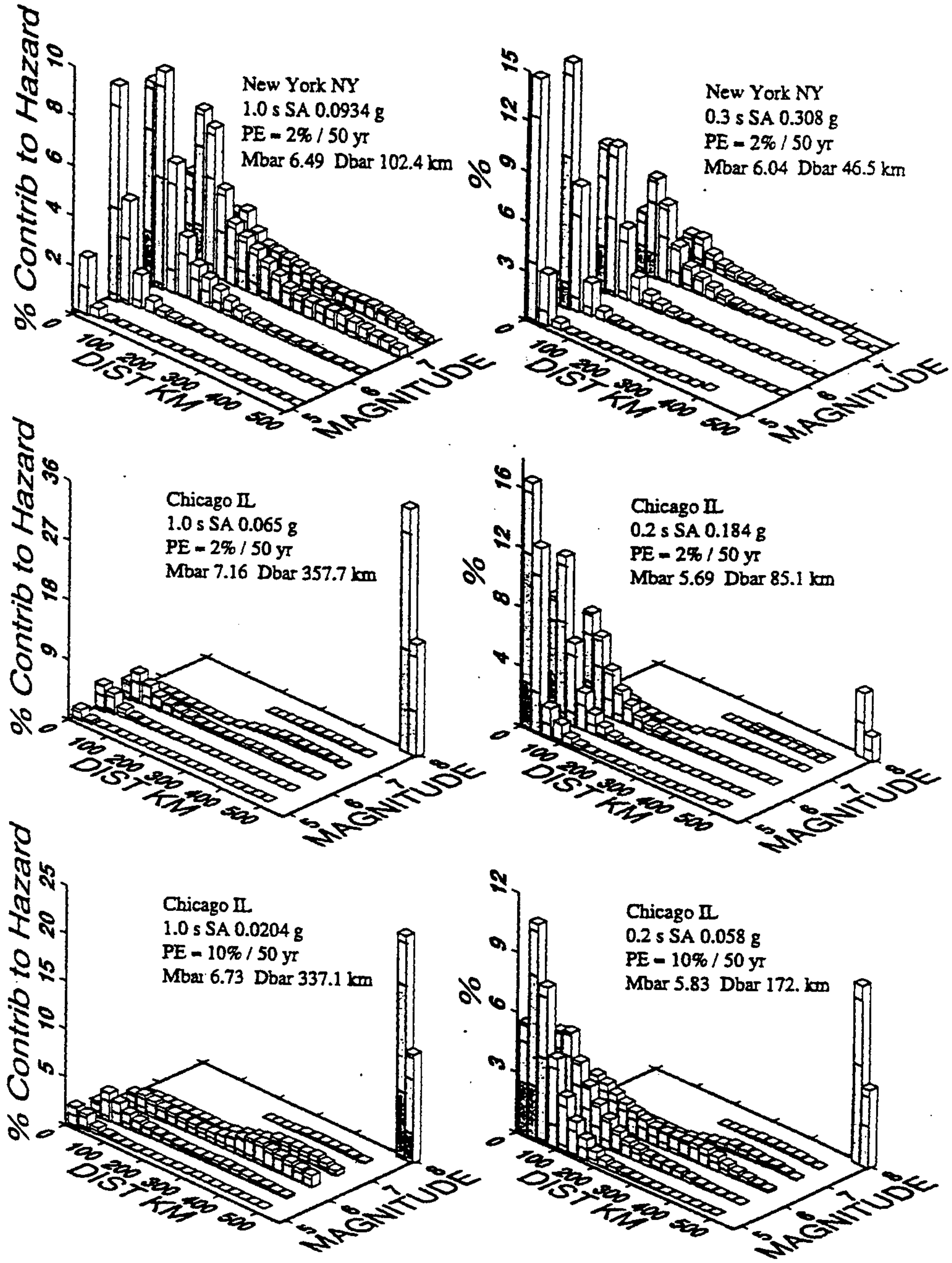


Figure 7-2 De-aggregation plots for various locations, spectral ordinates, and probability levels (Frankel et al., 1997)

Table 7-1 De-aggregation tables of peak ground acceleration for 2% probability of exceedance in 50 years for San Francisco and New York City (from Web page: http:/geohazards.cr.usgs.gov/eg)

Deaggregated Seismic Hazard PE = 2% in 50 years pga
San_Francisco CA 37.803 deg N 122.471 deg W PGA=0.89830 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
d<= 25.	0.000	0.708	0.668	1.897	1.749	17.794	77.139	0.000	0.000
50.	0.000	0.000	0.000	0.001	0.012	0.000	0.000	0.000	0.000
75.	0.000	0.001	0.001	0.004	0.004	0.000	0.000	0.000	0.000
100.	0.000	0.001	0.001	0.005	0.007	0.000	0.000	0.000	0.000
125.	0.000	0.000	0.000	0.003	0.003	0.000	0.000	0.000	0.000
150.	0.000	0.000	0.000	0.001	0.001	0.000	0.000	0.000	0.000

Deaggregated Seismic Hazard PE = 2% in 50 years pga
New_York NY 40.750 deg N 73.980 deg W PGA=0.24130 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	20.633	17.269	10.774	5.754	2.235	1.375
50.	3.698	5.968	6.742	5.841	3.154	2.385
75.	0.266	0.770	1.580	2.311	1.858	1.885
100.	0.022	0.096	0.311	0.667	0.718	0.914
125.	0.003	0.021	0.090	0.258	0.334	0.513
150.	0.001	0.006	0.033	0.121	0.189	0.349
175.	0.000	0.002	0.011	0.052	0.100	0.225
200.	0.000	0.000	0.003	0.021	0.048	0.135
225.	0.000	0.000	0.001	0.008	0.023	0.076
250.	0.000	0.000	0.000	0.003	0.011	0.041
275.	0.000	0.000	0.000	0.001	0.006	0.025
300.	0.000	0.000	0.000	0.001	0.003	0.017
325.	0.000	0.000	0.000	0.000	0.002	0.012
350.	0.000	0.000	0.000	0.000	0.001	0.009
375.	0.000	0.000	0.000	0.000	0.001	0.007
400.	0.000	0.000	0.000	0.000	0.000	0.004
425.	0.000	0.000	0.000	0.000	0.000	0.003
450.	0.000	0.000	0.000	0.000	0.000	0.002
475.	0.000	0.000	0.000	0.000	0.000	0.001
500.	0.000	0.000	0.000	0.000	0.000	0.001

SECTION 8
CONCLUSIONS

8.1 Some Areas of Further Research

While the new site coefficients and site categories included in the recent seismic building codes and proposed for AASHTO for bridges constitute a great advance compared with the old practice, they are not the last word. Both the definitions of site categories and the values of F_a and F_v necessarily include simplifications as well as analytical extrapolations from the strong-motion records available in 1992. Also, the same as in AASHTO and in the old building codes, 2D/3D effects are not considered. For these and other reasons, there are a number of areas where further research is especially important which may affect future generations of seismic codes. A possible list of these areas of needed further research is included in Table 8-1.

8.2 Recommendations and Conclusions

(1) The new provisions on site effects for buildings incorporated into the 1994 NEHRP and 1997 UBC reflect a broad consensus of the geotechnical engineering and earth science communities and constitute a significant advance over the provisions contained in older code versions.

(2) The site categories are now based unambiguously on the average shear wave velocity ($\overline{V}_s$) of the top 100 ft of the profile at the site.

(3) The main changes in the site coefficients include: replacement of the old coefficient S by F_v at long periods, introduction of a new coefficient F_a at short periods, and dependence of F_a and F_v on both site category and level of rock shaking to consider soil nonlinearity.

(4) The low seismicity areas of the US are affected more by these changes than high seismicity areas such as California.

(5) An analysis of site coefficients F_a and F_v from records of the 1994 Northridge earthquake performed in this report (Table 6-8), generally verified these coefficients for NEHRP site profile types C and D, in terms of the NEHRP values and ranges being about equal or larger than the recorded ranges. The exception is F_a of sites C, where larger amplifications were recorded than considered by NEHRP; further research is recommended on this issue

(6) A preliminary analysis of peak ground accelerations recorded on rock and soil stations at very close distances to the fault in the 1995 Kobe earthquake failed to show the expected deamplification of acceleration on soft ground at acceleration levels typically in excess of 0.4 g on rock and stiff sites. A more refined study is suggested on the basis of additional information about the subsurface conditions at the recording stations as well as of detailed consideration of other significant factors influencing the soil and rock records.

It is recommended that AASHTO be updated incorporating into the provisions for seismic design of bridges the advances in site effects already specified in NEHRP and UBC. Also, recommendations are provided on the use of the short-period site coefficient F_a and of the seismic hazard parameters being mapped nationally by USGS, for soil liquefaction evaluations in AASHTO and other codes.

Table 8-1 Some Areas of Further Research

Areas of Research
Influence of soil and rock properties under 100ft.
Amplification of long period motions by deep stiff sites on hard rock
Amplification of nearby earthquakes by shallow stiff sites on hard rock
Soft sites subjected to very strong ground motions
2D/3D and basin effects
Site effects on near fault ground motion
Site effects and spatial variation
Can F_a be used always for amplification of peak acceleration?
Site effects on very long period motions (T > 2 seconds)

SECTION 9

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