Scaling Earthquake Motions in Geotechnical Design

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SYNOPSIS The traditional approach to scaling earthquake ground motion for geotechnical design applications is based on peak ground acceleration. This approach is useful when the physics of the problem depends linearly only on the nature of the high frequency (short wavelength) inertial part of strong motion. For nonlinear response analyses, the representative strain (~ velocity) and the number of stress reversals (~ duration of shaking) must also be considered. For long (large) structures (bridges and dams), the relative displacement of multiple foundations and the quasi-static deformation of the complete structure may contribute to the largest design levels. Thus, the modern design criteria must consider all the relevant scaling parameters and not just the high frequency inertial part of strong earthquake shaking. In this paper, the above points are illustrated via several examples.

INTRODUCTION

On geological time and length scales, geotechnical engineering deals with the most recent and the shallowest deposits on the earth surface. The vertical dimensions of the soil deposits are measured by hundreds of feet. Their horizontal dimensions are one to two orders of magnitude larger, and often of irregular shape, determined by topography of base rocks and by continuous action of water, wind and tectonic forces. In earthquake engineering, from the wave propagation view point, soil materials represent very thin, low velocity, highly attenuating, surface wave guides.

The significance of soil deposits, in influencing the nature of strong earthquake shaking, and damage of man made structures, has been studied quantitatively since 1920's and 1930's (Kanai, 1949, 1951; Duke, 1958). Thin vertical dimensions of soil deposits, lack of strong motion data prior to 1970's, and lack of detailed geophysical exploration work at sites where strong motion data has been recorded, have all contributed to a relatively slow introduction of wave propagation theory and non-linear mechanics into the analysis of soil response to earthquake excitation. Analysis of the dynamic response of soils is complicated by its low strength, leading to a variety of non-linear responses (settlement, liquefaction, slope instability, slides). While the examples of surface expressions of non-linear response of soils can be mapped in epicentral regions following strong earthquake shaking, it is not clear to what depth and how this non-linear response is manifested below ground surface.

For rational geotechnical design, one needs to have specific amplitudes, frequency content, duration of shaking, and other related characteristics of the forcing functions (rotations, strains, curvature, energy, power,...) to solve the governing equations for the specific problems. To this end in this paper, a brief summary of some of the recent work on describing these scaling variables and functions will be presented. Some examples on how to select the appropriate scaling parameters (functions) will also be given.

STRONG MOTION AMPLITUDES

If the analysis calls for a linear representation, and a transfer function approach is used, then the strong motion amplitudes can be described best by the corresponding Fourier Amplitude Spectrum (FS). Response spectrum amplitudes (PSV) can be used also, but FS is more directly related to the strong motion amplitudes and can be employed to evaluate PSV amplitudes (Udwadia and Trifunac, 1974). Before the age of computers, when only analog film or paper, records, of strong motion were available, simple scaling in terms of peak ground acceleration was the only practical alternative. Numerous empirical equations have been developed, to predict peak accelerations, in terms of earthquake magnitude, M, epicentral (R), or hypocentral (Δ) distances to the earthquake source, and some representation of the local recording conditions (Trifunac and Brady, 1975). Since mid 1970's (Trifunac, 1976a) it has been proposed that scaling of strong motion amplitudes by peak acceleration can lead to difficulties, when the shape of design spectra is an important factor in determining the final design. During the last 20 years, direct scaling of spectral amplitudes gained popularity, but peak accelerations are still often used.

Translation. Peak acceleration and peak velocity of strong ground motion can be computed from (Trifunac, 1976b)

$$\log_{10}\begin{Bmatrix} a_{\max,p} \\ \mathrm{v}_{\max,p} \end{Bmatrix} = M + \log_{10} A_0(R) - \log_{10}\begin{Bmatrix} a_0(M,p,s,\mathrm{v}) \\ \mathrm{v}_0(M,p,s,\mathrm{v}) \end{Bmatrix}, \quad (1)$$

where p is the confidence level associated with the approximate bounds $a_{\max,p}$ and $v_{\max,p}$, for the peaks of ground acceleration and ground velocity, $a_{\max}$ and $v_{\max}$; s represents the type of site conditions $s = 0$ for sediments $s = 1$ for "intermediate" rock; and $s = 2$ for basement rock, and v is used to describe the component direction $\mathrm{v} = 0$ for horizontal and $\mathrm{v} = 1$ for vertical direction). $\log_{10} A_0(R)$ versus epicentral distance, R, can be approximated by $\log_{10} A_0(R) \sim -1.40 - f(R)$, where

$$f(R) = \begin{cases} R/50 & \text{for } R \leq 75 \text{ km} \\ 1.125 + R/200 & \text{for } 350 \geq R \geq 75 \text{ km.} \end{cases} \quad (2)$$

The change of slope at $R = 75$ km reflects the fact that for greater distances the main contribution to strong shaking comes from surface waves, which are attenuated less rapidly ($\sim 1/R^{\frac{1}{2}}$) than the near-field and intermediate-field ($\sim 1/R^2$ to $1/R^4$), or far-field body waves ($\sim 1/R$). The scaling functions $a_0(M,p,s,\mathrm{v})$ and $v_0(M,p,s,\mathrm{v})$ are

$$\log_{10} \left\{ \begin{matrix} a_0(M,p,s,\mathrm{v}) \\ v_0(M,p,s,\mathrm{v}) \end{matrix} \right\} = \begin{cases} ap + bM + c + ds + e\mathrm{v} \\ +fM^2 - f(M - M_{\max})^2, & \text{for } M \geqq M_{\max}, \\ ap + bM + c + ds + e\mathrm{v} \\ +fM^2, & \text{for } M_{\max} \geqq M \geqq M_{\min}, \\ ap + bM_{\min} + c + ds + e\mathrm{v} \\ +fM_{\min}^2, & \text{for } M \leqq M_{\min}, \end{cases} \quad (3)$$

where the scaling coefficients are given in the Table I.

$$\Delta = S \left(\ln \frac{R^2 + H^2 + S^2}{R^2 + H^2 + S_0^2} \right)^{-1/2} \quad (7)$$

Δ_0 is equal to Δ evaluated at $R = R_0$. R_0 is computed from the condition that the slope of $Att(\Delta, M, T)$ versus distance be equal to $-1/200$. At this R_0, $Att(\Delta, M, T)$ coincides with the Richters empirical attenuation equation for Southern California $\log_{10} A_0(R)$ (Richter, 1958).

The attenuation of spectral amplitudes depends explicitly on several source characteristics. R is epicentral distance and H is focal depth, both in kilometers. S_0 is coherence radius at the source, and is approximated by $S_0 \sim \beta T/2$. S is the characteristic source dimension which can be represented by

$$S = \begin{cases} -25.34 + 8.51M & \text{for } 3 < M < 6 \\ 30 & \text{for } M > 6.5. \end{cases} \quad (8)$$

Thus, through S, $Att(\Delta, M, T)$ and $f(M, \Delta, T, \mathrm{v}, h, s_L)$ depend on the earthquake magnitude M. Both depend on T through $A_0(T)$ and S_0 (Trifunac and Lee, 1990).

Table 1

Function	a	b	c	d	e	f	$M_{\min}$	$M_{\max}$
$\log_{10} a_0(M,p,s,v)$	-0.898	-1.789	6.217	0.060	0.331	0.186	4.80	7.50
$\log_{10} v_0(M,p,s,v)$	-1.087	-2.059	8.357	0.134	0.344	0.201	5.12	7.61

Soon, Eqn (1) will be 20 years old. It was developed from 186 "free field" strong motion recordings, and so far, its robust character was not compromised by the recent recordings in Southern California (Trifunac et al. 1994). Soon it will be refined and recalibrated with the abundant strong motion data and by considering detailed region specific frequency dependent attenuation of strong ground motion.

The modern empirical scaling equations for FS and PSV spectral amplitudes started to evolve in mid 1970's. Those employ the scaling parameters which depend on the earthquake source, propagation path and local site conditions. A typical equation is of the form (Lee and Trifunac, 1993)

$$\log_{10}[PSV(T)] = f(M, \Delta, T, \mathrm{v}, h, s_L) \quad (4)$$

where T is period of motion. h represents the depth of sediments and s_L is the soil classification parameter. Those describe the local site conditions (Trifunac, 1990a). When some of these parameters are not available, other related equations can be employed (Trifunac, 1993a,b). For example in place of M, the site intensity may be used.

All recent empirical scaling equations for FS and PSV amplitudes employ frequency (period) dependent attenuation of spectral amplitudes, with distance. This can be described by

$$Att(\Delta, M, T) = \begin{cases} A_0(T) \log_{10} \Delta & R < R_0 \\ A_0(T) \log_{10} \Delta_0 - \frac{R - R_0}{200} & R > R_0 \end{cases} \quad (5)$$

where

$$A_0(T) = \begin{cases} -0.732 & T \geq 1.8 \text{ sec} \\ a + b \log_{10} T + c (\log_{10} T)^2 & T < 1.8 \text{ sec} \end{cases} \quad (6)$$

and $a = -0.767$, $b = 0.272$ and $c = -0.525$, for example, in California (Trifunac and Lee, 1990). The representative source to station distance, Δ, is

Equations of the form of Eqn (4) can be extended to long periods (Trifunac 1993a, 1994a,c), as $T \to 100$ sec., and to short periods $\frac{1}{25} > T > \frac{1}{100}$ (Trifunac 1994b). These equations are too lengthy and detailed for presentation in this paper. Scaling of FS has been described by Trifunac (1993a, 1994a,b), and the scaling of PSV spectra by Lee (1989, 1990a, 1991 and 1993) and Trifunac (1994c).

Rotation. Response analyses and design of structures sensitive to excitation by short waves way have to consider contributions to response from rotational acceleration of strong ground motion. For example, tall structures, on soft soil, when excited by large long period motion, may experience large response caused by the rocking ground motion (Gupta and Trifunac, 1991). Soil pressure on retaining walls and stability of slopes are especially sensitive to excitation by SV and Rayleigh type waves which are accompanied by large rocking motions, particularly at high frequencies.

Rocking $-\psi$. Rotation about the transverse component of strong motion (radial component is oriented away from the earthquake source) can be computed exactly from the wave propagation theory and can be described either by spectra or by time series. For a fixed frequency, the rocking angle $|\psi| = \frac{\omega}{c_x}|u_2|$, where ω is circular frequency, u_2 is vertical component of ground motion and c_x is phase velocity in the direction of propagation. The computer program SYNACC, for example, uses such an expression to compute rocking accelerograms simultaneously with calculations for transverse acceleration components (Lee and Trifunac, 1987).

Torsion$-\psi_T$. We call rotational acceleration about vertical axis, "torsional" acceleration. It is caused by propagation of SH and Love waves, and can be computed from $|\Psi_T| = \frac{1}{2}\frac{\omega}{c_x}|u_3|$, where u_3 is transverse component of strong motion displacement (Lee and Trifunac, 1985). Torsional acceleration increases the equivalent eccentricity of nonsymmetric structures and may contribute large antisymmetric components of excitation and response in long structures (earth and rock fill dams, rigid concrete bridges, long buildings).

Strains at ground surface. Strains caused by passage of plane body and surface waves can be computed exactly from ground displacement amplitudes, angles of incidence θ_0, and appropriate wave numbers (Trifunac, 1979). This can be used then to construct the time histories of strain during strong shaking, to compute strain spectra and to evaluate the peak strain response of soil. Lee (1990b) used SYNACC computer program to evaluate the extreme strain amplitudes in epicentral region of large earthquakes. For a soft surface layer, with shear wave velocity equal to 50 m/sec and for peak accelerations and velocities about twice larger than those ever recorded, he obtained strain amplitudes of the order of 10^{-1}. He also found that an excellent estimate of peak strain amplitudes can be obtained by dividing peak ground velocity by the shear wave velocity in the medium. Recorded peak velocity of strong motion divided by the average soil velocity in the top 30 m results in shear strains between 10^{-6} and 10^{-2}.

Curvature. Long and stiff structures (large stiff foundations, large pipes, tunnels) should be designed for bending deformation caused by large curvature (small radii of curvature, ρ) during passage of seismic waves through soft soil. It can be shown that $\rho \quad c^2/\ddot{u}$, where c is the phase velocity and $\ddot{u}$ is the corresponding component of ground acceleration. The computer program SYNACC can be used to evaluate the possible range of radii of curvature for given soil and geologic site conditions. Trifunac (1990b) found that for large accelerations (1 to 3g), near the epicenter, and when the local soil velocities are very low (e.g. 50 m/sec), the radii of curvature may be as low as one to several hundred meters.

WAVE AMPLITUDES, STRUCTURAL DIMENSIONS AND WAVELENGTHS IN SOIL

Most engineering analyses of response to earthquake shaking are based on one-dimensional analysis (vertically propagating shear waves, lumped mass models in structural dynamics, with synchronous excitation at the base), where it is assumed that the horizontal phase velocities are infinite. In those analyses the earthquake shaking is represented by response spectra or by accelerograms representing motion of one reference point (e.g. center of the foundation). When foundation soil has low velocity, β, the incident wave lengths ($\lambda = \beta T$) of waves with period of motion T become short (relative to the dimensions of the foundation, say L, or height of the first story, H) and the formulation and analysis of the problem must consider two- or three-dimensional wave analysis. Designating the amplitude of the incident wave by a, and the ratio of the structural dimensions to the wavelength by η ($= L/(CT)$ or $= H/(CT)$), it can be shown that for

$$\frac{L}{a} \text{ or } \frac{H}{a} \gtrsim 10^{\eta+4.5}, \qquad (9)$$

analyses ignoring the wave propagation in soil will give good results. For

$$\frac{L}{a} \text{ or } \frac{H}{a} \lesssim 10^{\eta+2.6}, \qquad (10)$$

only two- or three-dimensional wave propagation analysis can be expected to yield meaningful results (Todorovska and Trifunac, 1990). In the region between Eqns (9) and (10) specific details of the problem will guide one to determine whether wave propagation analysis is required.

For $\eta \ll 1$ wave scattering and diffraction will be negligible. For typical soils, for strains $\epsilon < 10^{-n}$, when $n = 4$ to 5, one can employ both linear analysis and can ignore wave propagation effects and still obtain very representative and accurate results. On the other hand, even for $\eta \sim 1$, the borderline given by Eqn (10) implies that non-linear response and wave propagation must be considered simultaneously, if analysis is to capture the physically realistic behavior of the problem. The current state of the art in non-linear wave propagation is such that only very sophisticated studies can afford to go into such detail. Therefore one must carefully evaluate the adequacy of all analyses which use linear (or equivalent linear) equations, ignore two and three dimensional wave propagation in the problem, and whose governing parameters are below the limit suggested by Eqn (10).

LOCAL AMPLIFICATION

A plane wave propagating from "hard" to soft medium will experience at most a factor of two amplification of its amplitude. Repeated subsequent reflections and interference within soft soil and sediments will lead to further amplification or attenuation of the incident waves. In irregular three-dimensional soil and sedimentary layers, the amplification of incident waves will increase monotonically between $T = \infty$ and $T \sim 4H_L/\beta$. Beyond period $\sim 4H_L/\beta$ (for which the layer thickness H_L coincides with one quarter of the wave length), interference will result in complicated amplification and deamplification patterns, which will depend on the ratio of impedance in the soil and in the underlying medium, $(\beta_L\rho_L)/(\beta\rho)$, where β and ρ are the shear wave velocity and material density in the basement material and β_L and ρ_L are the corresponding quantities in the layer. These amplification patterns will depend on the geometry of the soil layers, on the azimuth of wave arrival and on the incidence angles in the vertical plane. While for $T \lesssim 4H_L/\beta$ there may exist a simple repeatable amplification from one earthquake to the next, for $T > 4H_L/\beta$, the amplification will be some complicated function, changing from one event to the next, so that one can work perhaps only with the average of spectrum amplification factors. For $a = \frac{\rho_L\beta_L}{\rho\beta}$ the average amplitude of this amplifications is $\sim \frac{1}{2}\ln\frac{16}{a^2}$ (Trifunac, 1990a).

In representing and estimating amplification factors we employ various local site indicator variable. To describe the local soil conditions we use $s_L = 0$ for "rock" soil sites, $s_L = 1$ for stiff soil sites and $s_L = 2$ for deep soil sites (Trifunac, 1990a). More recently there is a trend to consider the average velocity of shear waves, $\overline{\beta}$, in the top 30 meters of soil deposits. It should be noted that neither s_L nor $\overline{\beta}$ have been chosen because some rational analysis of local amplification indicated that such indicator variables are physically most representative and stable for use in engineering applications. Indeed both variables have been created out of necessity, recognizing that this is all the data we can have at present. Detailed studies on what should be the indicator variables to describe the local soil conditions have never been carried our. On a large, geological, scale we use $s = 0$ for sediments, $s = 2$ for basement rock and $s = 1$ for intermediate sites (Trifunac, 1990a). When available, we prefer to use the depth of sediments beneath the site, h, the horizontal dimensions of the sedimentary basin, R, and the angle subtended by the basement rocks, φ, which are capable of reflecting wave energy towards the site (Novikova and Trifunac, 1993a,b; 1994a,b). Depending on which site parameters are available, different regression models have been developed, for estimating spectral amplitudes of strong motion and the associated duration of this motion.

ANELASTIC ATTENUATION AND Q

Attenuation of high frequency strong motion waves is not understood well. It results from the action of several attenuation mechanisms simultaneously. It can be described empirically by $e^{-\pi\tau f}$. τ is proportional to the travel time along the ray and inversely proportional to Q. For soil deposits, near ground surface, Q can be as small as 1 to 10. It increases with depth and for igneous rocks it is measured in thousands. For frequencies higher than 1 Hz, $Q \sim f^n$ where $n \sim 1$ (Trifunac 1994b).

In California, for small epicentral distance, $\beta \sim 3.35 + .00175\Delta$, approximations $\tau \sim [f(1.23 - .00405\Delta)]^{-1}$ and $Q \sim (.367\Delta - .0014\Delta^2)f$ are consistent with high frequency Fourier spectrum amplitudes of strong motion for $\Delta < 100$ km (Trifunac 1994b).

DURATION OF STRONG MOTION

Fourier and response spectrum amplitudes describe the amplitudes of the total energy in ground shaking, and the amplitudes of relative peak response. Since the destructiveness of strong motion depends on the time rates of release of this energy, one must specify the duration or the time interval(s) during which this energy arrives to the site and is available to excite the structures and the soil. If this rate of energy input is higher than what the system is capable of absorbing, damage will be initiated. If the high power input persists the system may be damaged severely or may collapse. Of course, during large amplitude (non-linear) response, the phenomena associated with dynamic stability will become complicated functions of many parameters, with the duration of shaking being only one of them. In geotechnical engineering, analyses of liquefaction potential, of slope stability, of slides and in general of non-linear response of soils, will also depend on the duration of shaking (number of stress reversals and the number of times certain acceleration level has been exceeded). To provide a direct measure of the time during which strong motion energy is fed into a structure, we employ functionals of strong motion of the form $\int_0^{t_0} f^2(t)dt$, where $f(t)$ is acceleration, velocity or displacement, which can all be related to energy associated with incoming waves ($f(t)$ = ground velocity) or the energy of structural response ($f(t)$ = ground acceleration, and $f(t)$ = response). We define the strong motion duration to correspond to the time interval during which 90% of this energy is realized (Novikova and Trifunac, 1993a,b, 1994a,b). Then we represent the total duration of strong motion at a site by

$$\text{dur} = d_0 + d_\Delta + d_{\text{region + site}} \tag{11}$$

where d_0 is duration of faulting, d_Δ represents prolongation of strong motion caused by dispersion of seismic waves and $d_{\text{region + site}}$ is the additional prolongation caused by late arrivals and reflections in the vicinity of the recording site.

The duration of the earthquake source depends on W (fault width) L (fault length) and v (dislocation velocity) most directly, and since W and L may be related exponentially to the earthquake magnitude M,

$$d_0 = \alpha \exp(\gamma M). \tag{12}$$

Depending on the frequency range and the features of the fault motion of interest (overall duration of faulting $d_0 = \frac{L}{v} + \frac{W}{2\beta}$, dislocation rise time T_H, duration of breaking major asperities L_A/v, where L_A is the size of the asperities, etc.) $\alpha \sim .03$ to .1 and $\gamma \sim .4$ to 1. Transient, larger displacement pulses will be associated with the passage of spreading dislocations. At near recording sites, the fault width (W/v) and the dislocation rise time (T_H) will influence the pulse duration. The duration of high frequency large acceleration pulses will depend on the duration associated with breaking asperities, on the number of such asperities, and on their proximity to the recording site.

Since $1/f_1 = d_0$ and $1/f_2 = W/v$, it is seen that the duration of faulting and the corner frequencies of the near-field spectral amplitudes are directly related. The dissociation rise time $T_0 \sim \overline{u}\mu/(\sigma\beta)$ (where $\overline{u}$ is dislocation amplitude and σ is the effective stress deep) is inversely proportional to the intermediate frequency ($f_2 < f < f_0$) spectral amplitudes of strong motion via σ.

DISCUSSION AND CONCLUSIONS

It should be clear from the above that the strong motion amplitudes cannot be described only by one scaling variable (e.g. peak acceleration, peak velocity, or selected spectral amplitudes). Also, one scaling function (or functional) will not suffice for all applications. Therefore, every sound design approach will first select the physical model most representative for the problem to be solved, and will then use the properties of this model to select the needed scaling parameters and functions of strong motion.

To illustrate this, assume that the energy arriving at a site will be dissipated in the soil and that some of this energy will contribute towards an increase in the pore pressure. In absence of other loads, on level ground, the increase in pore pressure required to produce liquefaction may be proportional to σ_0, the overburden pressure. Computing the Fourier amplitude spectra of strong motion acceleration from Eqn (4), it is possible to evaluate the energy observed at the site from ($\omega = 2\pi/T$)

$$cn \sim \int_0^\infty \left|\frac{FS(\omega)}{\omega}\right|^2 dw. \tag{13}$$

It is also possible to derive the borderline values of $\overline{N}$, such that for smaller values liquefaction will occur, and for larger values it will not occur,

$$\overline{N} = c\left(\frac{cn}{\sigma_0^{3/2}}\right), \tag{14}$$

where c and n can be determined from data on observed liquefaction (or no liquefaction) during past earthquakes. $\overline{N}$ is the number of blows in the standard penetration tests, corrected for overburden pressure.

If the seismic wave energy recorded at the site was to be approximated by peak ground velocity, i.e. $cn \sim v_{\max}^2 dur$, or by the peak amplitude of Fourier amplitude spectra of strong motion velocity, FV, near period of 0.4 seconds, then

$$\overline{N} = c_1 \left(\frac{v_{\max}^2 dur}{\sigma_0^{3/2}} \right)^{1/n_1} \quad (15)$$

and

$$\overline{N} = c_2 \left((FV)^2 \frac{1}{\sigma_0^{3/2}} \right)^{1/n_2} \quad (16)$$

would be analogous to Eqn (14), and with c_1, c_2, n_1 and n_2 determined from the regression analysis. In the above, the duration of strong motion can be approximated by

$$dur = 7.8 - 3.86M + 0.57M^2 + 0.07R + 1.14(1 - s/2)$$

and $v_{\max}$ can be computed from Eqn (3)

The shear stress on the horizontal surface in the layer of sand is proportional to the shear strain ($\sim v/\beta$) and the Lame constant ($\mu = \rho\beta^2$). Thus

$$\frac{\tau}{\sigma_0} \sim \frac{v}{\beta} \frac{\mu}{\sigma_0}. \quad (17)$$

Since the number of stress reversals is proportional to the duration of strong shaking, dur, it is possible to write

$$\overline{N} = c_3 \left(\frac{v \ \mu \ dur}{\sigma_0} \right)^{1/n_3}, \quad (18)$$

with c_3 and n_3 again determined from regression of the observed data on liquefaction. The details on how Eqns (13) through (18) can be derived are beyond the scope of this paper. Here, the purpose of these equations is only to illustrate that even the simplest analysis for developing the borderline equations for $\overline{N}$ involves broad band representation of FS, or $v_{\max}$ and dur, or just FV. None of these functions or variables can or should be "derived" from one scaling parameter, like peak acceleration. Some authors who use peak acceleration to determine the liquefaction potential at the site introduce correction factors to account approximately for longer (further or large magnitude events) or shorter (closer and or smaller M) duration of strong motion, but it should be clear that one scaling parameter just cannot describe all of the above trends.

Other design tasks will call for different scaling variables and different functionals of strong ground motion. Stability of slopes and earth pressure on retaining walls and bridge abutments should include rocking component of ground acceleration in the analyses, especially when significant contributions to strong shaking from SV- and Rayleigh waves are expected. Design of long bridges, pipelines, and tunnels will require consideration of near surface strains, curvature (Trifunac, 1990b), and of differential motions at different support points. On level ground, differential motion can be evaluated from the amplitudes of near-surface strains, and from expected dislocation amplitudes (for sites crossing faults, Trifunac, 1993a), but for irregular topography and irregular geometry of soil deposits, it will be necessary to consider differential motions caused by the wave scattering from surface topography and from wave interference in sedimentary basins (Moeen-Vaziri and Trifunac, 1988). In non-linear numerical algorithms for computation of response of the waste disposal sites, for the design of plastic insulators and their soil protective layers, it will be necessary to specify differential motion at the base rock. This will require three-dimensional scattering and diffraction from irregular surface topography to be solved in real time.

The above discussion considered the scaling of strong motion amplitudes for one earthquake, at a given epicentral distance. In a realistic setting, a site will be exposed to many different earthquakes occurring in many different directions and of different sizes (magnitudes, epicentral intensities). Thus it becomes necessary to consider distribution functions of strong motion amplitudes, duration, strains, rotations..., or some appropriate form of their functionals. For scaling aimed at selection of design criteria, distributions of chosen motion characteristics can be given by Uniform Hazard Functionals (Todorovska, 1994a-d). For more specific applications, for example, simplified representative of liquefaction potential in terms of $\overline{N}$, distribution functions of $\overline{N}$ can be presented. For all such applications, the above scaling for single events then becomes a building block for the development of distribution functions of desired characteristics of strong ground motion. The methodology of seismic hazard analysis then balances the contributions from all future events (with specified, but, when necessary, different type of occurrence rates), arrival direction (local amplification depends on direction of wave arrival) and local site amplification patterns.

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