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USE AND KNOWLEDGE OF DESIGN SPECTRA

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ABSTRACT

The factors that influence the response of inelastic systems and the methodology for the derivation of design spectra from actual earthquake records is reviewed and current practice is examined with regard to whether seismic codes make use of the available knowledge. Particular attention is given to the response modification factor R used in codes to reduce the elastic spectrum ordinates to account for inelastic behavior; the difficulty of using separate constant factors to represent the various effects included in R is emphasized, and a direct integrated approach is proposed. Detailed discussion of the uncertainties in the estimation of maximum deformation demands is also presented, and recommendations for future improvement are made.

KEYWORDS

Design spectrum, inelastic response, earthquake response, soil effect, ground motion

STATE OF THE ART ON THE DERIVATION OF DESIGN SPECTRA

The most important factors and methodological aspects that influence the derivation of design spectra from actual earthquake records are summarized next, together with comments regarding whether seismic codes have assimilated such a knowledge:

a) Ensemble of records. The ideal ensemble for a given location should comprise severe ground motions obtained during several large magnitude earthquakes which occurred at short or moderate distances, so that intensities are relevant for seismic design, and a variety of conditions regarding earthquake source and travel path effects are present, besides the influence of the local geology itself. Unfortunately, such a complete data set is rarely available; thus, it is necessary to group motions recorded in different locations with similar soil and tectonic conditions. It is worth to point out however that the findings of studies considering records in quite different seismic zones feature remarkable similarity (Riddell, 1995).

b) Normalization. Since the intensity of the selected ground motion records normally differ from each other, the computed responses cannot be compared on an absolute basis. In order to make meaningful comparisons, it is customary to scale the spectra to some predetermined parameter. Normalization can be made either to maximum ground acceleration, maximum ground velocity, or maximum ground displacement, over the entire range of frequencies; but it is preferable to consider normalization relative to maximum acceleration for high frequencies, to maximum velocity for intermediate frequencies, and to maximum displacement for low frequencies. Normalization to three parameters is implicit in the Newmark-Hall formulation, which applies amplification factors to estimates of the three design ground-motion parameters to obtain the design spectrum. On the other hand, the procedure leads to a minimization of the dispersion of the data (Newmark et al, 1972; Riddell and Newmark, 1979).

c) Design ground motion. The relations between ground motion peaks vary with magnitude, distance to source of energy release, and site subsurface conditions. Therefore a better characterization of the design ground motion is by means of the three ground motion parameters, instead of one used in most current codes.

d) Effect of structural damping. The reduction of elastic response due to damping has been studied since long ago (Arias and Husid, 1962). The important fact worth to point out here is that the effect of damping depends on the period of vibration of the system, being more pronounced for intermediate frequency systems (0.4 - 2 hertz) and reducing to zero for infinitely flexible or rigid systems, even for damping factors as high as 100% of critical (Riddell and Newmark, 1976). In turn, in the case of damped inelastic systems, the effect of damping becomes less important as ductility increases (Riddell, 1980). Most building codes have design spectra implicitly based on 5% of critical damping, but do not consider damping as a variable parameter; indeed, it is erroneously recommended to pass from one value of damping to another by multiplying the spectral ordinates by a constant factor (NEHRP, 1985). Furthermore, for the reason given above, the ratio between elastic and inelastic spectra is not the same for all damping factors, i.e, the response modification factor R does depend on damping, decreasing as damping increases (Riddell and Newmark, 1979). Codes should discriminate between different energy dissipation capacities by damping according to structural characteristics, material, and use (for example available damping in a R/C frame building is likely to be much larger than in a R/C bare frame parking lot structure, and similarly in steel construction between an office and an industrial building).

e) Effect of type of load-resistance relationship. The responses of individual systems with different types of non-linearity and all other parameters the same, may either be equal or substantially different, with larger differences occurring for intermediate frequency systems and larger ductilities. No conclusive statements can be made from the observation or particular systems, however, comparisons of average spectra for a number of records and for various nonlinear models clearly reveal some general trends: the mean spectrum ordinates may vary up to 30-50% for frequencies between 0.2-5 hertz and large ductilities ($u=10$), while differences are practically negligible at the low and high frequency ends of the spectrum and for low ductilities; use of the elasto-plastic idealization provides, in almost every case, a conservative estimate of the average response (Riddell, 1980).

f) Effect of soil conditions. It is particularly relevant for softer soils, for intermediate period systems, and for low displacement ductility response, and results in wider regions of maximum response amplification as the soil softens; accordingly, a constant soil factor as used in seismic codes is not appropriate. The maximum response amplification with respect to ground, in terms of spectral acceleration, is not very sensitive to soil conditions. Since soil conditions affect elastic and inelastic responses in a different proportion, the response reduction factor R depends on soil conditions, and significantly so for softer soil (Riddell, 1994, 1995; Miranda 1993; Rahnama and Krawinkler, 1994)

g) Response reduction factor. The already mentioned R factor, defined as the ratio of elastic to inelastic spectral ordinates, is a function of period, damping and soil conditions. None of them is explicitly considered in current seismic codes. In the selection of R-factor values for the various framing systems defined in seismic codes, it is supposed that the amount of damping present in the system has been considered (NEHRP, 1985); however it is hard to see what portion of R is due to structural damping. In any case, a constant R can not represent the effects of damping and inelastic behavior either individually or jointly.

SPECIFICATION OF DESIGN SPECTRA

The first step for the specification of design spectra is the definition of the intensity of the design ground motion. On the basis of an earthquake hazard analysis, selection of the acceptable risk (probability of exceedance during the useful life of the structure), and consideration of subsurface geology conditions, the design effective peak-ground-acceleration A, velocity V, and displacement D, can be defined. As mentioned above most codes only specify A. Next, two kinds of approach can be followed: *indirect* or *direct*. The *indirect method* firstly requires the specification of the elastic design spectrum, which is subsequently deamplified to obtain the inelastic design spectrum. The Newmark-Hall method is typically an *indirect* method, as well as standard code spectra are.

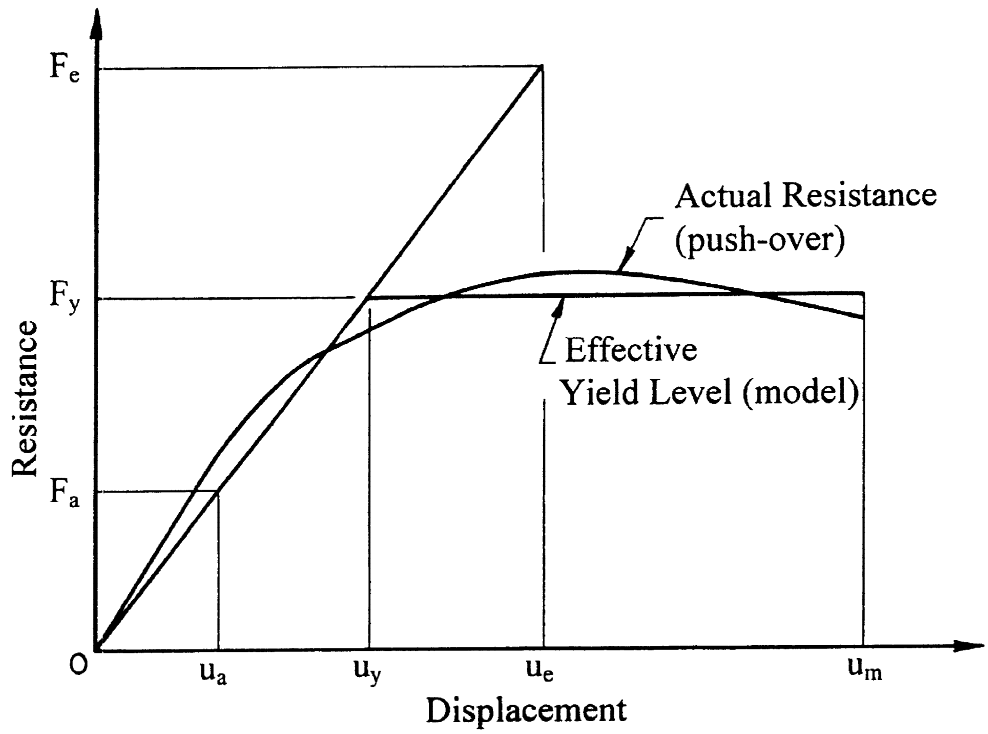

Fig. 1 Notation for Strength and Displacement Levels

The difficulty in the practical application of the indirect method is the complexity of the functional relationship between elastic and inelastic spectral ordinates. Indeed, defining the response modification factor R as the ratio of the elastic response of a given system (maximum elastic force F_e or maximum elastic displacement u_e) to its inelastic spectral ordinate (yield strength F_y or yield displacement u_y associated to a ductility of μ as shown in Figure 1):

$$R = F_e/F_y = u_e/u_y \tag{1}$$

it is clear that R is a function of period (T), ductility (μ), damping factor (ξ), soil conditions (S), load-deformation relationship (F-u), and overstrength factor (R_s):

$$R = R\,(T,\ \mu,\ \xi,\ S,\ F\text{-}u,\ R_s) \tag{2}$$

and, indeed, according to the discussion in the previous section, the corresponding variables are not independent. In fact, the soil effect $S=S(T,\mu)$, $\xi=\xi(T,\mu)$, $F\text{-}u=F\text{-}u(T,\mu)$, and R_s has been found to depend on structural system, design ductility class, and period (Fishinger et al; 1994).

Hence, none of these effects can be represented by a constant factor, nor can R be. Then, a better alternative to specify inelastic design spectra is a *direct approach* using functions that directly fit to the average spectra. The procedure requires to define a function in terms of a number of coefficients that are determined by regression analysis, i.e., by minimizing the error of the observed average response with respect to the value predicted by the given function. For this purpose the following function has been proposed (Riddell, 1995):

$$S_a = \frac{a_1 + a_2 T^{a_3}}{1 + a_4 T^{a_5}} \tag{3}$$

Figure 2 shows the corresponding curves for the case $\xi=5\%$, F-u elastoplastic, and $R_s=1$; the tree soil types are rock, firm ground, and medium stiffness soil. The five a_i coefficients, function of the ductility factor and the type of soil, are given by Riddell, 1995.

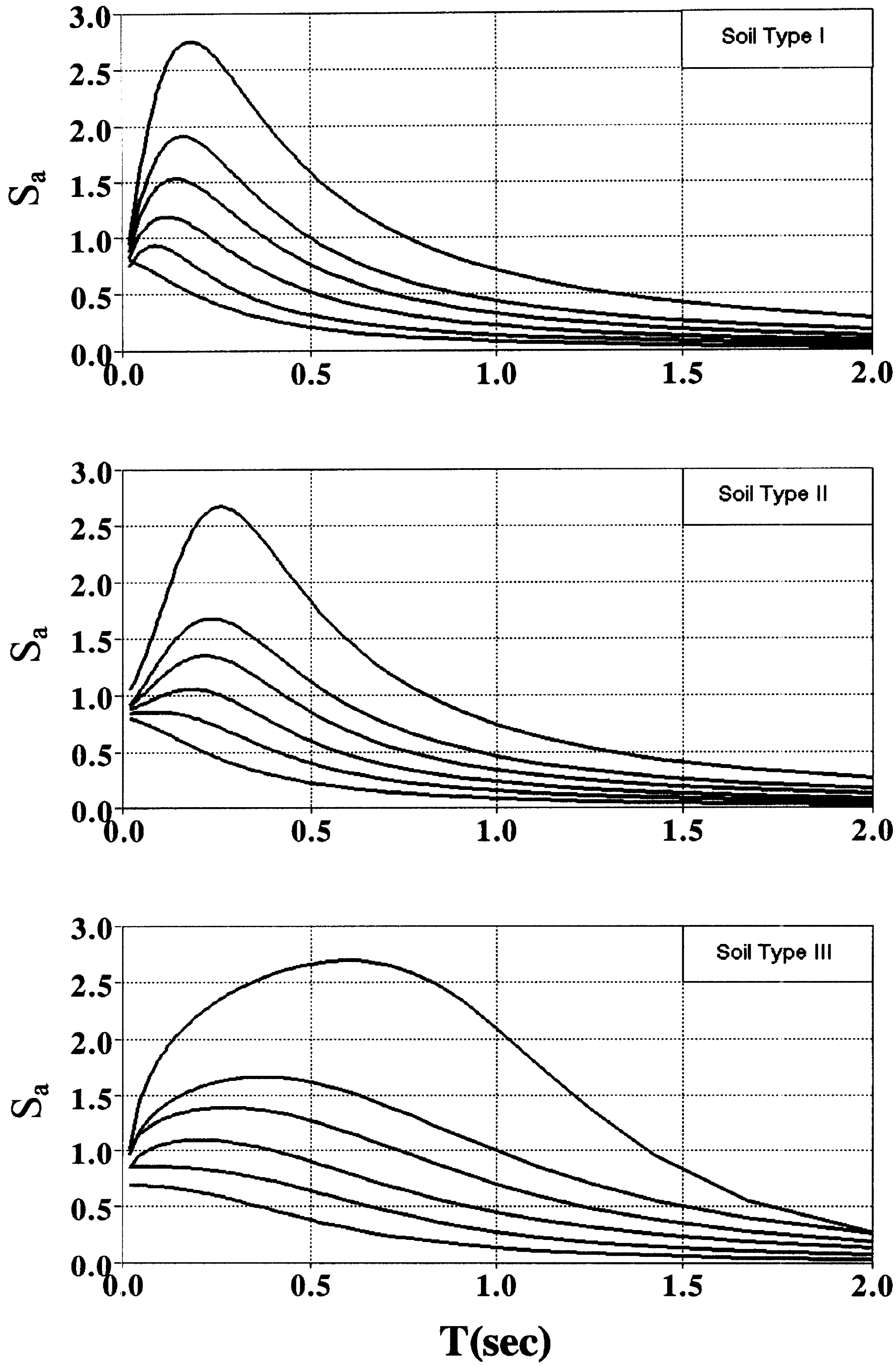

Fig. 2 Inelastic Design Spectra by Direct Approach. Ductilities of 1, 1.5, 2, 3, 5, and 10.

ESTIMATION OF MAXIMUM DEFORMATION DEMANDS

The basic knowledge for single degree of freedom systems is simple and well known, however, seismic codes have not assimilated such a knowledge, and simplifications, which may fail to be adequate, are often used to estimate maximum deformations. A common assumption made is that the maximum elastic (u_e) and inelastic (u_{max}) responses are equal. This is exact for an infinitely flexible system and approximate for long period systems in general. UBC 1988 uses $u_{max} = u_a\ 3R_w/8$ which can be easily shown to mean that $u_{max}=3u_e/8$ which is in general incorrect (u_a is the analysis or calculated displacement for code loads).

Before examining actual spectra it is convenient to derive an approximate relationship for rigid systems, which will show that inelastic displacements may considerably exceed the UBC expression. According to codes, the analysis strength $F_a=F_e/R_w$, where R_w is the code reduction factor, then, the yield strength of the system is $F_y=F_eR_s/R_w$, as shown in Fig. 1. (Note that besides the perhaps unintended factors that affect R_s as mentioned in the previous section, the overstrength factor as defined herein also includes the load factors that apply to the design earthquake load, when combined with gravity loads for example, and the material-code factors, like strength reduction factors in R/C design. It must be emphasized that the use of such a factors should be revised; a basic concept in seismic-design is that the ultimate strength of the structure is settled, capacity that is certain to be reached under moderate or severe earthquake excitation). For rigid systems, say $0.1<T< 0.4$, and for 5% damping, the equal energy rule holds: $F_e/F_y=R_w/R_s=\sqrt{2\mu\text{-}1}$; wherefrom the response ductility can be estimated as $\mu=(R_w^2/R_s^2 + 1)/2$. Since $u_{max}=u_y\mu$ and $u_y=u_aR_s$, the following expression is obtained:

$$u_{max} = \frac{u_a}{2}\left(\frac{R_w^2}{R_s}+R_s\right) \tag{4}$$

wherefrom it is apparent that $(R_w^2/R_s+R_s)/2>>3R_w/8$.

Figure 3 shows displacement spectra for the three soil types mentioned earlier and for ductility factors of 1, 3, and 10. These spectra are indirectly obtained from actual average acceleration spectra (S_a) normalized to A=1g ($u_{max}=\mu S_aT^2/\pi^2$). For different values of A, say A=0.3g, the spectral ordinates shall simply be factored by 0.3. It is apparent from Fig. 3 that for low period systems and large ductilities ($\mu=10$ and also $\mu=5$ not shown in the picture) the maximum displacement exceeds the elastic response displacement; for rock and firm soil this occurs up to T equal 1.6 and 2 respectively. It appears from these figures that for T=0 u_{max} is the same regardless of ductility, but it is not the case; the curves for soil type II plotted in log-log scale in Fig. 4 illustrate that actually the difference between elastic and inelastic displacements increases as the period tends to zero. Figure 4 also shows the well known fact that as $T\rightarrow\infty$ elastic and inelastic displacements become equal, however, it can also be seen that even for very long period systems the inelastic displacements may exceed the elastic response.

Since the response of flexible systems is controlled by the ground displacement motion rather than the ground acceleration history, a more reliable estimation of the maximum deformation should result from average spectra normalized to peak ground displacement. Indeed, Fig. 5 shows the coefficient of variation (mean/standard derivation) of average spectra normalized to ground acceleration and ground displacement (Fig. 5 is specifically for soil type II and $\mu=10$, however the observations that can be made from it are general). It can be seen in Fig. 5 that for T=2 a quite unreliable estimate of maximum deformation is inferred from the average spectrum normalized to ground acceleration (COV=1.2), while the contrary occurs if the average spectrum normalized to ground displacement is used (COV=0.3).

The estimation of maximum deformations is a key issue for flexible systems like tall buildings and base isolated structures; the latter usually have fundamental periods between 2 and 3. Maximum displacement response is also relevant for displacement-based design. Figure 6 shows average response displacement spectra normalized to peak ground displacement D; it can be seen that elastic systems with periods $1.5<T<4$ have maximum displacements of the order of 2D, and approximately so do inelastic systems as well.

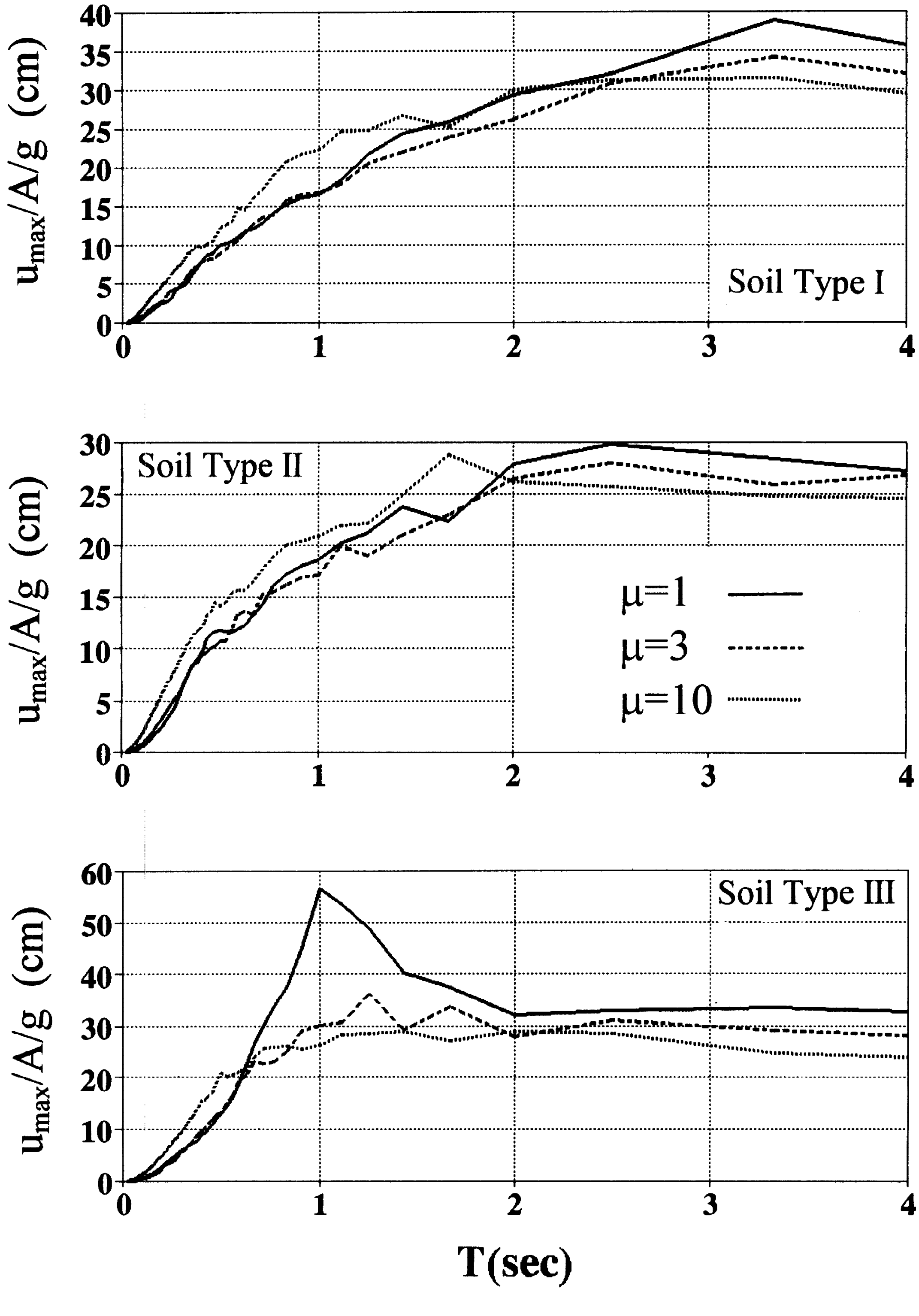

Fig. 3 Maximum Displacement Spectra Derived from Average Acceleration Spectra Normalized to Peak Ground Acceleration

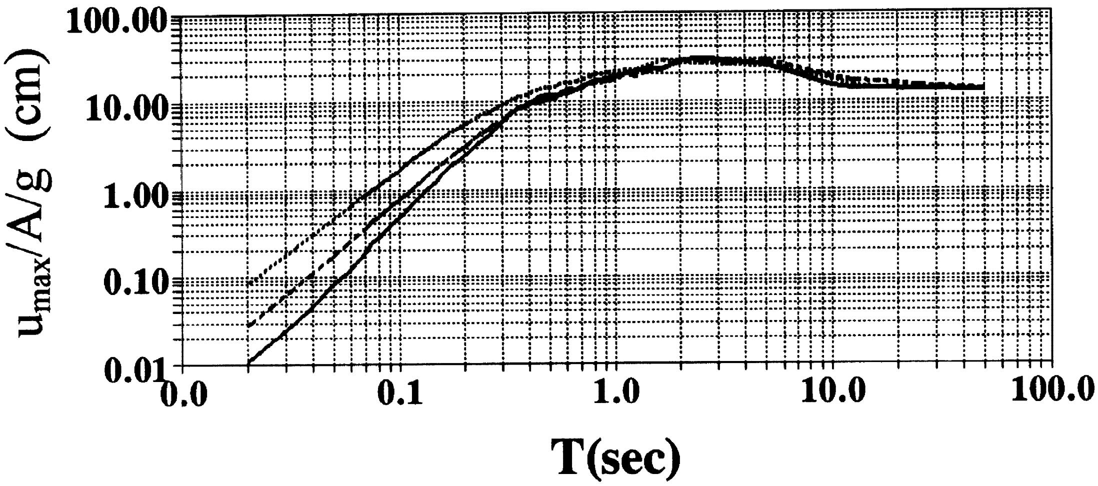

Fig. 4 Maximum Displacement Spectra Derived from Average Acceleration Spectra Normalized to Peak Ground Acceleration for Soil Type II

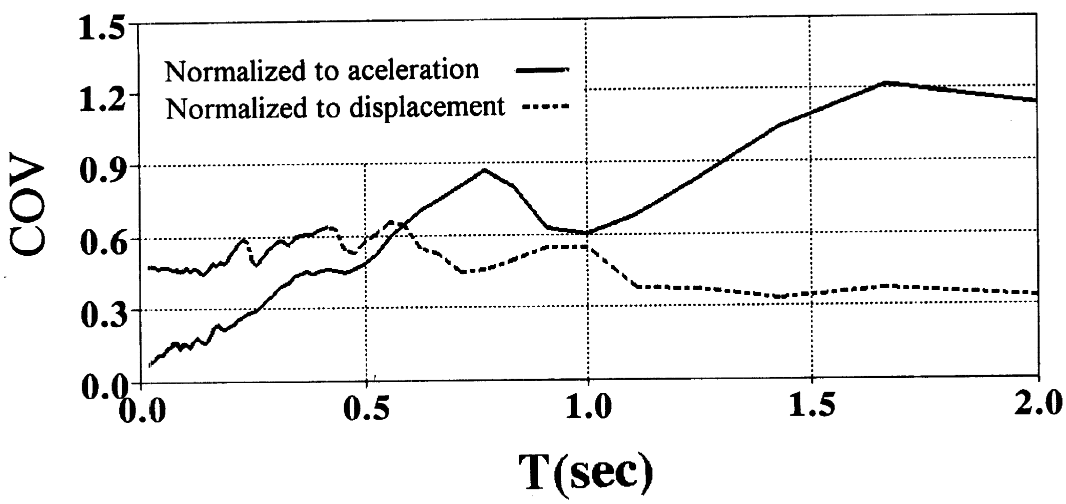

Fig. 5 Coefficient of Variation of Average Spectra Depending on the Normalization Parameter

Unfortunately, peak ground displacements calculated by double integration of the acceleration histories are quite unreliable since conventional data processing of strong-motion accelerograms seems to eliminate most long-period information. Iwan, 1994, has shown that peak ground displacements of near-field records may be considerably underestimated from standardly processed records; his study of the Lucerne Valley record from the Landers earthquake gives peak ground displacements of 75 and 260 cm for the two horizontal motion components, while the results of processing with the conventional method are 3.5 and 9.1 cm respectively. Low-frequency distortions of response spectra due to incorrect specification of the initial conditions of the oscillator have been also reported (Pecknold and Riddell, 1978). It is believed that processing procedures should be reviewed, since proper estimation of peak ground displacements is crucial for long period structures.

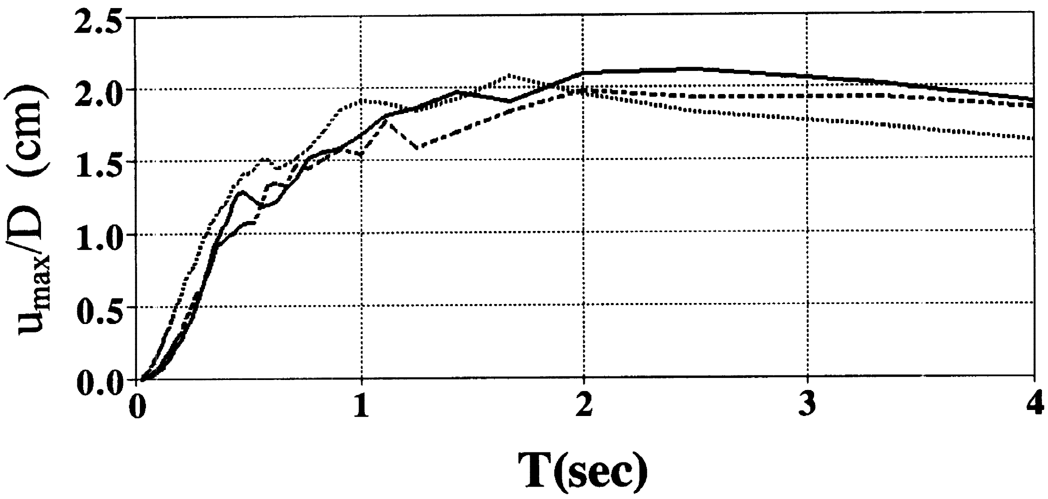

Fig. 6 Maximum Displacement Spectra Derived from Average Spectra Normalized to Peak Ground Displacement

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