EVALUATION OF STRUCTURAL RESPONSE FACTORS USING GROUND MOTIONS RECORDED DURING THE LOMA PRIETA EARTHQUAKE

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ABSTRACT

Present procedures of defining seismic design forces are based on the use of elastic spectra reduced by a response modification factor which only depends on the type of structural system. This paper summarizes the results of an analytical study concerning strength reduction factors. Strength demand and strength reduction spectra were computed using simplified elastic and inelastic bilinear SDOF structural models and 36 ground motions recorded during the 1989 Loma Prieta earthquake. Special emphasis was given to the influence of local site conditions. Results show that strength reduction factors are significantly affected by the level of inelastic deformation, the period of vibration and by local site conditions.

INTRODUCTION

While current building codes accept structural damage in the event of a major earthquake, the estimated loss of 10 billion dollars produced by the 1989 Loma Prieta earthquake was not expected and was beyond what could be considered as an acceptable damage during moderate earthquake ground motions.

The procedure now commonly used is to design for lateral forces that are obtained from reducing a Smoothed Linear Elastic Design Response Spectra, SLEDRS by a force reduction factor referred as either a response modification factor, R, in the National Earthquake Hazard Reduction Program (NEHRP) [1] and the Applied Technology Council (ATC) [2] design recommendations, or as a system performance factor, R_w, in the 1988 Uniform Building Code [3]. There is no rational procedure from which the recommended factors have been derived and, as presently specified, they are empirical and only dependent on the type of structural system, thus, assuming that force reductions are the same regardless of the period of vibration or the local site conditions. By using this procedure combined with present methods of allowable stress design used in steel structures and of the method referred as strength method used in reinforced concrete structures codes may lead to conservative or unconservative design for the maximum credible event, depending on the type of structure (i.e. the response modification factor that is used).

While the rational way to design should be based on the use of Smoothed Inelastic Design Response Spectra, SIDRS, derived from statistical studies of inelastic response spectra of all possible critical earthquake ground motions that might occur at the structure site, at present, most of the practicing structural engineers are not familiar with the derivation and use of such SIDRS. Most designers prefer to obtain the seismic design forces through the use of SLEDRS and the use of response modification factors. Therefore based on the information that has been gathered from recent earthquakes, it is of primary importance to be able to specify more reliable SLEDRS and to derive more reliable values for the response modification factors.

Previous studies have investigated strength reductions due nonlinear behavior, however, the effect of local soil conditions has not been taken into account or the study has been limited to only one type of soil conditions [4,5,6]. The main objective of this investigation is to improve the understanding of strength reduction factors through the use of ground motions recorded during the 1989 Loma Prieta earthquake and simplified elastic and inelastic bilinear single-degree-of-freedom, SDOF, structural models.

STATISTICAL STUDY ON THE RESPONSE OF SDOF SYSTEMS

Selected Ground Motions: A total of 36 ground motions recorded during the October 17, 1989 Loma Prieta earthquake. The selected ground motions are listed in Table 1. They were obtained at recording stations with epicentral distances between 7 and 100 km and site conditions and geologic conditions that range from rock to soft clayey soils (bay mud). Additionally, strong ground motions recorded in previous earthquakes were used to compare the results to those obtained using Loma Prieta data. The following records were used for this purpose: El Centro (1940 Imperial Valley), Taft (1952 Kern County), Sendai (1978 Miyagi-Ken-Oki), James Road (1979 Imperial Valley), Llolleo (1985 Chile), SCT (1985 Mexico), San Salvador (1986 El Salvador), and Colonia Roma (1989 Acapulco, Mexico). The ground motions were classified into three groups according to the geologic conditions at the recording station. These groups were rock, alluvium and very soft soil.

Results from the Statistical Study: Constant ductility nonlinear spectra were computed for all records in each soil group. Strength demands for each record were then normalized using peak ground acceleration (PGA). Details on the procedure used to compute the constant ductility nonlinear spectra can be found in Ref.7. For ground motions recorded on rock or alluvium sites, nonlinear spectra were computed for a fixed set of 50 periods between 0.05 and 3.0 seconds. In the case of ground motions recorded on very soft soil, spectra were computed for a fixed set of 50 ratios of T/T_g, where T_g is the predominant period of the ground. The reason for using T/T_g instead of T is that T_g can have large variations depending on the shear wave velocity of the soil and the depth of the soft deposits. For statistical analyses of spectra it makes no sense to average spectral ordinates at a certain period for ground motions with significantly different predominant periods. For structural design purposes, it is important to characterize the seismic demands on structures with periods shorter, longer or near the predominant period.

In this study the predominant period was computed as the period corresponding to the maximum spectral velocity ordinate. It can be shown that essentially the same predominant period would be obtained if the Fourier amplitude spectrum or the input energy spectrum are used instead of the velocity spectrum because of the relationship between these three spectra.

Computation of constant ductility response spectra involves iteration on the yielding strength of the system. The iteration is successful when the computed ductility reaches the specified (target) ductility within a certain tolerance that can be specified by the user. In this study, ductilities were considered satisfactory if they were within 1% of the target ductility. The displacement ductility ratio is defined as the ratio of the maximum absolute value of the displacement response divided by the yield displacement of the system,

$$\mu = \frac{u_{max.}}{u_y} \quad (1)$$

The following values of ductility were selected for this study: 1 (elastic), 2, 3, 4, 5 and 6. Details on the procedure used to compute the constant ductility nonlinear spectra can be found in Ref.7. Due to the large computational effort involved in calculating constant ductility nonlinear spectra, the study was limited to bilinear systems with a post-elastic stiffness of 3% of the elastic stiffness and with a damping ratio of 5% of critical.

Inelastic strength demand spectra normalized with respect to the maximum ground acceleration ($\eta = C_y/\ddot{x}_{g_{max}} / g$) for the six ground motions representative of all three site conditions are presented in Figure 1. As shown in this figure, spectral shapes for inelastic strength demands differ significantly from elastic (μ=1) spectral shapes. For soft soil records (SCT and Emeryville) and periods smaller than the predominant period of the site there is a small difference between the strength demands for ductilities between 2 and 6. This implies that small changes in the strength of yielding structures in this period range may produce large changes in ductility demands.

The strength reduction factor in a SDOF system undergoing a certain displacement ductility μ_i, is defined as

$$R_\mu = \frac{C_y(\mu=1)}{C_y(\mu=\mu_i)} \tag{2}$$

where $C_y(\mu=1)$ is the strength demand on a linear elastic system (i.e. the strength required to maintain the system elastic) and $C_y(\mu=\mu_i)$ is the strength demand on a nonlinear system undergoing a displacement ductility μ_i.

Computed strength reduction factors of six ground motions are shown in Figure 2. For each ground motion, reduction factors are plotted for displacements ductilities of 6, 5, 4, 3, and 2 (from top to bottom). Examples of predominant period of 4 soft soil sites in the San Francisco Bay Area and 2 sites in Mexico City are shown in Figure 3. It can be seen that, even within soft soil sites, large variations occur in the predominant site period depending on the shear wave velocity of the different soil layers and total depth of the soft soil deposits.

Mean inelastic strength demands of 14 ground motions recorded at rock sites during the Loma Prieta earthquake are shown in Figure 4. It can be seen that the shape of the elastic strength demands (μ=1) differs significantly from the shape of inelastic strength demands, suggesting that the currently used procedure which accounts for inelastic behavior by specifying a reduced spectra with the the same shape as the elastic is inadequate. Mean inelastic strength demands of 14 ground motions recorded at soft soil (bay mud) sites in the San Francisco Bay Area during the Loma Prieta earthquake are shown in Figure 5. For structures responding elastically and with fundamental periods that are close to the predominant period of the site experience large amplification of seismic forces. These large amplifications, however, are significantly reduced when inelastic behavior occurs. For ductilities larger than 3, inelastic strength demands decrease monotonically with increasing periods.

By comparing the average spectra of ground motions recorded on rock and soft soil it can be seen that the largest dynamic amplification for elastic response (μ=1) is produced for soft soil sites. These results are different to those reported previously by Seed et al. [8] who computed larger amplifications for rock and alluvium sites than for soft soil sites. Moreover, the maximum amplification (with respect to PGA) for soft soil sites computed in that study is nearly 30% smaller than the maximum amplification computed here. For rock and alluvium sites the maximum amplifications computed in this study are practically the same as those found by Seed et al. with a smaller set of ground motions.

Mean strength reduction factors of 14 ground motions recorded at rock sites during the Loma Prieta earthquake are shown in Figure 6. It can be seen that strength reductions are by no means constant as implied by current seismic recommendations. Reduction factors which are based on assuming that the maximum displacement is the same in elastic and inelastic systems (i.e. $R_\mu = \mu$) is unconservative in the short period range. Reduction factors which are based from assuming equal energy in elastic and inelastic systems ($R_\mu = [2\mu - 1]^{1/2}$) are also unconservative in this period range.

Mean strength reduction factors of 14 ground motions recorded at bay mud sites during the Loma Prieta earthquake are shown in Figure 7. Strength reduction factors in this case are characterized by small values for periods smaller than the predominant site period ($T/T_g < 1$) and by very large reductions for periods close to the predominant site period. Strength reduction factors are approximately equal to μ for T/T_g ratios greater than 2.5.

CONCLUSIONS

Elastic and inelastic response spectra were computed for 36 ground motions recorded during the Loma Prieta earthquake and 8 ground motions recorded in previous earthquakes. The results indicate that the shape of the inelastic strength demands differs from the shape of elastic strength demands. It is concluded that strength reductions produced in nonlinear systems are strongly affected by the natural period of vibration, the level of inelastic deformation, and the local site conditions. For soft soil sites, the estimation of the predominant period of the site is particularly important on the estimation of inelastic strength demands. The use of period-independent strength reduction factors, as currently specified in many seismic design recommendations, may lead to unconservative designs.

ACKNOWLEDGEMENTS

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STATION NAME	GEOLOGY	EARTHQUAKE DATE	MAGN.	EPICTR. DIST.[km]	DIRECTION	PGA [g's]
CORRALITOS Eureka Canyon Road	Landslide deposits	Loma Prieta October 17, 1989	7.1(M_S)	7	90 360	0.47 0.62
SANTA CRUZ UCSC	Limestone	Loma Prieta October 17, 1989	7.1(M_S)	16	90 360	0.41 0.43
SAN FRANCISCO Cliff House	Franciscan sandstone	Loma Prieta October 17, 1989	7.1(M_S)	99	90 0	0.11 0.07
SAN FRANCISCO Pacific Heights	Franciscan sandstone	Loma Prieta October 17, 1989	7.1(M_S)	97	360 270	0.05 0.06
SAN FRANCISCO Presidio	Serpentine	Loma Prieta October 17, 1989	7.1(M_S)	98	90 0	0.20 0.10
SAN FRANCISCO Rincon Hill	Franciscan sandstone	Loma Prieta October 17, 1989	7.1(M_S)	95	90 360	0.09 0.08
YERBA BUENA ISLAND	Franciscan sandstone	Loma Prieta October 17, 1989	7.1(M_S)	95	90 360	0.06 0.03
CAPITOLA Fire Station	Alluvium	Loma Prieta October 17, 1989	7.1(M_S)	9	90 360	0.39 0.46
HOLLISTER South & Pine	Alluvium	Loma Prieta October 17, 1989	7.1(M_S)	48	90 360	0.17 0.36
OAKLAND 2-Story Office Bldg.	Alluvium	Loma Prieta October 17, 1989	7.1(M_S)	92	290 200	0.24 0.19
STANFORD Parking Garage	Alluvium	Loma Prieta October 17, 1989	7.1(M_S)	51	360 90	0.26 0.22
EMERYVILLE Free Field South	Bay mud	Loma Prieta October 17, 1989	7.1(M_S)	97	350 260	0.21 0.26
EMERYVILLE Free Field North	Bay mud	Loma Prieta October 17, 1989	7.1(M_S)	97	350 260	0.20 0.22
OAKLAND Outer Harbor Wharf	Bay mud	Loma Prieta October 17, 1989	7.1(M_S)	95	305 125	0.27 0.29
TREASURE ISLAND Naval Base	Fill	Loma Prieta October 17, 1989	7.1(M_S)	98	90 360	0.16 0.10
SAN FRANCISCO International Airport	Bay mud	Loma Prieta October 17, 1989	7.1(M_S)	79	90 360	0.33 0.23
SAN FRANCISCO 18-Story Comercial Bldg.	Fill over bay mud	Loma Prieta October 17, 1989	7.1(M_S)	95	980 350	0.13 0.16
FOSTER CITY Redwood Shores	Bay mud	Loma Prieta October 17, 1989	7.1(M_S)	63	90 0	0.28 0.26

Table 1. Loma Prieta earthquake ground motions selected for this study

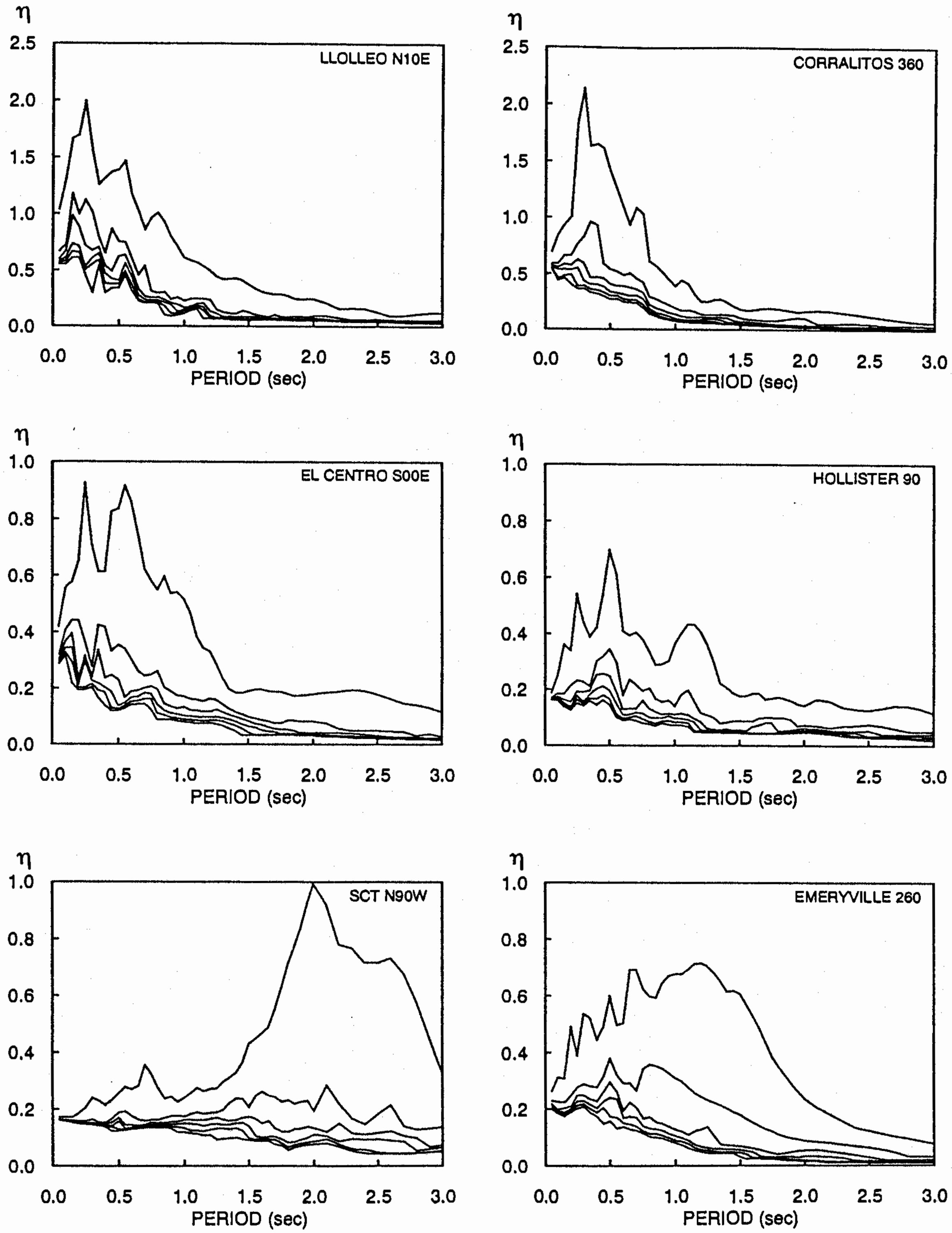

FIGURE 1. Normalized inelastic strength demands (μ=1,2,3,4,5,6).

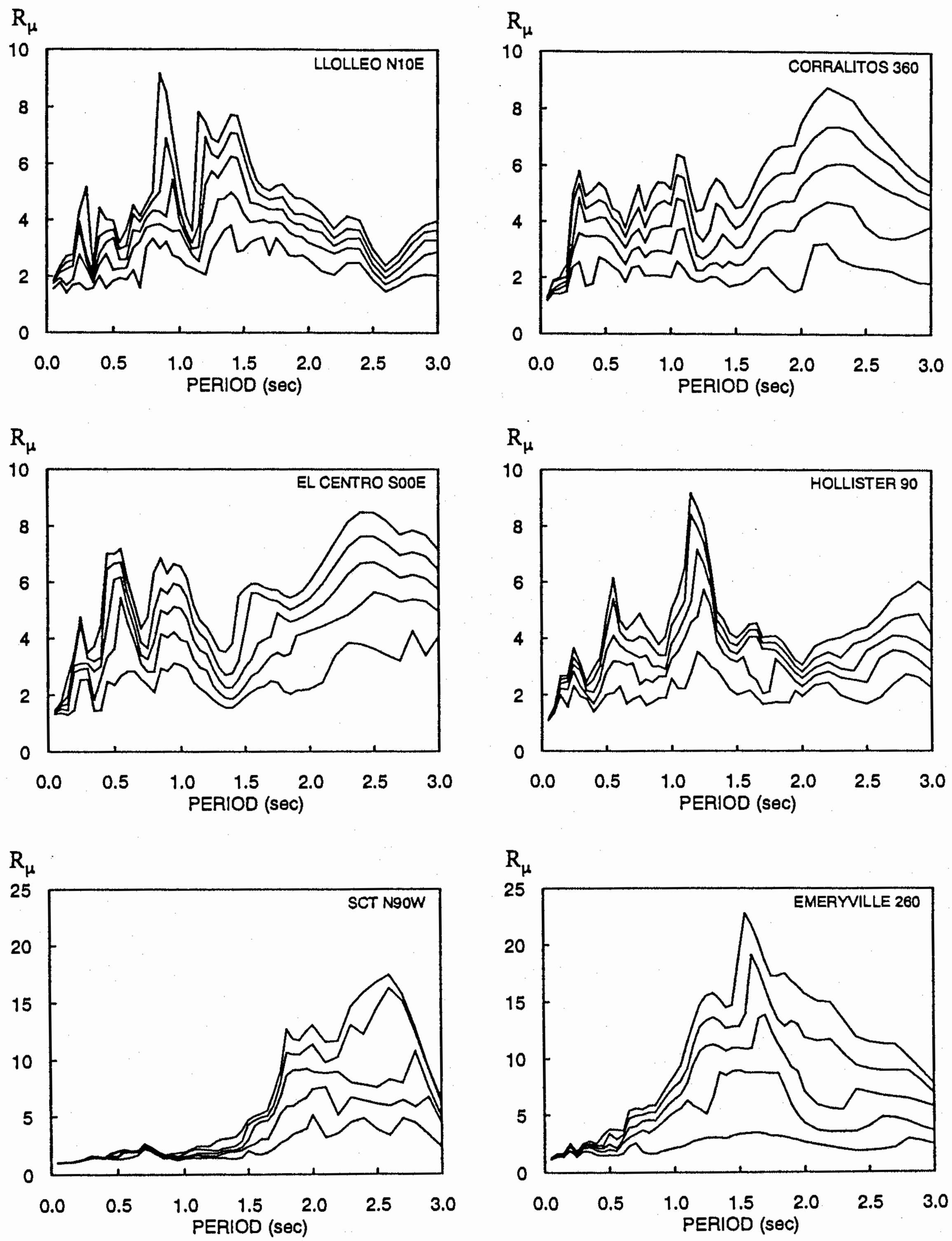

FIGURE 2. Strength reduction factors (μ=6,5,4,3,2).

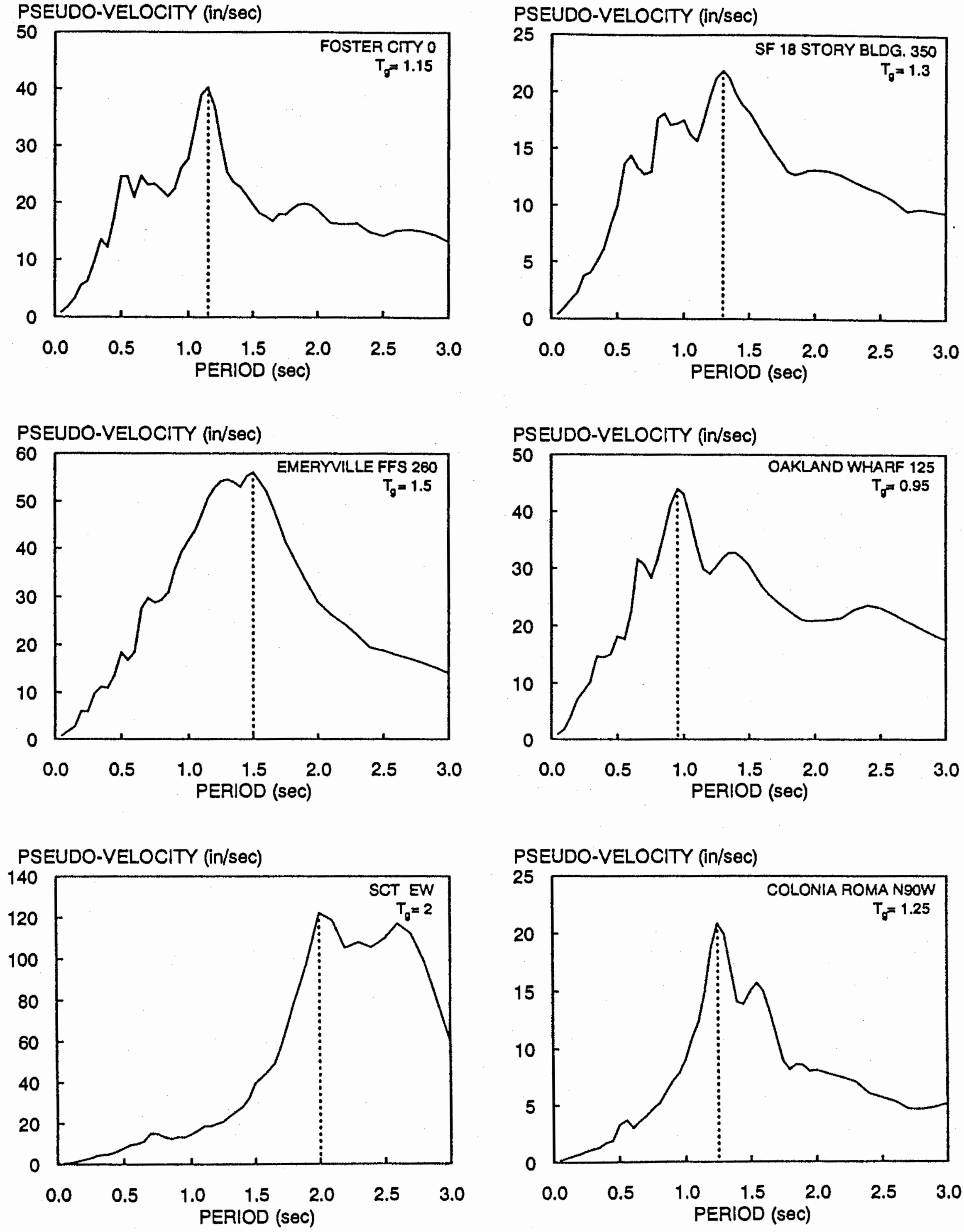

FIGURE 3. **Predominant periods for various soft soil sites in the San Francisco Bay Area and Mexico City.**

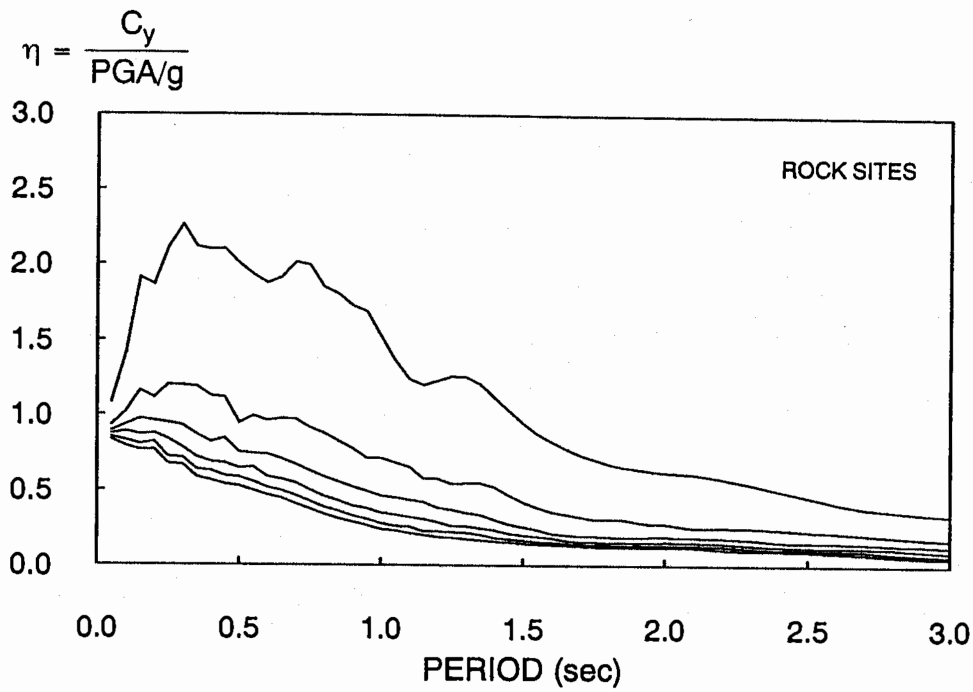

Figure 4. Mean strength demands of ground motions recorded on rock when normalized using PGA (μ=1,2,3,4,5,6).

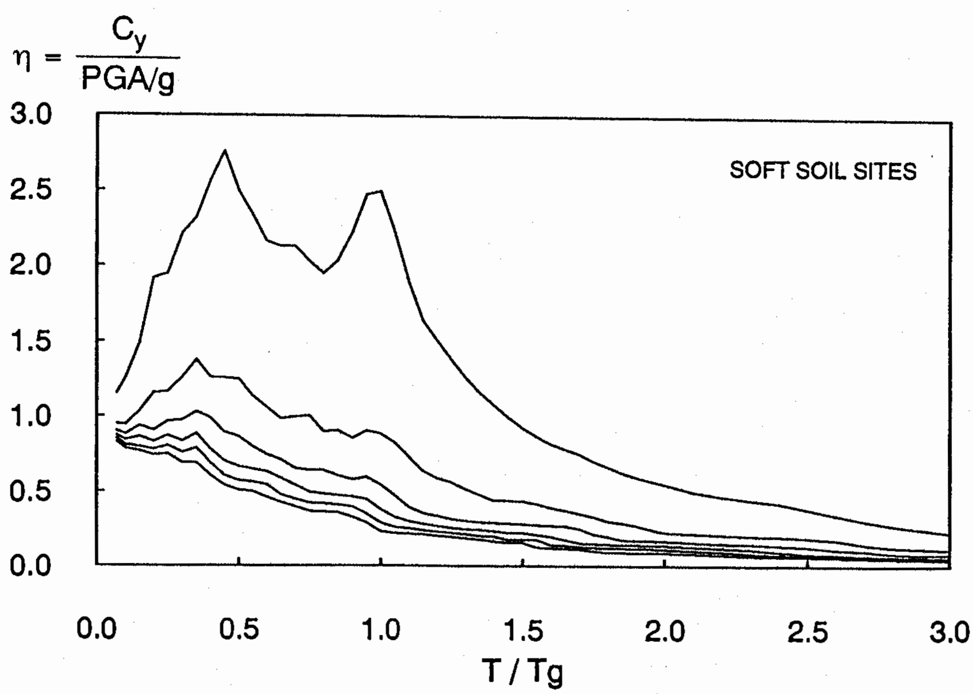

Figure 5. Mean strength demands of ground motions recorded on soft soil when normalized using PGA (μ=1,2,3,4,5,6).

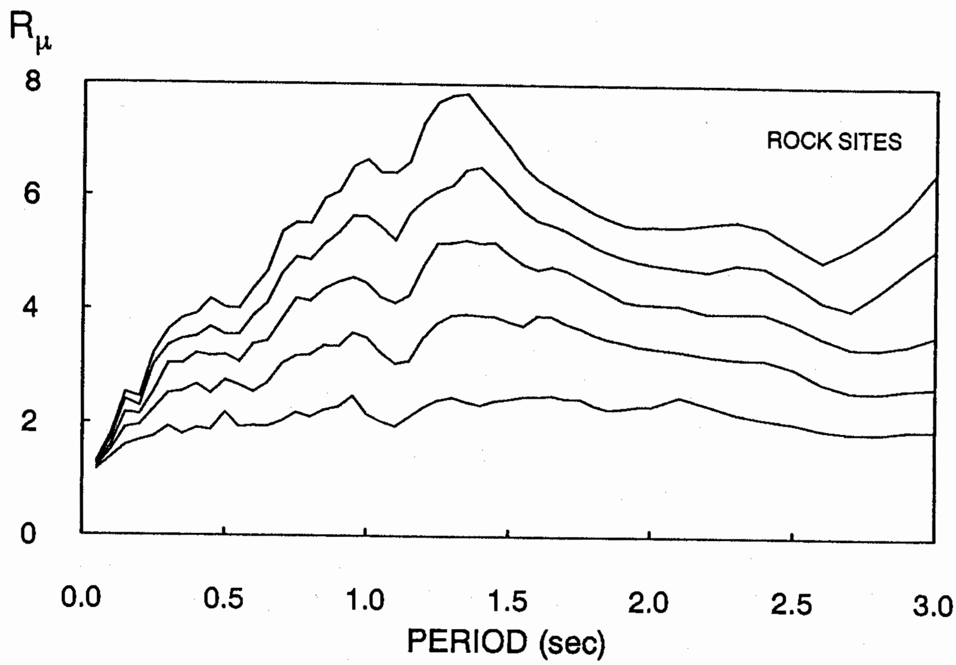

Figure 6. Mean strength reduction factors of ground motions recorded on rock (μ=6,5,4,3,2).

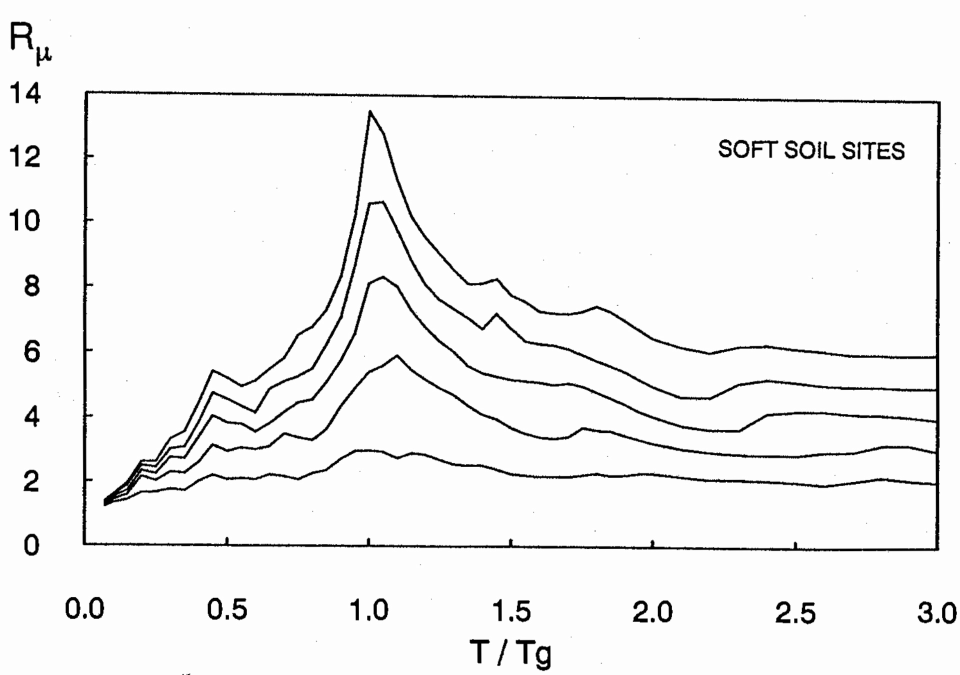

Figure 7. Mean strength reduction factors of ground motions recorded on soft soil (μ=6,5,4,3,2).