EARTHQUAKE GROUND MOTION SIMULATION AND RELIABILITY IMPLICATIONS

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Seismic loads have been a dominant concern in design of buildings and structures in western United States for many years. Mid-America is a region of low to moderate seismicity, infrequent moderate to large events have occurred in the past and can occur again causing large and widespread losses due to relatively slow attenuation. For purpose of performance evaluation of structures, a phenomenological model is developed for generation of earthquake ground motions. The simulation procedure is based on the latest information of seismicity, the most recent ground motion models and simulation methods. A strong emphasis is placed on uncertainty modeling and efficiency in application to performance evaluation and reliability analysis. Site locations of special interest are Memphis TN, Carbondale IL, St. Louis MO and Santa Barbara CA, because they present U.S. cities in Mid-America and western United States of different seismicity. The soil condition of a city is approximately modeled by a generic profile. Ground motions at bedrock, however, are also generated for the Mid-America cities such that if detailed information of local soil variation is available, one can use appropriate soil amplification computer software to obtain the surface ground motions. Suites of ten ground motions are selected to be compatible with the uniform hazard response spectra of a given exceedance probability. They represent the seismic hazard from the causative faults surrounding a given site location. They also allow an accurate and efficient evaluation of structural performance and reliability. The uniform hazard ground motion suites for the three Mid-America cities are also available on the web at http://mae.ce.uiuc.edu/Research/RR-1/GMotions/Index.html.
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CHAPTER 1
INTRODUCTION

1.1 Overview

Seismic loads have been a dominant concern in design of buildings and structures in western United States for many years. Mid-America is a region of low to moderate seismicity, infrequent moderate to large events have occurred in the past and can occur again causing large losses in widespread areas due to relatively slow attenuation. The performance of the building stocks and other structures under future earthquakes has received attention of the profession, especially in the Mid-America cities that are in close proximity to the New Madrid and Wabash Valley seismic zones.

After the recent Northridge and Kobe earthquakes, a main concern of engineers is the performance-based design. For example, a design framework has been proposed in recent building design documents (e.g. 1997 NEHRP, FEMA 273) to ensure that various performance objectives according to building categories are met. For this purpose, uniform hazard ground motions corresponding to various levels of probability of exceedance are needed for nonlinear time history analyses and structural performance checks. When available earthquake records are not sufficient to determine the uniform hazard ground motions, one may: (1) scale existing ground motion records to match target response spectra, (2) use ground motion records from other seismic zones with similar tectonic environments, or (3) generate synthetic ground motions based on seismotectonic characteristics of the region surrounding the site location.

All three approaches were used in the recent SAC steel project, which provided ground motion suites for Los Angeles, Seattle and Boston for various probability levels (Somerville et. al., 1997). These ground motions are obtained by scaling records of past earthquakes and time histories based on broadband simulations. The median response
spectra of the SAC ground motions for stiff and soft soil sites match the target response spectra in the 1994 NEHRP provisions. The scaling factors used in the SAC ground motions vary widely from 0.27 to 10.75. Also, due to the large computational efforts required in the broadband simulation procedure, generation of a large samples (say thousands) is time consuming and expensive. At most locations in Central and Eastern United State, the number of ground motion records is generally not enough for the first two approaches, especially for low levels of probability of exceedance. Simulation of earthquake motions is therefore necessary. A phenomenological model of simulation is proposed in this study, which allows an efficient simulation of a large number of ground motions and probabilistic performance analyses.

The simulation procedure is an extension of that in Collins et al. (1995). It is based on the latest information of seismicity and the most recent ground motion models and simulation methods appropriate for engineering applications. In view of the differences in earthquake records and seismotectonic data available in Mid-America and Western United States, two slightly different procedures are developed. Since in Western United States data are more available and seismic zones are better defined, the simulation procedure for this area is more data based. All earthquakes with distance less than 50 km to the site are modeled as either a dip-slip or strike-slip fault according to the tectonic information. In Mid-America, due to the scarcity of records of engineering interest, the simulation method is largely theory and model based. A strong emphasis is placed on uncertainty modeling and efficiency in application to performance evaluation and fragility analysis. The procedure may be improved as more knowledge on earthquakes in the corresponding regions and more accurate and efficient methods of ground motion modeling become available, especially in Mid-America.

Site locations of special interest in this study are Santa Barbara CA, Memphis TN, Carbondale IL, and St. Louis MO. These cities are selected for study because they present a wide cross section of cities at risk. Since ground motions are strongly dependent on local soil condition and yet the soil profile variation within a city has not been mapped in detail, in this study the soil condition of a city will be approximately
modeled by a generic profile. Ground motions at bedrock, however, will also be generated for Mid-America cities such that if detailed information of local soil variation is available, one can use appropriate computer software to include soil amplification in the surface ground motions.

1.2 Objective And Scope

The objective of this study is to generate uniform hazard response spectra and ground motions for probabilistic performance evaluation (fragility analysis) and loss estimation of buildings under future earthquake excitations. To achieve this objective, the following studies are required:

1. Develop an efficient procedure to generate synthetic ground motions based on regional seismicity.
2. Develop a procedure to construct uniform hazard response spectra and generating uniform hazard ground motions for structural performance evaluation.

1.3 Organization

In Chapter 2, a simulation procedure based on a 2-corner point source model and a finite fault model is proposed to generate a large number of ground motions for Mid-America cities. The seismological data from USGS Open File Report 96-532 are used. Probabilistic distributions of various seismic parameters (magnitude, epicenter, focal depth, attenuation uncertainty, etc.) are assumed based on field observations and then used to simulate earthquake sources and path effect. An empirical correlated random field model is used to simulate asperity in magnitude-8 events in the New Madrid seismic
zone. The local site effect is considered by using quarter wavelength method. After baseline correction following the USGS BAP routine (Converse, 1992), the resulting ground motions are used for statistical study.

In Chapter 3, a more databased ground motion model is developed for Western United States. The seismological data from the 1995 WGCEP report (Working Group on California Earthquake Probabilities) are used. The Fourier spectrum of Trifunac (1994) was used in simulation and then a near-source factor is incorporated in the simulation to account for some of the important near-source effects (Sommerville et al. 1997).

In Chapter 4, uniform hazard response spectra results for B/C boundary site are constructed based on statistical analyses of synthetic ground motions. They are compared with USGS, FEMA-273 design spectra and similar results in the literature. A period-independent procedure is then developed to select suites of uniform hazard ground motions for Mid-America cities at 10% and 2% probabilities of exceedance in 50 years. Linear structural analyses are performed to verify that the selected uniform hazard ground motion suites can be used for unbiased response estimation.

In Chapter 5, nonlinear structural systems under uniform hazard ground motions are investigated for to ensure that such motions yield unbiased structural response estimates including the influence of system degradation on ductility reduction factor. An efficient method of constructing probabilistic performance curve is then proposed. The accuracy of using spectral acceleration versus that of using peak ground acceleration for fragility analysis is examined.

In Chapter 6, the significant conclusions of this study and recommendations for future research are summarized.
CHAPTER 2
METHOD OF SIMULATION – MID-AMERICA

2.1 Overview

The procedure proposed by Collins et al. (1995) is extended to simulate future seismic events and ground motions within a reference area over a given period of time (e.g. 10 years). The ground motion model basically follows the procedure suggested by Herrmann and Akinci (1999), which is based on a point-source simulation method SMSIM (Boore 1996). To catch some of the important near-source effects due to large events, however, the finite fault model by Beresnev and Atkinson (1997, 1998) is also used for magnitude-8 events. The soil amplification is modeled by the quarter-wavelength method by Boore and Joyner (1991, 1997). The tectonic and seismological data are mainly taken from USGS Open-File Report 96-532 (Frankel et al. 1996). Possible future earthquake events are generated for a trial period of 10 years, and then each earthquake source propagates its seismic waves accordingly to site locations of interest. A total of 9000 simulations of 10-year period are carried out to provide a large number of ground motions for statistical analysis. The proposed simulation procedure is shown in Figure 2.1 and details of the simulation method are given in the following.

2.2 Selection of Locations and Soil Profiles

Site locations of special interest in this study are Memphis TN (35.117°N, 90.083°W), Carbondale IL (37.729°N, 89.246°W), and St. Louis MO (38.667°N, 90.190°W). These cities are selected for study because they present a wide cross-section of the Mid-America cities at risk. Since ground motions are strongly dependent on local soil condition and yet the soil profile variation within a city has not been mapped in
detail, the soil condition of a city will be approximately modeled by a generic profile. Ground motions at bedrock, however, will also be generated for these cities so that if detailed information of local soil variation is available, one can use appropriate soil amplification computer software to obtain the surface ground motions.

2.3 Seismicity and Tectonics

Based on the seismicity database from USGS Open-File Report 96-532 (Frankel et al. 1996), the annual occurrence rate of earthquakes with body wave magnitude $m_b$ greater than 5 per $0.1 \times 0.1$ degree square for the zone of interest is obtained as shown in Figure 2.2. Due to the lack of tectonic information, only moment magnitude 8 earthquakes in the New Madrid seismic zone are treated events with finite faults similar to the 1811-1812 New Madrid earthquakes. However, the faulting mechanism is simplified as a vertical strike-slip fault with a rupture plane of 140 km (along-strike) x 33 km (down-dip) according to Johnston (1996b). A 34.69° azimuth angle is assumed, which is an approximate estimate based on USGS OFR-96-532. The distance from the ground surface to the upper edge of the rupture plane is assumed to be 5 km. Attenuation, location of fault rupture, hypocenter, and asperity are varied from event to event to account for uncertainty in a future $M_w$-8 event. Figure 2.3 shows seismic zone of these $M_w$-8 earthquakes.

2.4 Reference Area and Occurrence Model

All known probable earthquake sources in the Central and Eastern United States (CEUS) are considered. For a given city, however, only those that fall within the reference area (effective zone) of the city and have significant contributions to the seismic risk are used in the simulation. Different cities may share some of the seismic sources and hence the same simulated seismic events. Considering the relatively low attenuation in the CEUS, the effective zone is defined as a circular area with a radius of
500 km centered at a specified site location, a value used by most researchers (e.g. USGS OFR-96-532). The occurrence in time is generated according to a Poisson process. The magnitude given the occurrence is then generated according to the magnitude distribution for events of moment magnitude less than 8 following the 1996 USGS Open-File Report.

The magnitude-8 events are generated separately using Poisson process with a mean recurrence time of 1000 years (Johnston 1996b, Frankel et al. 1996) and with an epicenter uniformly distributed within the New Madrid fault zone shown in Figure 2.3. The orientation of the fault is assumed to be along that of the NMSZ. Once rupture occurs, it propagates toward both ends of the rupture surface.

A total of 9000 simulations of 10-year period are carried out. As a result, 9260 ground motions are generated in Memphis (TN), 9269 in Carbondale (IL) and 8290 in St. Louis (MO). Figure 2.4 shows the epicenters and magnitudes of earthquakes corresponding to 600 simulations of 10-year records in the region of interest in the CEUS. It corresponds, therefore, to about 6000 years of records. Most of the events are located in the New Madrid, Wabash Valley and East Tennessee seismic zones. The proximity of the events to each city is shown. According to the simulation results, the occurrence rates of earthquakes of body wave magnitude 5 and above in the reference areas for Memphis, Carbondale, and St. Louis are 0.0989, 0.0980 and 0.0882 per year, respectively. These values are very close to the actual occurrence statistics.

Before the statistics of occurrence and magnitude from the USGS OFR-96-532 can be used in the simulations, the incremental seismicity rate provided by USGS needs to be converted into the cumulative seismicity rate using recurrence relation of Herrmann (1977), and the results are shown in Figure 2.2. In each $0.1^\circ \times 0.1^\circ$ cell during a 10-year period, the number of earthquakes, magnitudes and hypocenters are determined according to the procedures described in the following.
2.4.1 Number of Earthquakes

To determine the number of earthquakes in each 0.1°×0.1° cell during a 10-year period, a random variable $u_k$ with a uniform distribution between 0 and 1 is generated. The number of earthquakes is then determined from:

$$
\sum_{x=0}^{n_k-1} \left( \frac{x}{X!} \right) e^{-t\nu_k} < u_k \leq \sum_{x=0}^{n_k} \left( \frac{x}{X!} \right) e^{-t\nu_k} \quad ; \quad k = 1, 2, \cdots, N_{\text{cell}}
$$

(2.1)

where $\nu_k$ is the mean annual occurrence rate of earthquakes in cell $k$ with body wave magnitude greater than 5; $n_k$ is the number of earthquakes that occur with body wave magnitude greater than 5 in cell $k$ during a time interval of $t$ years ($t = 10$ in this study); and $N_{\text{cell}}$ is the total number of cells in the reference area of a specific site.

2.4.2 Magnitude

In most literature, the body wave magnitude is used for sizing earthquakes; e.g., in USGS OFR-96-532. The body wave magnitude $m_b$ is then converted into moment magnitude $M_w$ using an empirical relation of Johnston (1994):

$$
M_w = 3.45 - 0.473m_b + 0.145m_b^2
$$

(2.2)

where $m_b-5$ is equivalent to $M_w-4.71$. It should be noted that Equation 2.2 differs slightly from a revised relation of Johnston (1996a). To be consistent with the seismological data from USGS OFR-96-532, Equation 2.2 is used in this study. The cumulative distribution function of moment magnitude $F_{M_w}$ is then defined as:

$$
\begin{cases}
F_{M_w}(M_w \leq m) = 1 - \frac{N(m, m_k, \nu_k)}{N(4.71, m_k, \nu_k)} \quad ; \quad m = 4.71 - m_k + 10^{4.71-m_k} \cdot \frac{1 - 10^{m-m_k}}{1 - 10^{4.71-m_k}}
\end{cases}
$$

(2.3)
where \( m_x \) is the maximum moment magnitude in cell \( k \) (Figure 2.5); \( N(m, m_x, \nu_k) \) is the annual cumulative rate of earthquakes greater than moment magnitude \( m \) in cell \( k \). Note that \( 4.71 \leq m \leq m_x \).

2.4.3 Epicenter

It is assumed that earthquakes are equally likely to occur anywhere within a cell, which is numerically done by generating two random numbers of uniformly distributed and independent of each other along latitudinal and longitudinal directions.

2.4.4 Focal Depth

The focal depth distribution model proposed by Wheeler and Johnston (1992) is shown in Table 2.1. According to EPRI TR-102293 Report (1993), Wheeler and Johnston's model is more appropriate in the Eastern North America (ENA) due to their quality selection criteria. Since the 2-corner point source model (Atkinson and Boore 1995) is widely used in generating earthquake events in the CEUS, the Wheeler-Johnston distribution (1992) is more appropriate for this purpose. To exclude very shallow earthquakes within the soil stratum, a cut-off point at 1 km is used. Within each depth bin, the earthquake focus is then assumed to be equally likely to occur anywhere in the corresponding depth interval. The focal depth distribution of moment magnitude 8 earthquakes modeled by finite fault is shown in Table 2.2. Since magnitude 8 earthquakes are all assumed to occur within the NMSZ, which is classified as a Stable Continent Region (Frankel et al 1996), a focal depth distribution model of EPRI TR-102293 (1993) is used with minor revisions in view of the discretized depths required in the finite fault model. In the focal depth column of Table 2.2, the figures inside parentheses stands for the center of the corresponding depth bin and each depth bin represents a sub-fault segment along the down-dip direction.
2.5 Modeling of Ground Motions

2.5.1 Point Source Model

For point sources, the two-corner-frequency model (Atkinson and Boore, 1995) for generic S-wave type ground motions on hard rock is used since this model has been shown to give better ground motion prediction in the CEUS (Atkinson and Boore, 1998). The mathematical form for the Fourier amplitude spectrum of the ground motion using this two-corner point source model is:

\[ A(M_0, R_h, f) = E(M_0, f) \cdot D(R_h, f) \cdot P(f) \cdot I(f) \]  

(2.4)

where \( E(M_0, f) \) is two-corner-frequency source spectrum; \( D(R_h, f) \) is a diminution factor; \( P(f) \) is high-cut filter; \( I(f) \) is response indicator; \( f \) is cyclic frequency (Hz), \( R_h \) is focal distance (km), and \( M_0 \) is moment magnitude (dyne-cm). The various functions are:

\[ E(M_0, f) = C \cdot (2 \pi f)^2 \cdot M_0 \cdot \left[ \frac{1 - \zeta}{1 + \left( \frac{f}{f_A} \right)^2} + \frac{\zeta}{1 + \left( \frac{f}{f_B} \right)^2} \right] \]  

(2.5)

in which,

\[ C = \frac{R^{6g} \cdot FS \cdot V}{4 \pi \cdot \rho_0 \cdot \beta_0^3} \]

\( R^{6g} = 0.55 \), average radiation pattern

\( FS = 2 \), free-surface amplification

\( V = \frac{1}{\sqrt{2}} \approx 0.7071 \), partition factor

\( \rho_0 = 2.8 \text{ gm/cm}^3 \), crustal density

\( \beta_0 = 3.6 \text{ km/sec} \), crustal shear wave velocity

\( \zeta \) = weighting parameter
\( f_A, f_B \) = corner frequencies (Hz)

\[
\log \zeta = 2.52 - 0.637 M_w
\]

\[
\log f_A = 2.41 - 0.533 M_w
\]

\[
\log f_B = 1.43 - 0.188 M_w
\]

\( M_0 = 10^{1.5 M_w + 16.05} \) (dyne-cm)

\[
D(R_h, f) = D_g(R_h) \cdot D_m(R_h, f) \quad \text{diminution factor} \quad (2.6)
\]

in which,

\[
D_g(R_h) = \begin{cases} 
1 / R_h & \text{if } R_h \leq 70 \text{ km} \\
1 / 70 & \text{if } 70 \leq R_h \leq 130 \text{ km} \\
(1 / 70) \cdot (130 / R_h)^{0.5} & \text{if } R_h \geq 130 \text{ km}
\end{cases}
\]

\[
D_m(R_h, f) = e^{-\frac{\pi f R_h}{2Q(f)}} \quad \text{geometric attenuation}
\]

and

\[
Q(f) = 680 \cdot f^{0.36} \quad \text{anelastic material attenuation}
\]

\[
P(f) = \frac{1}{\sqrt{1 + (f / f_{\text{max}})^8}} \quad \text{high-cut filter} \quad (2.7)
\]

where \( f_{\text{max}} = 50 \text{ Hz} \) is used.

\[
I(f) = \frac{1}{(2\pi f)^p} \quad \text{response indicator} \quad (2.8)
\]

where \( p = 0 \) for acceleration, 1 for velocity, and 2 for displacement.

Total duration \( T_d \) (sec) for the strong-motion phase is decomposed into source and path durations:

\[
T_d = T_0 + T_p \quad (2.9)
\]

where source duration \( T_0 \) (sec) is related to corner frequency \( f_A \) following Boatwright and Choy (1992):

\[
T_0 = \frac{1}{2 \cdot f_A} \quad \text{(sec)} \quad (2.10)
\]
and the path duration $T_p$ (sec) is of a tri-linear form following Atkinson and Boore (1995):

$$
T_p = \begin{cases} 
0 & \text{if } R_h \leq 10 \text{ km} \\
0.16 \cdot (R_h - 10) & \text{if } 10 \leq R_h \leq 70 \text{ km} \\
9.6 - 0.03 \cdot (R_h - 70) & \text{if } 70 \leq R_h \leq 130 \text{ km} \\
7.8 + 0.04 \cdot (R_h - 130) & \text{if } 130 \leq R_h \leq 1000 \text{ km}
\end{cases}
$$

(2.11)

To incorporate intensity evolution, an exponential window function (Saragoni and Hart, 1974) is introduced to obtain more realistic accelerograms:

$$
\psi(t) = a \cdot t^b \cdot e^{-ct}
$$

(2.12)

and shape parameters $a$, $b$ and $c$ are defined as:

$$
a = \left( \frac{e}{\varepsilon \cdot T_w} \right)^b
$$

$$
b = -\frac{\varepsilon \cdot \ln \eta}{1 + \varepsilon \cdot (\ln \varepsilon - 1)}
$$

$$
c = \frac{b}{\varepsilon \cdot T_w}
$$

(2.13)

where $\varepsilon$ defines the fraction of a specified duration at which the maximum envelope amplitude will occur; $\eta$ defines the fraction of the maximum envelope amplitude which is reached at time $T_w$ (Figure 2.6). According to Boore (1983), $\varepsilon = 0.2$, and $\eta = 0.05$ was found consistent with the envelope function to 22 accelerograms in Saragoni and Hart (1974) and $T_w = 2T_d$ gives a record with a strong phase close to $T_d$. Therefore, they are used in all applications in this study for point sources.

2.5.2 Comparisons of Point Source Model with Broadband Model

The accuracy and validity of the point-source model can be found in the literature (Boore 1996, Atkinson and Boore 1998). There have been no records of moderate to large events near any of the three cities that can be used for comparison with the simulated ground motions. The closest is the simulation results based on the broadband
approach for St. Louis due to events of magnitude 6.5 to 7.5 in the NMSZ by Saikia and Somerville (1997). The broadband approach considers the details of the geometry of the fault, rupture surface, and wave propagation for low frequency motion and uses records for high frequency motion. Large-scale simulations, which require detailed information of each fault that is generally unknown, are fairly computationally expensive. The results of these two methods for a rock site at St. Louis are compared. An event based on the point-source model is chosen such that source and path parameters are comparable to those of the magnitude-7 event studied by Saikia and Somerville. The response spectra of hard rock motions based on these two models are compared in Figure 2.7. It is seen that agreements are fairly good in the period range from 0.2 sec to 3 sec.

2.5.3 Finite Fault Model

This so-called finite fault model emphasizes the effects of a large fault dimension, including rupture propagation, directivity, and source-receiver geometry, which can have significant influence on amplitudes, frequency content and duration of ground motion. The examples of applications in Beresnev and Atkinson (1997, 1998b) showed that the model produces ground motions that match well field records on rock sites in 4 earthquakes. They are the M_w-8.0 1985/9/19 Michoacan (Mexico), the M_w-8.0 1985/3/3 Valparaiso (Chile), the M_w-5.8 1988/11/25 Saguenay (Québec), and the M_w-6.7 1994/1/17 Northridge earthquakes. For soil sites (Beresnev and Atkinson, 1998c), however, their model tends to overestimate the ground motions due to soil nonlinearity, which is not considered. The application of this recently proposed model is still limited to simulation of a single strong ground motion without considering surface waves and spatial variation: therefore, applications in dynamic analyses for bridges on multiple supports are not possible at present. Despite these disadvantages, this study adopts the Beresnev-Atkinson model for the simulation task in the CEUS, because

1. The Beresnev-Atkinson model is shown to provide an unbiased fit to 11 earthquake events in the eastern North America (ENA) earthquakes, the magnitudes of which range from 4.0 to 7.3 (Beresnev and Atkinson, 1999).
2. Unlike wave propagation based modeling procedures (Saikia and Somerville, 1997), the Beresvnev-Atkinson model requires much less computational efforts, which allows generation of a large number of ground motions in Monte Carlo simulation.

3. The Beresnev-Atkinson model requires less faulting parameter inputs, which is suitable for areas with scarce or no seismotectonic data, such as in CEUS.


5. The Beresnev-Atkinson model can be applied to simulate two-dimensional strong ground motions if motions are decomposed onto two principal directions according to Penzien and Watabe (1976).

It is also possible to simulate the vertical component in the ENA by using the H/V spectral ratio proposed by Atkinson (1993). In addition, spectral ratio in areas of interest may be also available in the literature, e.g. NMSZ. A 3-component simulation, however, is not attempted in this study since phase delay in the stochastic model is of random nature and phase delay due to different arrival times of seismic waves is not accounted for explicitly as in the wave propagation model.

Figure 2.8 shows the geometry of the finite fault model proposed by Beresnev and Atkinson (1997, 1998a). A rupture plane of 140 km (along-strike) by 33 km (down-dip) (Johnston 1996) with a vertical strike-slip faulting and 34.69° azimuth is assumed, which is an approximate estimate based on USGS OFR-96-532. The distance from the ground surface to the upper edge of the rupture plane is assumed to be 5 km. For reference, 0.6 km is used in Saikia and Somerville (1997) and 10 km is used in USGS OFR-96-532.

The fault plane is divided into 64 (16x4) sub-faults. Each sub-fault is then treated as a one-corner point source (Brune 1970, 1971, Frankel et al. 1996) and may be “triggered” a few times during an earthquake event. The delay between triggers is of random nature depending on sub-fault rise time $T_{rise}$ to simulate the complexity in slip process. The resulting ground motion is therefore a combination of waveforms from different sub-faults accounting for differences in arrival times and path attenuation. The arrival time
delay between sub-faults is accounted for by the rupture propagation from the hypocenter
\((i_0,j_0)\) to the center of the sub-fault \(F\), and the travel time from the center of the sub-fault
\(F\) to the observation point \(P\).

The mathematical form for the Fourier amplitude spectrum of each sub-fault follows basically Equation (2.4). Source spectrum \(E(\Delta m_0, f)\) for each sub-fault is modeled as a single-corner form:

\[
E(\Delta m_0, f) = C \cdot (2\pi)^2 \cdot \Delta m_0 \cdot \left\{ \frac{1}{1 + \left( \frac{f}{f_c} \right)^2} \right\}^{2}
\]

(2.14)

where \(\Delta m_0 = \Delta \sigma \cdot \left( \frac{\Delta \ell + \Delta w}{2} \right)^3\) indicating sub-fault seismic moment (dyne-cm) released in one trigger; \(\Delta \sigma\) is the Kanamori-Anderson (1975) stress parameter (bars) and is related to the stress drop of the sub-fault in one trigger; \(\Delta \ell\) and \(\Delta w\) are sub-fault length and width, respectively (Figure 2.8). The corner frequency \(f_c\) (Hz) can be expressed as

\[
f_c = \frac{yz}{\pi} \frac{\beta_0}{\frac{1}{2} (\Delta \ell + \Delta w)}
\]

(2.15)

in which \(z = 1.68\), a calibration constant related to maximum slip rate; \(\beta_0\) is shear wave velocity of the Earth crust (km/sec); \(y\) is the fraction of rupture-propagation velocity to \(\beta_0\), which ranges from 0.6 to 1.0. In this study, we assume \(y = 0.8\) and anelastic material attenuation \(Q(f) = 670*f^{0.33}\), following Beresnev and Atkinson (1997, 1999). Geometric attenuation and high-cut filter take the same form as two-corner point source model. The total duration \(T_d\) of each sub-fault consists of sub-fault rise time \(T_{rise}\) and path duration \(T_p\):

\[
T_d = T_{rise} + T_p
\]

(2.16)

where sub-fault path duration \(T_p\) depends on the distance from the center of sub-fault to the observation point and takes the same form as two-corner model;
\[ T_{\text{rise}} = \frac{1}{2} \cdot \frac{(\Delta \ell + \Delta w)}{V_{\text{rup}}} \]  

(2.17)

in which rupture velocity \( V_{\text{rup}} = 0.8 \beta_0 \). The delay between two consecutive triggers of a sub-fault is modeled as \((1 + u) \cdot T_{\text{rise}}\) in which \( u \) is a random number uniformly distributed between 0 and 1. The Saragoni-Hart (1974) exponential window for each sub-fault is padded with rapid sinusoidal taper zones as shown in Figure 2.9.

The sub-fault spectrum is basically a functional of stress drop and sub-fault size. To make the resulting ground motion insensitive to stress parameter and sub-fault size, Beresnev and Atkinson suggested a stress parameter of 50 bars. To determine sub-fault size, Beresnev and Atkinson (1999) provide a generic formula calibrated according to earthquake records in the ENA. In this study, 64 sub-faults, each with a stress parameter of 200 bars, are used in the finite fault model. The model gives results in good agreements with the USGS OFR-96-532 target spectral accelerations at 0.2-, 0.3- and 1.0-sec structural periods. A stress parameter of 150 bars is also tested for the finite fault model. As expected, it causes lower spectral values in the low frequency range in the 2% in 50 years hazard level.

Before the Beresnev-Atkinson model can be applied in Monte Carlo simulation, the slip distribution (asperity) needs to be included in the model. Since there are no records of large events in the NMSZ, observations based on California earthquakes are used. The slip distribution within the rupture surface is modeled as a correlated random field according to Saikia and Sommerville (1997), and Sommerville et al (1999), using the following wave number spectrum:

\[ \mathcal{W}(k_x, k_y) = \frac{1}{\sqrt{1 + [(k_x / C_x)^2 + (k_y / C_y)^2]^n}} \]  

(2.18)
In which $k_x$ and $k_y$ are wave numbers in the along-strike and down-dip directions respectively; $n = 2.0$ and $C_x$ and $C_y$ are correlation length constants depending on magnitude:

\[
\begin{align*}
\log_{10} C_x &= 1.72 - 0.5M_w \\
\log_{10} C_y &= 1.93 - 0.5M_w
\end{align*}
\] (2.19)

This random field model is then modulated to ensure larger slip in the middle than at the edge of the rupture surface, a feature observed in recent earthquakes. Two sample slip distributions on the rupture surfaces (33 km×140 km) based on the correlated random field model and their corresponding discretizations for the finite fault model are shown in Figures 2.10-11. It should be noted that the finite fault model uses slip values as weights to distribute the target seismic moment over the rupture plane. Therefore, only relative slip values are important in the simulation procedure. In the simulation procedure, the intensity of seismic motion of an individual sub-fault is determined by its distributed seismic moment. The distributed moment is also used to determine the number of triggers in its corresponding sub-fault.

2.5.4 Uncertainty in Attenuation

Path attenuation due to geometric radiation and material damping in the earth crust is accounted for by the semi-empirical formulae shown above (Eqs 2.6 to 2.8). The uncertainty in the attenuation is modeled by a truncated lognormal distribution with the median value given by the above diminution factor as a function of distance and frequency (Eq. 2.6). A value of 0.75 is used for the natural logarithms of the standard deviation of PGA following USGS OFR-96-532. To prevent unrealistic large variation, cut-off limits of mean plus and minus three standard deviations are used. The resulting peak ground acceleration ($PGA_{simulation}$) is then given by,

\[
\ln PGA_{simulation} = \ln PGA_{median} + \varepsilon \cdot \zeta
\] (2.20)

where $\zeta = 0.75$ and $\varepsilon$ (epsilon) is a standard normal random variable, indicating the deviation from median attenuation. The coefficient of variation $\delta = 0.87$ from
\[ \zeta = \sqrt{\ln(1 + \delta^2)}. \] Since peak ground accelerations can be modeled by a lognormal distribution, \( \ln \text{PGA} \) in Equation (2.20) is of normal distribution and \( \varepsilon \) can be modeled by a truncated normal density function (Appendix A).

### 2.6 Local Site Effect

Soil amplification due to the local site soil profile is considered. In general, nonlinear soil properties are needed for an accurate estimate of soil amplification, which is also earthquake intensity dependent. Detailed information on soil profile is usually hard to obtain especially for a deep soil column. In view of this, soil is treated approximately as an elastic medium, which generally gives an overestimate of the amplification in the high frequency range and an underestimate in the long period range for a severe event. The quarter wavelength method (QWM) (Joyner et al., 1981, Boore and Joyner, 1991, Boore and Joyner, 1997, Boore and Brown, 1998) is used to model the soil amplification. Soil profiles are based on boring log data in Memphis, Carbondale and St. Louis (Hashash 1999, Herrmann 1999). In the QWM, the total amplification is expressed as:

\[ \text{Amp}(f) = \mathcal{A}(f) \cdot \mathcal{P}(f) \]  
\[ (2.21) \]

where \( \mathcal{A}(f) \) is the amplification function and \( \mathcal{P}(f) \) is the attenuation function. The amplification is approximated by

\[ \mathcal{A}(f) = \frac{\rho_0 \cdot \beta_0}{\bar{\rho}_s(f) \cdot \beta_s(f)} \]  
\[ (2.22) \]

in which \( \rho \) and \( \beta \) are the density (g/cm\(^3\)) and shear wave velocity (m/sec) at the source (subscript 0) and site (subscript s); \( f \) is frequency (Hz); over-bar indicates average value. At the site, the frequency-dependent effective velocity \( \bar{\beta}_s(f) \) is defined as the average shear wave velocity from the surface to a depth of quarter wavelength for a given frequency. If the travel time to the depth of a quarter wavelength, \( t_{\frac{1}{4}}(f) \), is defined as:
Then the depth of a quarter wavelength $z$ can be determined by solving the following equations:

\[
\begin{align*}
\frac{tt_z(f)}{4f} &= \sum_{i=1}^{m} \frac{h^{(i)}}{V_s^{(i)}} \\
z &= \sum_{i=1}^{m} h^{(i)}
\end{align*}
\]  

(2.24)

where $h^{(i)}$ is the thickness of the $i^{th}$ layer (m); $V_s^{(i)}$ is the shear wave velocity of the $i^{th}$ layer (m/sec); $m$ is the number of layers to satisfy the equality relation. The effective velocity $\overline{\beta}_s(f)$ at a given frequency $f$ is therefore determined by

\[
\overline{\beta}_s(f) = \frac{z}{tt_z(f)}
\]

(2.25)

A travel-time-weighted average is taken of the density $\overline{\rho}_s$:

\[
\overline{\rho}_s(f) = \frac{1}{tt_z(f)} \left[ \sum_{i=1}^{m} tt_z^{(i)}(f) \cdot \rho_s^{(i)} \right]
\]

(2.26)

where $tt_z^{(i)}(f) = \frac{h^{(i)}}{V_s^{(i)}}$, travel time of shear wave in the $i^{th}$ layer. At high frequency, the soil attenuation is accounted for by

\[
\mathcal{P}(f) = e^{-\kappa_0 f}
\]

(2.27)

in which $\kappa_0$ is a term that accounts for shear velocity and damping over the soil column:

\[
\kappa_0 = \sum_{i=1}^{N} \left[ \int_{0}^{h^{(i)}} \frac{dz}{V_s^{(i)} \cdot Q(h)} \right]
\]

(2.28)
where \( N \) is the number of soil layers; \( h \) is the depth measured from the ground surface (m); and \( Q \) is the quality factor for a frequency-independent measure of damping. For small material damping, \( Q \) is related to the damping ratio \( \xi_d \) as follows:

\[
Q = \frac{1}{2 \xi_d} \quad \text{for} \quad Q^{-1} << 1
\]  

(2.29)

It should be noted that \( Q \) is only a simplified phenomenological description of a complex process involving an intrinsic and scattering attenuation mechanism. For soil, it is assumed that \( Q = 6h^{0.24} \) (Herrmann and Akinci, 1999) and \( Q = 500 \) for rock.

The “representative” profiles for the three cities used in this study are shown in Table 2.3 to 2.5. According to QWM, \( \kappa_0 = 0.063 \) sec, 0.043 sec and 0.0076 sec for Memphis, Carbondale and St. Louis, respectively. The calculated \( \kappa_0 \) value for Memphis agrees with Akinci and Herrmann (1999) corresponding to a soil deposit of 1000 m. The resultant soil amplifications as a function of frequency for the three cities are shown in thick lines in Figures 2.12-14. It is seen that while amplification at St. Louis is restricted to the high frequency range (> 3 Hz) due to the very shallow soil layer on hard rock, the much deeper soil layers in Memphis and Carbondale produce much larger amplification in the longer period range. For comparison, the soil amplification at these three cities based on the well-known computer software SHAKE is also shown for bedrock ground motions of 10 % and 2 % probability of exceedance in 50 years. The soil amplification factor used in the 1997 NEHRP provisions for soil classification of B/C boundary is also shown since this is the soil condition used for USGS national earthquake hazard maps.

Note that the quarter-wavelength model will give a good estimate of the averaged amplification and will miss the peaks and valleys. Also, it has a tendency of overestimating the amplification in the high frequency range for ground motion of very high intensity when effects of soil nonlinearity may be significant. In spite of these limitations, the agreements are generally good.

Soil property is a 3-dimensional random field and depends on earthquake intensity. It varies widely within a given city. For instance, the soil deposit in Memphis City varies
from a thickness of 800 m with alluvium surface layer (east of Memphis) to a thickness of 1000 m with loess surface layer (west/downtown of Memphis) according to compiled data from Dorman and Smalley (1994), and Hwang et al. (1999). To give a general understanding of the variation in Memphis, soil amplification for the soil profile at Pier C of the Hernando Desoto Bridge located at Mud Island (35.1529°N, 90.0579°W) is shown in Figure 2.15 for comparison with the representative soil amplification. Also shown are soil amplifications from Herrmann et al. (1999), and Boore and Joyner (1991, 1997).

In passing, it is pointed out that soil amplification considering nonlinear effects is an extremely complex problem. SHAKE is the most widely used program but is based on an equivalent linear wave propagation method. It is expected to yield good results when the excitation intensity is not very high and the soil layer is not very deep. It is therefore expected to work better for the St. Louis soil profile than those of Memphis and Carbondale. There are other truly nonlinear programs available for this purpose. However, they have not been commonly accepted by engineers, therefore are not used for comparison in this study.

It is of interest to compare the proposed method of simulation with the broadband simulation method by Somerville et al. Table 2.7 shows the attributes of the two simulation methods.

### 2.7 Baseline Correction

Physics dictates that all earthquake ground motions should start and end with zero velocity with however a possible permanent offset of the displacement during a severe event. Due to instrumental difficulties, however, raw ground motion records usually do not satisfy this requirement and therefore need baseline correction. In the case of the stochastic simulation, undesirable nonzero conditions often occur in the simulated ground motions; therefore, the baseline correction procedure is also needed. To do this, this study basically follows guidelines from US Geological Survey BAP routine (Converse 1992). The accelerogram is first padded with leading and trailing zeros, then passed
through Ormsby filter, and then de-trended by a linear or parabolic function, depending on which one results in close-to-zero end velocity and displacement. The Ormsby filter $H(f)$ takes the following mathematical form:

$$H(f) = \begin{cases} 
0 & 0 \leq f \leq f_1 \text{ Hz} \\
\sin^2\left(\frac{\pi f - f_1}{2 f_2 - f_1}\right) & f_1 \leq f \leq f_2 \text{ Hz} \\
1 & f_2 \leq f \leq f_3 \text{ Hz} \\
\sin^2\left(\frac{\pi f - f_3}{2 f_4 - f_3}\right) & f_3 \leq f \leq f_4 \text{ Hz} \\
0 & f \geq f_4 \text{ Hz}
\end{cases} \quad (2.30)$$

where $f_1 = 0.1 \text{ Hz}$; $f_2 = 0.25 \text{ Hz}$; $f_3 = 24.5 \text{ Hz}$; $f_4 = 25.5 \text{ Hz}$. For $M_{w}-8$ earthquakes, a parabolic equation (Brady 1966, Nigam et al. 1968) is adopted:

$$A^\star(t) = A(t) - C_0 - C_1 t - C_2 t^2$$
$$V^\star(t) = V(t) - \frac{1}{2} C_1 t^2 - \frac{1}{3} C_2 t^3 \quad (2.31)$$

where $A(t)$ is the uncorrected acceleration time history; $A^\star(t)$ is the corrected acceleration time history; $V(t)$ is the uncorrected velocity time history; $V^\star(t)$ is the corrected velocity time history; $C_0$, $C_1$, $C_2$ are regression coefficients to minimize the mean square of the corrected velocity $V^\star(t)$ and will satisfy the following simultaneous equations

$$\frac{\partial}{\partial C_i} \int_0^T [V^\star(t)]^2 dt = 0 \quad ; \quad i = 0, 1, 2 \quad (2.32)$$

For earthquakes smaller than $M_{w}-8$, it is observed that a linear function will give satisfactory results:

$$\begin{cases} 
C_0 = \overline{A(t)} - C_1 \overline{t} \\
C_1 = \frac{\sum (t_i - \overline{t})(A(t_i) - \overline{A(t)})}{\sum (t_i - \overline{t})^2} \\
C_2 = 0
\end{cases} \quad (2.33)$$

where $\sum = \sum_{i=1}^n$ ; $n$ is the number of data points. To integrate acceleration over time into velocity and displacement, the Newmark $\beta$ method with linear variation of
acceleration is adopted according to Berg and Housner (1961). Two sample acceleration time histories after baseline correction are shown in Figures 2.16 and 2.17.

### 2.8 Directivity Effects of Magnitude-8 Events

The near-source effects using the finite fault model are demonstrated by a comparison of the ground motions at Memphis (Figure 2.18(a)(&(b))) due to two simulated events, both of magnitude 8. The location, size, and orientation of the faults are the same with a closest distance of 62 km from the fault surface to the site. The epicenters, however, are located such that one is far (186 km) from the site with the rupture propagating toward the site and the other closer (79 km) to the site with the rupture propagating generally away from the site (Figures 2.10 and 2.11). Sample rupture surface (33 km×140 km) slip distributions based on the correlated random field model and the corresponding discretizations for the finite fault model for the two events are shown in the figures. The time histories of the ground motion for a soil site at Memphis are shown in Figure 2.18(a) and (b) for both events. The directivity effects can be clearly seen that Figure 2.18(a) produces a shorter but more intense ground motion due to the “Doppler effect” even though the epicenter is much farther away. One accelerogram simulated for Carbondale, IL due to the same fault rupture in Figure 2.11 is also shown (Figure 2.18(c)) to further demonstrate the difference in ground motions (between (a) and (c)) due to the same event but from two observation points.

### 2.9 Final Remarks

Simulation of strong ground motions in Mid-America is an ongoing research problem, particularly in view of the controversy on interpretation of the recent seismotectonic observations in the NMSZ (Newman et al. 1999, Mueller et al. 1999). Definitive conclusions may not be reached for some time to come. In this study, for purpose of performance-based structural analyses, a simulation method is developed
following closely the guidelines of USGS OFR-96-532. As more information becomes available, the procedure can be modified with minor efforts. The ground motion models used in this study can account for effects of finite fault dimension and have been shown to produce results which are in good agreement with broadband simulation method in the frequency range of engineering importance. In the following chapters, these ground motions will be used for statistical study of structural responses.
Table 2.1  Focal depth distribution for point source model (Wheeler and Johnston, 1992).

<table>
<thead>
<tr>
<th>Depth (km)</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>1~5</td>
<td>0.250</td>
</tr>
<tr>
<td>5~10</td>
<td>0.500</td>
</tr>
<tr>
<td>10~15</td>
<td>0.050</td>
</tr>
<tr>
<td>15~20</td>
<td>0.050</td>
</tr>
<tr>
<td>20~25</td>
<td>0.015</td>
</tr>
<tr>
<td>25~30</td>
<td>0.135</td>
</tr>
</tbody>
</table>

Table 2.2  Focal depth distribution for finite fault model (modified from EPRI, 1993).

<table>
<thead>
<tr>
<th>Depth, km (mean)</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.00 ~ 13.25 (9.10)</td>
<td>0.40</td>
</tr>
<tr>
<td>13.25 ~ 21.50 (17.40)</td>
<td>0.40</td>
</tr>
<tr>
<td>21.50 ~ 29.75 (25.60)</td>
<td>0.15</td>
</tr>
<tr>
<td>29.75 ~ 38.00 (33.90)</td>
<td>0.05</td>
</tr>
</tbody>
</table>
Table 2.3  Representative soil profile of Memphis, TN (after Hashash, 1999).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil column</th>
<th>Thickness (m)</th>
<th>Vs (m/sec)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Alluvium</td>
<td>7.2</td>
<td>360</td>
<td>1.92</td>
</tr>
<tr>
<td>2</td>
<td>Alluvium</td>
<td>4.8</td>
<td>360</td>
<td>2.00</td>
</tr>
<tr>
<td>3</td>
<td>Alluvium</td>
<td>14.9</td>
<td>360</td>
<td>2.08</td>
</tr>
<tr>
<td>4</td>
<td>Loess</td>
<td>9.0</td>
<td>360</td>
<td>2.16</td>
</tr>
<tr>
<td>5</td>
<td>Fluvial Deposits</td>
<td>7.9</td>
<td>360</td>
<td>1.98</td>
</tr>
<tr>
<td>6</td>
<td>Jackson Formation</td>
<td>47.3</td>
<td>520</td>
<td>2.08</td>
</tr>
<tr>
<td>7</td>
<td>Memphis Sand</td>
<td>245.6</td>
<td>667</td>
<td>2.30</td>
</tr>
<tr>
<td>8</td>
<td>Wilex Group</td>
<td>83.3</td>
<td>733</td>
<td>2.40</td>
</tr>
<tr>
<td>9</td>
<td>Midway Group</td>
<td>580</td>
<td>820</td>
<td>2.50</td>
</tr>
<tr>
<td>10</td>
<td>Bed Rock</td>
<td>Half-Space</td>
<td>3600</td>
<td>2.80</td>
</tr>
</tbody>
</table>

Table 2.4  Representative soil profile of Carbondale, IL (after Hashash, 1999).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil column</th>
<th>Thickness (m)</th>
<th>Vs (m/sec)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cahokia Alluvium</td>
<td>10.4</td>
<td>140</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>Henry Formation</td>
<td>10.0</td>
<td>250</td>
<td>2.1</td>
</tr>
<tr>
<td>3</td>
<td>Henry Formation</td>
<td>25.6</td>
<td>270</td>
<td>2.1</td>
</tr>
<tr>
<td>4</td>
<td>Mississippi Embayment</td>
<td>119.0</td>
<td>280</td>
<td>2.3</td>
</tr>
<tr>
<td>5</td>
<td>Pennsylvanian Limestone</td>
<td>835.0</td>
<td>2900</td>
<td>2.6</td>
</tr>
<tr>
<td>6</td>
<td>Bed Rock</td>
<td>Half-Space</td>
<td>3600</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Table 2.5  Representative soil profile of St. Louis, MO (after Hashash, 1999 and Herrmann, 1999).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil column</th>
<th>Thickness (m)</th>
<th>Vs (m/sec)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Modified Loess</td>
<td>5.7</td>
<td>185</td>
<td>1.9</td>
</tr>
<tr>
<td>2</td>
<td>Glacio-Fluvial</td>
<td>10.0</td>
<td>310</td>
<td>2.1</td>
</tr>
<tr>
<td>3</td>
<td>Mississippian Limestone</td>
<td>984.3</td>
<td>2900</td>
<td>2.6</td>
</tr>
<tr>
<td>4</td>
<td>Bed Rock</td>
<td>Half-Space</td>
<td>3600</td>
<td>2.8</td>
</tr>
</tbody>
</table>
Table 2.6  Soil profile at Pier C of the Hernando Desoto Bridge located at Mud Island, Memphis, TN (after Hwang et al., 1999).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil column</th>
<th>Thickness (m)</th>
<th>Vs (m/sec)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Alluvial Deposits</td>
<td>13.94</td>
<td>170</td>
<td>1.87</td>
</tr>
<tr>
<td>2</td>
<td>Alluvial Deposits</td>
<td>15.45</td>
<td>210</td>
<td>1.79</td>
</tr>
<tr>
<td>3</td>
<td>Jackson Formation</td>
<td>21.52</td>
<td>330</td>
<td>1.91</td>
</tr>
<tr>
<td>4</td>
<td>Memphis Sand</td>
<td>132.09</td>
<td>425</td>
<td>2.00</td>
</tr>
<tr>
<td>5</td>
<td>Memphis Sand</td>
<td>120.00</td>
<td>600</td>
<td>2.12</td>
</tr>
<tr>
<td>6</td>
<td>Flour Island Formation</td>
<td>91.00</td>
<td>708</td>
<td>2.18</td>
</tr>
<tr>
<td>7</td>
<td>Fort Pillow Sand</td>
<td>157.00</td>
<td>700</td>
<td>2.18</td>
</tr>
<tr>
<td>8</td>
<td>Porters Creek Clay</td>
<td>141.00</td>
<td>789</td>
<td>2.22</td>
</tr>
<tr>
<td>9</td>
<td>McNairy Sand</td>
<td>308.00</td>
<td>1050</td>
<td>2.31</td>
</tr>
<tr>
<td>10</td>
<td>Knox Dolomite</td>
<td>Half-Space</td>
<td>3500</td>
<td>2.70</td>
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</tbody>
</table>
Table 2.7  Comparison of the proposed simulation method with broadband simulation.

<table>
<thead>
<tr>
<th>Methods</th>
<th>Broadband Simulation (Sommerville et al. 1997, Saikia &amp; Sommerville 1997)</th>
<th>Stochastic Model (this study)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Source Model</td>
<td>finite fault</td>
<td>finite fault (near-field)</td>
</tr>
<tr>
<td>Local Site Effect</td>
<td>equivalent linear method</td>
<td>quarter-wavelength method</td>
</tr>
<tr>
<td>Theoretical Background</td>
<td>hybrid: field records (short period) wave propagation (long period)</td>
<td>spectral representation based on field observations</td>
</tr>
<tr>
<td>Computational Effort</td>
<td>extensive</td>
<td>not excessive</td>
</tr>
<tr>
<td>Scaling Factor</td>
<td>0.27 ~ 10.75</td>
<td>0.41 ~ 4.48</td>
</tr>
<tr>
<td>Median Response Estimate</td>
<td>10% ~ 25%</td>
<td>10% ~ 15%</td>
</tr>
<tr>
<td>Comments</td>
<td>theoretically rigorous basis, but not easy for practicing engineers to generate ground motions</td>
<td>easy for engineers to use</td>
</tr>
</tbody>
</table>
Initiate the $n^{th}$ simulation

Determine no. of earthquakes in the $n^{th}$ simulation

Generate random numbers for each earthquake sample function e.g. magnitude, focus (epicentral distance $R_e$, focal depth), attenuation uncertainty, and slip distribution (finite-fault), etc.

- far-field 2-corner point source empirical model (Boore, 1996)
- soil amplification due to local site effects by quarter wave-length method (Joyner et al. 1981)
- near-field ground motion using finite fault model (Beresnev and Atkinson 1997, 1998)

Baseline correction (USGS BAP or Brady 1966)

Sample size is large enough?

Yes

END

No

$n = n^{th}$ simulation is initiated

Figure 2.1  Ground motion simulation flowchart for Mid-America cities.
Seismicity surrounding Memphis TN, Carbondale IL, and St. Louis MO

Figure 2.2 Annual occurrence rate per 0.1×0.1 degree square of earthquakes of $m_s$ 5 or larger.
Figure 2.3  Seismic zone of $M_w$-8 earthquakes.
Figure 2.4 Epicenters and magnitudes of earthquakes of simulated 6000-year record.
Figure 2.5 $M_{\text{max}}$ zones used in the Central and Eastern United States. Numerical values are moment magnitude (after USGS OFR-96-532).
Figure 2.6 Envelope function used in point source model. The meaning of parameters is shown here. Note that the abscissa has been normalized by a quantity proportional to the duration of the motion ($T_w = 2 \cdot T_d$).
Figure 2.7. Comparison of response spectra based on point-source model with Saikia and Somerville’s broadband model (1997).
O Origin
δ₁ fault dip
φ₁ fault strike
φ₂ azimuth to observation point
d depth to fault upper edge
F center of subfault
Δw subfault width
Δl subfault length
i, j subfault number
r₁ distance from subfault to observation point

\[
\overrightarrow{OP} = \{ R \cdot \cos(\phi₂ - \phi₁), R \cdot \sin(\phi₂ - \phi₁), -d \} \\
\overrightarrow{OF} = \{(2i - 1)\Delta l / 2, (2j - 1)(\Delta w / 2)\sin \delta, (2j - 1)(\Delta w / 2)\cos \delta \} \\
\overrightarrow{r₁} = \overrightarrow{OP} - \overrightarrow{OF} \\
r₁ = \left[ R \cdot \cos(\phi₂ - \phi₁) - (2i - 1)\Delta l / 2 \right]^2 + \left[ R \cdot \sin(\phi₂ - \phi₁) - (2j - 1)(\Delta w / 2)\sin \delta \right]^2 + \left[ d + (2j - 1)(\Delta w / 2)\cos \delta \right]^2 \right]^{1/2}
\]

Figure 2.8 Beresnev-Atkinson finite fault geometry.
Figure 2.9 Envelope function used in subfaults of finite fault model. The meaning of parameters is shown here. The abscissa has been normalized by a quantity proportional to the duration of the subevent \(T_d\) in Equation 2.16. Note that the scale of the leading and trailing taper zones is exaggerated.
Figure 2.10  Sample of random slip distribution based on correlated random field model and discretization for finite fault model ($M_w = 8, h = 25.6 \text{ km}$).
Figure 2.11 Sample of simulated slip distribution based on correlated random field model and discretization for finite fault model ($M_w = 8, h = 33.9 \text{ km}$).
Figure 2.12 Soil amplification factor for representative soil profile, Memphis, TN.

Figure 2.13 Soil amplification factor for representative soil profile, Carbondale, IL.
Figure 2.14 Soil amplification factor for representative soil profile, St. Louis, MO.

Figure 2.15 Comparison of various soil amplifications in Memphis, TN.
Figure 2.16 Baseline corrected sample 10% in 50 years ground motion for representative soil profile, Memphis, TN.
Figure 2.17 Baseline corrected sample 2% in 50 years ground motion for representative soil profile, Memphis, TN.
Figure 2.18 Near-source "Doppler" effects of simulated acceleration time histories in (a) Memphis, TN, (b) Memphis, TN and (c) Carbondale, IL by the finite fault model.
CHAPTER 3
METHOD OF SIMULATION – WESTERN UNITED STATES

3.1 Overview

For simulation of earthquake ground motions in Western United States, a different procedure is used in view of much larger number of earthquake records including those of near-source events and much better defined seismic zones. To simulate future seismic events and ground motions, the procedure proposed by Collins, Wen and Foutch (1995) is used again with some modifications. The ground motion model basically follows the empirical Fourier spectrum proposed by Trifunac (1994), and then a modification factor is incorporated to account for some of the important near-field effects based on data (Sommerville et al. 1997). The tectonic and seismological data are taken from the 1995 WGCEP report (Working Group on California Earthquake Probabilities). Possible future events are generated for a period of 10 years. The site is at Santa Barbara, California. A total of 1000 simulations of 10-year period are carried out for to construct uniform hazard response spectra. The simulation procedure is shown in Figure 3.1 and details of the simulation method are given in the following.

3.2 Selection of Location and Soil Profile

The site location is (34.42°N, 119.70°W) in Santa Barbara, California. The average shear wave velocity is shown to be around 406 m/sec according to the average shear wave velocity map Park and Elrick (1998) generated for the uppermost 30 m soil profile of Southern California using surface geology. We therefore assume a local site condition of very dense soil and soft rock corresponding to the 1997 NEHRP Type C site condition ($V_S = 360 \sim 760$ m/sec) or Site Class B according to Boore et al. (1993), i.e. rock with $V_S$
= 360 ~ 750 m/sec. The same site condition was used in Collins et al. (1995) for Los Angeles. This facilitates the comparison of seismic hazard obtained in this study with Collins et al. (1995) and USGS (1997).

### 3.3 Reference Area, Seismicity and Tectonics

Considering the relatively faster attenuation in California than in the Mid-America, the effective zone is defined as a circular area of a radius of 150 km centered in Santa Barbara. The choice of 150 km is based on Collins et al. (1995), USGS deaggregated seismic hazard data (1997) and USGS OFR-96-532 (Frankel el al. 1996). In the effective zone of Santa Barbara (Figure 3.2(b)), there are 29 seismic zones contributing to the seismic hazard at the site. The major fault locations are listed in Table 3.1 and plotted in Figure 3.2 (a). The seismological data are taken from the 1995 Working Group Report on California Earthquake Probabilities (WGCEP). According to the available geodetic, geologic and seismic information, the seismic zones are categorized into three types of seismotectonic zones. Type A zones contain faults for which paleoseismic data suffice to model characteristic earthquakes as time dependent events; i.e. a renewal model is used. Type B zones contain faults with insufficient data, so the characteristic earthquakes in Type B zones are modeled as a Poisson process. Types C zones contain diverse or hidden faults and therefore there are no characteristic earthquakes. The detailed information, including source faulting mechanism and seismicity rate, is given in Table 3.2.

### 3.4 Occurrence and Source Models

To facilitate the proposed simulation procedure, the reference area is further discretized into 10 km×10 km cells as shown in Figure 3.3. In Type A zones, occurrences of characteristic earthquakes are time dependent and the inter-occurrence time follows a lognormal distribution. In the B zones, occurrences of earthquakes are
time independent (i.e. Poisson distribution) and occurrence time intervals are exponentially distributed. The main random occurrence parameters are generated as follows.

### 3.4.1 Number of Earthquakes

To determine the number of earthquakes in a certain seismic zone, a random variable, $u_k$, with a uniform distribution between 0 and 1 is generated and must satisfy:

$$P(X = n_k - 1) < u_k \leq P(X = n_k) \quad ; \quad k = 1, 2, \ldots, N_{sz}$$  \hspace{1cm} (3.1)

where $n_k$ is the number of earthquakes that occur with a moment magnitude greater than 6 in seismic zone $k$ during a period of $t$ years from present ($t = 10$ in this study); $N_{sz}$ is the total number of seismic zones in the reference area of Santa Barbara ($N_{sz} = 29$ in this study); $P(X = n_k)$ is the probability of having $n_k$ earthquake occurrences with moment magnitude greater than 6 in seismic zone $k$ during a period of 10 years. According to this relationship, for characteristic earthquakes in Type B zones and distributed earthquakes in all three different types of seismic zones, $n_k$ must satisfy:

$$\sum_{X=0}^{n_k} \frac{(t \nu_k)^X}{X!} e^{-t \nu_k} < u_k \leq \sum_{X=0}^{n_k} \frac{(t \nu_k)^X}{X!} e^{-t \nu_k} \quad ; \quad k = 1, 2, \ldots, N_{sz}$$  \hspace{1cm} (3.2)

where $\nu_k$ is the mean annual occurrence rate of earthquakes in seismic zone $k$ with moment magnitude greater than 6. For characteristic earthquakes in Type A zones, lognormally distributed random numbers $T_i$’s are generated as occurrence time intervals and must satisfy:

$$\left( \frac{\sum_{i=1}^{n_k} T_i}{T_0} \right) - T_0 \leq t$$  \hspace{1cm} (3.3)

where $T_0$ is the time elapsed from last event to present; the relationship between $T_i$ and $T_0$ is shown schematically in Figure 3.4. The total number of earthquakes in seismic zone $k$ during a period of 10 years, $N_k$, can be expressed as

$$N_k = n_{k,c} + n_{k,d}$$  \hspace{1cm} (3.4)
where \( n_{k,c} \) is the number of characteristic earthquakes in zone \( k \); \( n_{k,d} \) the number of distributed earthquakes in zone \( k \). In this study, a total number of 1000 simulations of 10-year periods (starting from the year 2000) are simulated resulting in 1815 ground motions. The annual probability of no earthquakes greater than moment magnitude 6 based simulation is 0.83, which is close to the target value 0.78 (calculated from the 1995 WGCEP report). 1000 simulations of 10-year period give a satisfactory estimate of uniform hazard response spectra, which will be shown in Chapter 4.

3.4.2 Magnitude

For characteristic earthquakes, the moment magnitude \( M_w \) is related to the length of the fault segment \( L \) and the characteristic displacement \( D \) as follows:

\[
M_w = \frac{2}{3} \cdot \log_{10}(M_0) - 6
\]

\[
M_0 = \mu \cdot H \cdot L \cdot D
\]

where \( \mu \) is the rigidity of the earth crust (assumed to be \( 3 \times 10^{10} \) Nm\(^{-2}\)); \( H \) is the thickness of the brittle crust (taken to be 11 km following here).

For distributed earthquakes, the magnitude is considered a random variable following the modified Gutenberg-Richter distribution:

\[
F_{M_w}(M_w \leq m) = 1 - \frac{N(m, m_x, f_d)}{N(6, m_x, f_d)}
\]

\[
N(m, m_x, f_d) = f_d \cdot 10^{6-m} \cdot \frac{1 - 10^{m-m_x}}{1 - 10^{6-m_x}}
\]

where \( F_{M_w}(M_w \leq m) \) is the cumulative distribution function of moment magnitude \( M_w \) (note that \( 6 \leq m \leq m_x \) and \( m_x \) indicates the limiting moment magnitude of a given seismic zone); \( N(m, m_x, f_d) \) is the cumulative rate of \( m \leq m_x \) earthquakes and \( f_d \) is the occurrence rate of distributed earthquakes with moment magnitude greater than 6. The modified Gutenberg-Richter distribution is shown in Figure 3.5 for comparison with the original distribution.
3.4.3 Epicenter

It is assumed that earthquakes are equally likely to occur anywhere within a given seismic zone. To do so, an initial epicenter location is generated from a uniform distribution function and then a rupture plane within the given seismic zone is drawn according to this initial epicenter location to have a rupture size comparable with its magnitude, and appropriate strike and dip angles. However, if this initial epicenter location fails to satisfy the required rupture condition, another epicenter will be generated until all requirements are satisfied.

3.4.4 Focal Depth

The focal depth, $H$ (km) is assumed to have a triangular probability density function with a lower bound of 4 km, an upper bound of 24 km, and a mode of 14 km. This choice of distribution is based on data compilation from the earthquake catalogue in the effective area of Santa Barbara (Seekins et al. 1992, NOAA, 1996). To generate random numbers conforming to a given triangular probability density function, the inverse transform method is employed. The details can be referred to Ang and Tang (1990)

3.5 Ground Motion Modeling

For far-field events, a point source is used. For near-field earthquakes, a strike-slip or dip-slip rupture is assumed according to the tectonic characteristics of the corresponding seismic zone. The ground motions are modeled as a random process composed of sinusoidal waves with specified frequencies and random phase delays (Shinozuka and Jan 1972, Shinozuka and Deodatis 1991), with a Fourier amplitude spectrum according to the empirical model by Trifunac (1994). The attenuation effect takes the form of Boore et al. (1993), in which prediction error is also considered. Near-field effects are incorporated according to Somerville et al. (1997a) with modification to
emphasize long-period motions in the strike-normal direction, which are generally associated with velocity pulses. The resulting ground motions are then baseline-corrected using a 2\textsuperscript{nd}-order parabola (Brady 1966, Nigam and Jennings 1968).

3.5.1 Duration

The significant duration of strong motion, \( t_d \) (sec), follows Eliopoulos and Wen (1991):

\[
\log_{10} t_D = -0.14 + 0.2 M_w + 0.002 R_e + \varepsilon_d
\]  

where \( t_D \) is the significant duration of strong motion, i.e., the time interval required to build up between 5% and 95% of the Arias intensity of the record (Trifunac and Brady, 1975); \( M_w \) is moment magnitude; \( R_e \) is epicentral distance (km); \( \varepsilon_d \) is the prediction error that follows normal distribution with a zero mean and a standard deviation of 0.135. The total duration is therefore determined by the following equation:

\[
t_F = c_1 \cdot t_D + t_D + c_2 \cdot t_D
\]

where \( c_1 \) and \( c_2 \) are random variables intended to model the buildup and decay phases of the ground motion. The random variables \( c_1 \) and \( c_2 \) are assumed be uniformly distributed between 0.1 and 0.5, and between 0.5 and 1.0, respectively. The upper bound of \( t_F \) is set at 60 seconds.

3.5.2 Fourier Amplitude Spectrum

The median amplitude of the Fourier spectrum, \( FS(T) \), is determined by the empirical scaling equation by Trifunac (Trifunac and Lee 1985, Trifunac 1994):

\[
\log_{10} FS(T) = M_c + Att(\Delta, M_w, T) + b_1(T) \cdot M_\infty + b_2(T) \cdot h + b_3(T) \cdot \nu + b_4(T) \cdot h \nu + b_5(T) \cdot M_\infty^2 + b_6(T) \cdot S_\infty + b_7(T) \cdot S_\infty^2
\]  

where

\( FS(T) \equiv \) Fourier amplitude spectrum of acceleration at period \( T \) (inches/second)
\[ \text{Att}(\Delta, M, T) = \text{frequency-dependent attenuation function ( } T=1/f \text{ ) of the spectral amplitudes versus the "representative" source to station distance } \Delta \text{ and magnitude } M \] (details can be referred to Trifunac, 1994)

\[ M_c = \min(M, M_{\max}) \]
\[ M_{\max} = \max(M_{\min}, M_c) \]

\[ h = \text{depth (thickness) of the sedimentary layer beneath the station (km)} \]
\[ v = \text{indicator variable ( } \nu = 0 \text{ for horizontal motion, } \nu = 1 \text{ for vertical motion) } \]
\[ b_I \sim b_I^{(2)} = \text{scaling coefficient functions of the period } T \]
\[ S_L^{(1)}, S_L^{(2)} = \text{indicator variables describing the local soil site condition} \]
\[ S_L^{(1)} = \begin{cases} 1 & \text{if } s_L = 1 \text{ (stiff soil)} \\ 0 & \text{otherwise} \end{cases} \]
\[ S_L^{(2)} = \begin{cases} 1 & \text{if } s_L = 2 \text{ (deep soil)} \\ 0 & \text{otherwise} \end{cases} \]

\[ s_L = 0, 1 \text{ and } 2, \text{ representing rock, stiff soil, and deep soil site, respectively.} \]

\[ \Delta = S \cdot \left( \ln \frac{R_s \, H^2 + S^2}{R_s \, H^2 + S_0^2} \right)^{1/2} \]

\[ R_s = \text{epicentral distance (km)} \]
\[ H = \text{focal depth (km)} \]
\[ S = \text{source dimension (details can be referred to Trifunac, 1994)} \]

The above equation is valid only for periods below a certain cut-off period, which depends on the magnitude under consideration (Trifunac, 1994). For periods above the cut-off period, the Fourier amplitude spectrum is extrapolated using a straight line with a slope of \( 2^{s-M} \) when \( FS(T) \) vs \( T \) plot is on log-log scale (Collins et al. 1995). The sensitivity of the Trifunac Fourier amplitude to varying magnitude-distance combinations is shown in Figure 3.6. It is noted that in this study the Fourier amplitude is used to
describe the general spectral shape due to attenuation and the actual amplitude is scaled using target peak ground acceleration due to Boore et al. (1993, 1994, 1997) considering attenuation uncertainty:

\[
\ln(PGA) = -0.038 + 0.216 \cdot (M_w - 6) - 0.777 \cdot \ln(r) + 0.158 \cdot G_B + 0.254 \cdot G_C + \varepsilon_A \quad (3.10)
\]

and

\[
r = \sqrt{R_{jb}^2 + d^2} \quad (3.11)
\]

where \(R_{jb}\) is the closest horizontal distance to the vertical projection of the rupture (km); \(d\) is the fictitious depth for better fit of earthquake records (\(d = 5.48\) km is used); \(PGA\) is the peak ground acceleration (g); \(M_w\) is moment magnitude; \(G_B\) and \(G_C\) are parameters related to site soil condition (\(G_B = 1\) and \(G_C = 0\) are used for Santa Barbara); \(\varepsilon_A\) is the uncertainty term modeled as a normally distributed random variable with a zero mean and a standard deviation of 0.205.

### 3.5.3 Spectral Representation of Ground Motions

According to Shinozuka and Deodatis (1991), ground motion, \(A(t)\), can be simulated as follows:

\[
\begin{align*}
A(t) &= I(t) \cdot \left[ \sum_{m=1}^{p} a_1(t) \cdot Amp(m\Delta f) \cdot \cos(2\pi m\Delta f) t + \phi_m \right] \\
&\quad + \sum_{m=p+1}^{N} a_2(t) \cdot Amp(m\Delta f) \cdot \cos(2\pi m\Delta f) t + \phi_m \right] \\
&\quad + \sum_{m=p+1}^{N} a_2(t) \cdot Amp(m\Delta f) \cdot \cos(2\pi m\Delta f) t + \phi_m \right]
\end{align*}
\]

and

\[
Amp(m\Delta f) = 2 \sqrt{\frac{\Delta f}{t_D}} \cdot FS(m\Delta f) \quad (3.13)
\]

where \(I(t)\) is intensity modulation function; \(a_1(t)\) and \(a_2(t)\) are frequency modulation functions; \(FS(m\Delta f)\) is the Fourier amplitude spectrum at a certain frequency \(f_m = m\Delta f\); \(t_D\) is the significant duration of ground motion (sec); \(\Delta f\) is the cyclic frequency increment.
(0.015 Hz is used); $\phi$ is the phase angle modeled as a uniform distributed random variable between 0 and $2\pi$; $N$ is the number of frequencies at which values of the Fourier amplitude spectrum are estimated; $p = \text{integer}(f_{rms}/Af)$; $f_{rms}$ is the central frequency or the so-called root-mean-square frequency (Hz) defined as:

$$f_{rms} = \sqrt{\frac{\sum_{m=1}^{N} f_m^2 \cdot FS(f_m)}{\sum_{m=1}^{N} FS(f_m)}}$$  \hspace{1cm} (3.14)$$

The intensity modulation function $I(t)$ takes the form suggested by Jennings et al. (1968):

$$I(t) = \begin{cases} 
\left( \frac{t}{c_it_D} \right)^2 & \text{if } t < c_it_D \\
1 & \text{if } c_it_D \leq t \leq c_it_D + t_D \\
\exp[-\kappa(t-(c_it_D+t_D))] & \text{if } t > c_it_D + t_D 
\end{cases}$$  \hspace{1cm} (3.15)$$

where $\kappa$ is defined such that $I(t_f) = 0.05$. To incorporate the time-varying frequency content, a simple ramping function is introduced for a weak frequency modulation of the resulting ground motion (Collins et al. 1995):

$$\begin{cases} 
a_1(t) = \frac{t}{c_it_D} ; & a_2(t) = 1 & \text{if } t < c_it_D \\
a_1(t) = 1 ; & a_2(t) = 1 & \text{if } c_it_D \leq t \leq c_it_D + t_D \\
a_1(t) = 1 ; & a_2(t) = 1 - \frac{t-c_it_D-t_D}{c_it_D^2} & \text{if } t > c_it_D + t_D 
\end{cases}$$  \hspace{1cm} (3.16)$$

3.5.4 Near-Field Effects

For near field earthquakes, the length and the orientation of the rupture segment become more important. In view of this, three different models are used to describe the seismological characteristics of the site. For earthquakes with a closest rupture distance
less than 50 km, a faulting mechanism of vertical strike-slip or dip-slip with surface rupture is used with parameters of the 1995 WGCEP report; otherwise, a point source model would yield satisfactory results. For a given magnitude, the length and width of the rupture plane can be expressed by empirical formulae (Wells and Coppersmith, 1994):

\[
\log_{10}(SRL) = \begin{cases} 
-3.55 + 0.74M_w & \text{for strike-slip} \\
-2.86 + 0.63M_w & \text{for dip-slip}
\end{cases}
\]  

(3.17)

\[
\log_{10}(RA) = \begin{cases} 
-3.42 + 0.90M_w & \text{for strike-slip} \\
-3.99 + 0.98M_w & \text{for dip-slip}
\end{cases}
\]  

(3.18)

\[RW = RA / SRL\]  

(3.19)

where \(SRL\) is the surface rupture length (km), \(RA\) is the rupture area (km²), \(RW\) is the rupture width (km), and \(M_w\) is moment magnitude. After the rupture plane is generated, the next step is to determine whether the modification due to near-field effects is necessary. Based on an empirical analysis of near-field data, Somerville et al. (1997a) proposed a modification due to rupture directivity effect, which indicates larger spectral acceleration for periods longer than 0.6 second when the rupture propagates toward the site. The modification can be expressed as:

\[y = \cos 2\zeta \left[ C_1 + C_2 \ln(R_{rup} + 1) + C_3(M_w - 6) \right]\]  

(3.20)

where \(y\) is the natural logarithm of the strike-normal to average horizontal spectral acceleration ratio; \(C_1, C_2,\) and \(C_3\) are period-dependent coefficients (Somerville et al., 1997); \(R_{rup}\) is the closest distance to the rupture plane (km); \(M_w\) is moment magnitude; \(\zeta = \theta\) if a strike-slip fault is considered; \(\zeta = \phi\) if dip-slip fault is considered; \(\theta\) is the azimuth angle between fault plane and ray path to site (Figure 3.7); \(\phi\) is the zenith angle between fault plane and ray path to site (Figure 3.7). Empirical equation (3.20) is valid only for \(M_w > 6, R_{rup} < 50\) km and \(\zeta < 45^\circ\); otherwise, \(y = 0\). If spectrum modification is required, the following iteration procedure (Levy and Wilkinson, 1976, Shinozuka et. al., 1988) is activated:
where $\text{Ampl}_i(f_m)$ is the spectral amplitude after $i^{th}$ iteration at cyclic frequency $f_m (= mAf)$; $RS_i(f_m)$ is the target response spectrum at cyclic frequency $f_m$; $RS_i(f_m)$ is the response spectrum after $i^{th}$ iteration at cyclic frequency $f_m$.

### 3.6 Simulation Results

300 simulations of 10-year records in the reference area of Santa Barbara are shown in Figure 3.8, which contains 517 earthquakes. By comparison of Figure 3.8 and Figure 2.4, they both have about the same number of simulated earthquakes, but the simulation period in Figure 2.4 is doubled and the reference area is 10 times larger, owing to the lower seismic activities in Mid-America. The very frequent seismic activity shown to the north of Santa Barbara is in the San Andreas Fault (Zone 3 more precisely); however, due to the rapid attenuation effect in California, it is the local seismicity that makes major contribution to the seismic hazard in Santa Barbara (USGS, 1997). A reference zone of 150 km x 150 km therefore covers all possible hazards of engineering interest.

A sample acceleration time history generated in Zone 40 is shown in Figure 3.9(b). Its response spectrum is given in Figure 3.9(a), in which near-field effects are considered. A permanent displacement offset (Figure 3.9(d)) is observed due to the large magnitude and very close distance.

The number required of artificial ground motions in a Monte Carlo simulation depends on: (1) the occurrence rates of events in the seismic zones in the reference area, (2) required accuracy of the engineering problem under consideration, and (3) the demand of computational time involved in the engineering problem. More details will be given in the following chapter.
Table 3.1  Locations and zone types of major faults surrounding Santa Barbara.

<table>
<thead>
<tr>
<th>Major Surface Fault</th>
<th>Location</th>
<th>Zone Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arroyo Parida</td>
<td>46</td>
<td>C</td>
</tr>
<tr>
<td>Big Pine</td>
<td>26</td>
<td>B</td>
</tr>
<tr>
<td>Garlock</td>
<td>27</td>
<td>B</td>
</tr>
<tr>
<td>Kern Front</td>
<td>53</td>
<td>C</td>
</tr>
<tr>
<td>Malibu Coast</td>
<td>33</td>
<td>B</td>
</tr>
<tr>
<td>Newport-Inglewood</td>
<td>20</td>
<td>B</td>
</tr>
<tr>
<td>Oakridge</td>
<td>46</td>
<td>C</td>
</tr>
<tr>
<td>Ozena</td>
<td>65</td>
<td>C</td>
</tr>
<tr>
<td>Palos Verdes</td>
<td>34</td>
<td>B</td>
</tr>
<tr>
<td>Pine Moutain</td>
<td>40</td>
<td>B</td>
</tr>
<tr>
<td>Rinconada</td>
<td>37</td>
<td>B</td>
</tr>
<tr>
<td>San Andreas</td>
<td>3, 4, 5</td>
<td>B, A, A</td>
</tr>
<tr>
<td>San Cayetano</td>
<td>46</td>
<td>C</td>
</tr>
<tr>
<td>San Gabriel</td>
<td>31, 32</td>
<td>B, B</td>
</tr>
<tr>
<td>San Juan</td>
<td>50</td>
<td>C</td>
</tr>
<tr>
<td>Santa Cruz</td>
<td>35</td>
<td>B</td>
</tr>
<tr>
<td>Santa Monica</td>
<td>33</td>
<td>B</td>
</tr>
<tr>
<td>Santa Ynez</td>
<td>40</td>
<td>B</td>
</tr>
<tr>
<td>Sierra Madre</td>
<td>31</td>
<td>B</td>
</tr>
<tr>
<td>White Wolf</td>
<td>25</td>
<td>B</td>
</tr>
</tbody>
</table>
Table 3.2 Parameters of seismic source model in the effective area of Santa Barbara.

<table>
<thead>
<tr>
<th>Zone</th>
<th>Type</th>
<th>Return Period (yr)</th>
<th>Last Rupture</th>
<th>Limiting magnitude $m_c$</th>
<th>$t_1$ $(10^4$yr)</th>
<th>$t_2$ $(10^4$yr)</th>
<th>Faulting Mechanism</th>
<th>Range of Strikes</th>
<th>Dip</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>B</td>
<td></td>
<td></td>
<td>7.05</td>
<td>3.54</td>
<td>13.74</td>
<td>SS</td>
<td>$0^\circ$ ~ $180^\circ$</td>
<td></td>
<td>1857/11/9 Fort Tejon m=7.8 RL</td>
</tr>
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Initiate the $n^{th}$ simulation

Determine no. of earthquakes in the $n^{th}$ simulation, i.e. characteristic and distributed earthquakes.

Generate random numbers for each earthquake sample function e.g. magnitude (rupture length, width), distance ($R_e, R_{jb}, R_{rup}$), hypocenter (focal depth), duration, strike/dip, etc.

Determine the general spectral shape due to attenuation (e.g. Trifunac 1994)

- Determine peak ground acceleration due to attenuation e.g. Boore et al. 1993
- Construct intensity and frequency modulations e.g. Jennings et al. 1968, Collins et al. 1995
- Build mathematical form for stationary ground motion e.g. Shinozuka & Deodatis 1991

$n = n^{th}$ simulation is initiated

Modification due to near-field effects? (Somerville et al. 1997)

Baseline correction (Brady 1966)

Sample size is large enough?

Modify the resulting ground motion to match the response spectrum. e.g. iteration procedure due to Levy & Wilkinson 1976, Shinozuka et al. 1988

Figure 3.1 Ground motion simulation flowchart for Santa Barbara, CA.
Figure 3.2  (a) Major faults (thin black lines) in the effective area of Santa Barbara, (b) Seismic zones contributing to the seismic hazard at Santa Barbara.
Figure 3.3 The discretized reference area and seismic zones surrounding Santa Barbara.

Figure 3.4 Illustration of time intervals defined in the renewal model.
Figure 3.5 Comparison of original and modified Gutenberg-Richter distribution functions.
Figure 3.6 Sensitivity of Fourier amplitude spectrum by Trifunac (1994).
Figure 3.7 Definition of rupture directivity parameters $\theta$ for strike-slip faults, $\phi$ for dip-slip faults and possible region with hanging wall effect for dip-slip faults (unshaded area).
Simulated earthquake sources (300 10-yrs, 517 earthquakes)

Figure 3.8 Epicenters and magnitudes of earthquakes of simulated 3000-year record in the reference area of Santa Barbara, CA.
Figure 3.9  Artificial near-field ground motion with dip-slip faulting mechanism (PGA = 0.8 g, $M_w = 6.15$, $H = 8$ km, $R_e = 6.2$ km, $R_{jb} = 0$ km, $R_{rup} = 1.2$ km, $RL = 10.3$ km, $RW = 10.6$ km, strike = 38.5°, dip = 58.1°).
CHAPTER 4
UNIFORM HAZARD RESPONSE SPECTRA AND GROUND MOTIONS

4.1 Overview

In a performance-based design, earthquake ground motions are needed for evaluation of structural response. The 1997 NEHRP provisions consider two hazard levels – design earthquake and maximum considered earthquake ground motions. The former is defined as an event of 10% probability of exceedance in 50 years or a mean return period of 474 years; the latter has a 2% probability of exceedance in 50 years or a mean return period of 2475 years. FEMA-273 (1997) adopted the same framework and defined performance levels accordingly to various building categories and rehabilitation objectives. The Vision 2000 document (SEAOC 1995) proposed a more refined matrix of performance check as shown in Figure 4.1. At small probability level of 2% in 50 years, recorded motions are scarce, synthetic ground motions are generally necessary for structural time history analysis (Somerville et al. 1997). It can be done by scaling ground motion records to match target response spectra using ground motion records from seismic zones with similar tectonic environments, or simulating ground motions based on seismotectonic characteristics surrounding the site location.

In scaling ground motions, it should be pointed out that a response spectrum according to PSHA (Probabilistic Seismic Hazard Analysis) does not, and was never, intended to represent the response of a single-degree-of-freedom (SDOF) structure to the ground motion of a single event (Naeim and Lew, 1995). Rather, it is intended to be an envelope of responses to multiple events that correspond to a specified probability level. Therefore, the so-called "response spectrum compatible" acceleration time history which fits a design response spectrum by scaling a given ground motion may contain energy over a wide range of structural periods that is not seen in actual records. It can lead to
gros overestimate of the displacement demand and energy input. It is also noted that the
duration of the scaled ground motion record remains unchanged whereas the duration is
highly dependent on magnitude (rupture size) and distance (wave dispersion) according
to Carballo and Cornell (1998). As a result, scaled ground motions give poor results of
responses that are sensitive to the duration (e.g. hysteretic energy dissipation).

The phenomenological simulation procedure is therefore used to generate suites of
ground motions appropriate for structural performance analyses of general building
stocks. A large number (thousands) of ground motion time histories are simulated at a
given site, from which uniform hazard response spectra (UHRS) are constructed and
corresponding suites of ten ground motions are selected for analysis of nonlinear systems.
They allow efficient probabilistic performance evaluation of structures under future
earthquakes.

### 4.2 Uniform Hazard Response Spectra

The large number of ground motion time histories generated in Chapters 2 and 3
allow one to obtain the uniform hazard response spectra (UHRS). At a given city, the
response acceleration spectrum for each time history is first calculated. The probability
distributions of spectral acceleration for a given period (50 years) are then obtained from
which one can construct the UHRS. The annual exceedance probability can be
calculated from probability of exceedance in t years:

\[
PE_{ann} = 1 - (1 - PE_t)^{\frac{50}{t}}
\]  \hspace{1cm} (4.1)

where \( PE_{ann} \) is the annual probability of exceedance; \( PE_t \) is the probability of exceedance
in t years assuming occurrence independence except for the characteristic earthquakes in
Zone A of California. Conversions from 50-year to annual exceedance probabilities and
return periods are provided in Table 4.1 for quick reference. The UHRS can be used to
evaluate the performance of linear systems using method of modal superposition.
4.2.1 Linear Elastic Systems

For a single degree of freedom oscillator with a linear elastic restoring force, the governing equation of motion can be written as

$$\ddot{x} + 2\xi \omega_n \dot{x} + \omega_n^2 x = -\ddot{x}_g$$  \hspace{1cm} (4.2)

where \(x\) is the relative displacement of the mass to the ground; \(x_g\) is the ground displacement; \(\omega_n\) is the natural frequency of the oscillator; and \(\xi\) is the damping ratio. A damping ratio of 0.05 is used. As a measure of severity of structural response we define a nondimensional spring force coefficient \(C_e\) as a function of the maximum relative displacement \(S_d\) and natural frequency \(\omega_n\) of the SDOF oscillator

$$C_e = \frac{\text{maximum spring force}}{\text{mass} \times \text{gravity}} = \frac{\omega_n^2 S_d}{g} = \left(\frac{2\pi}{T_n}\right)^2 \frac{S_d}{g}$$  \hspace{1cm} (4.3)

where \(g\) is the acceleration due to gravity, and \(T_n\) is the natural period of the SDOF oscillator. To determine \(C_e\) corresponding to a given probability of exceedance, an extension of the Type II extreme value distribution is proposed (Appendix B):

$$P_{\text{e}}(C_e) = \left[1 - p_0 - w\right] \left[1 - \exp\left(-\frac{\nu_1}{C_e^{k_1}}\right)\right] + w \left[1 - \exp\left(-\frac{\nu_2}{C_e^{k_2}}\right)\right] \left[1 - \exp\left(-\frac{\nu_c}{C_e^{k_c}}\right)\right]$$  \hspace{1cm} (4.4)

where \(p_0\) is the annual probability of no earthquakes greater than \(m_b - 5\), and \(\nu_1, \nu_2, \nu_c, k_1, k_2, k_c\) and \(w\) are distribution parameters. Equation (4.4) reduces to the original Type II extreme value distribution (Ang and Tang, 1990) when \(w = 0\) and \(\nu_c \to \infty\), i.e. \(\nu_c \gg \max(C_e)\). The design spectral value according to a specified hazard level can be readily determined by Equation (4.4). The fitting of simulated results by Equation (4.4) and determination of the required elastic design force coefficient for a given probability are shown in Figure 4.2.
4.2.1 Nonlinear Inelastic Systems

The nonlinear inelastic restoring force of the SDOF oscillator is represented by the smooth hysteretic model proposed by Wen (1976). The governing equation of motion for an SDOF oscillator with this type of restoring force can be expressed as

\[ \ddot{x} + 2 \xi \omega_n \dot{x} + \omega_n^2 f(\alpha, x, z) = -\ddot{x}_g \quad ; \quad f(\alpha, x, z) = ax + (1 - \alpha)z \]  

(4.5)

where \( \alpha \) is the post-to-preyield stiffness ratio (or, strain hardening ratio), \( z \) is the auxiliary state variable describing the hysteretic path, and dots indicates time derivative. \( A, \beta, \gamma, \eta, \nu \) and \( n \) are parameters to define the shape of the hysteresis loop, in which \( A, \eta \) and \( \nu \) define the degree of degradation; \( \beta \) and \( \gamma \) control the level of the yield strength, and \( n \) defines the transition zone between elastic and plastic regions. A system with both strength and stiffness degradation is shown in Figure 4.3. According to Baber and Wen (1979), \( A, \eta \) and \( \nu \) take the form

\[ \begin{aligned} 
A &= A_0 - \delta_A E \\
\eta &= \eta_0 + \delta_\eta E \\
\nu &= \nu_0 + \delta_\nu E 
\end{aligned} \]  

(4.7)

where \( A_0, \eta_0 \) and \( \nu_0 \) are initial values (\( A_0=\eta_0=\nu_0=1.0 \) is commonly used); \( \delta_A, \delta_\eta \) and \( \delta_\nu \) represent the rate of degradation (\( \delta_A=\delta_\eta=\delta_\nu=0 \) for nondegrading systems); \( E \) is the normalized dissipated hysteretic energy (Yeh and Wen, 1989), defined as

\[ E = \frac{1-\alpha}{F_y \Delta_y} \int_0^t k_0 z \dot{x} \, dt \]

where \( F_y \) and \( \Delta_y \) are the yield strength and yield displacement, respectively. The integration represents the hysteretic energy dissipated by the system due to inelastic deformation and \( k_0 \) represents pre-yielding stiffness. \( \beta \) and \( \gamma \) are related to the yield displacement as follows:
\[ \beta = \gamma = \frac{1}{2 \Delta_y} \]  

(4.8)

\( n = 5 \) is used in this study. To better fit the actual hysteretic behavior of buildings, a minor modification is made on \( \eta \) and \( \nu \):

\[
\begin{cases}
\eta = \delta \left( \frac{E}{E_c} \right) \\
\nu = \delta \left( \frac{E}{E_c} \right)
\end{cases}
\]

(4.9)

where \( A = 1 \) is used in this study; \( \delta_\eta \) and \( \delta_\nu \) are parameters to define stiffness and strength degradation rates respectively (\( \delta_\eta, \delta_\nu \geq 1.0 \)); \( E \) is the normalized dissipated hysteretic energy; \( E_c \) is the threshold hysteretic energy at which \( \eta \) reaches \( \delta_\eta \) (or, \( \nu \) reaches \( \delta_\nu \)). We take \( E_c \) as the normalized dissipated hysteretic energy during one loading cycle. Analogous to the elastic system, a nondimensional yield force coefficient, \( C_y \), is defined as the ratio of the restoring force at the yield point to the weight of the SDOF oscillator:

\[ C_y = \frac{\omega^2 \Delta_y}{g} \]  

(4.10)

A system with both strength and stiffness degradation is shown in Figure 4.3. Figure 4.4(a) shows the nondegrading hysteresis model used in this study; Figure 4.4(b) shows the degrading case. We assume 5% stiffness degradation and the strength degradation at 5% and 10% in this study. The proposed modified hazard curve can be used to determine the values of yield force coefficients at various probability levels as follows,

\[
P(\mu > \mu_t, GIVEN C_y) = \\
\left[ 1 - p_0 - w \right] \cdot \left[ 1 - \exp \left( -\left( \frac{\mu_1}{C_y} \right)^{k_1} \right) \right] + w \cdot \left[ 1 - \exp \left( -\left( \frac{\nu_2}{C_y} \right)^{k_2} \right) \right] \cdot \left[ 1 - \exp \left( -\left( \frac{\nu_c}{C_y} \right)^{k_c} \right) \right]
\]

(4.11)

where \( \mu \) is displacement ductility; \( \mu_t \) is target displacement ductility; \( p_0, \nu_1, \nu_2, \nu_c, k_1, k_2, k_c \) and \( w \) are defined in Equation(4.4).
4.2.2 Results

4.2.2.1 Uniform Hazard Response Spectra (UHRS)

The UHRS is an efficient means of representing seismic hazards for probabilistic performance (fragility) evaluation of linear and nonlinear structures (e.g., Collins et al. 1995). The UHRS for Memphis, Carbondale, St. Louis and Santa Barbara are obtained from the simulated ground motions. For a validation of the results, the UHRS in Memphis obtained in this study are first compared with those of 1997 USGS national earthquake hazard maps for B/C boundary soil classification (i.e. firm rock with an average shear velocity of 760 m/sec in the top 30 m). This generic site condition facilitates the comparison with previous studies by USGS and other researchers. The UHRS constructed based on the simulated ground motions for three probability levels for Memphis are shown in Figure 4.5. The spectral accelerations at periods of 0.2 sec, 0.3 sec, and 1.0 sec according to 1997 USGS national earthquake hazard maps for Memphis are also shown in the figure. The agreements are generally very good. Since the input seismicity data to these two models are essentially the same, the differences can be attributed to:

1. The USGS study used a point-source model and the closest distance from the fault to the site for magnitude-8 events whereas a finite fault model with a epicenter located randomly within the fault is used in this study.
2. The USGS study used an S-shaped fault trace with a rupture length of more than 230 km occupy the entire New Madrid seismic zone, whereas this study uses a straight fault trace with a rupture length of 140 km that may change from occurrence to occurrence within the New Madrid seismic zone.
3. For point sources, the one-corner-frequency source model was used in the USGS study whereas the two-corner-frequency source model (Equation 2.5) is used in this study which has been shown to give better fit to records in the CEUS (Atkinson and Boore 1998). It is observed that the two-corner model
predicts considerably lower ground motions amplitude than one-corner model in the case of large events (e.g. moment magnitude greater than 7).

4. The Atkinson-Boore attenuation equation generally predicts lower ground motions than the empirical attenuation model proposed in USGS OFR-96-532 and the 1993 EPRI report.

The UHRS for the three Mid-America cities with the “representative” soil profiles are shown in Table 2.3 to 2.5 and compared with FEMA 273 recommendations for design. FEMA 273 spectra are for design check and are based on the 1997 USGS national earthquake hazard maps. There are five generic soil classifications according to the upper 30m of soil and the amplification factor is largely based on empirical results of Loma Prieta and Northridge earthquakes (e.g., Borcherdt, 1994). The UHRS for the Memphis representative soil profile are shown in Figure 4.6. It is seen that compared with those for the B/C boundary, the UHRS are amplified almost by a factor of two for periods greater than 1.0 sec. and reduced for T < 0.3 sec due to the deep soil layer. The agreements are generally good for T > 0.7 sec. The differences for T< 0.5 sec. are partly due to the differences in the source models and attenuation relations as mentioned in the foregoing and partly due to the differences in soil amplification factors. The UHRS for the Carbondale representative soil profile are shown in Figure 4.7. The agreements are generally good. The UHRS for St. Louis representative soil profile are shown in Figure 4.8. There are some major differences. Compared with the FEMA Class C spectra, the UHRS are much lower for T > 0.2sec and much higher for T < 0.2 sec because of the comparatively thin (16m) layer of soil on rock. The current results are more in agreement with the findings of Saikia and Somerville (1997) indicating a much lower seismic threat to St. Louis. It is pointed out that the actual soil profile in the St. Louis area may have a wide variation and could differ significantly from the “representative profile” used in this study, especially at locations close to the Mississippi River. Deeper soil layers causing larger soil amplification at longer periods are certainly possible at these locations. Ground motions at the surface can be generated from the bedrock ground motions using a proper soil amplification model when detailed information for the soil profile is available.
The UHRS obtained for Santa Barbara in this study is also compared with those of Collins et al. (1995) and USGS results (1997) in Figure 4.9. They are generally in very good agreement except that in the short period range at the 10% in 50 years hazard level where this study shows lower spectral amplitude than USGS. According to Stirling and Wesnousky (1998), the difference can be attributed to th in:

1. The size of maximum magnitude assigned to a given fault.
2. The proportion of predicted earthquakes that are distributed off the major faults.
3. The use of geodetic strain data to predict earthquake rates.

when different seismological databases are used.

4.2.2.2 Nonlinear Inelastic Uniform Hazard Response Spectra

The nonlinear inelastic UHRS of Memphis, TN for nondegrading systems are shown in Figure 4.10. UHRS for systems with 5% and 10% strength degradations are shown in Figures 4.11-12. The degrading systems are models for typical dynamic behavior of steel buildings with a 2% damping and 3% post-to-pre-yield stiffness ratio. The stiffness degradation rate is held constant at 5% for each cycle. The strength degradation rate is slightly varied at two different levels, 5% and 10%, considering that with appropriate detailing design the strength degradation rate should be within 10% for each loading cycle. The effect due to the difference in degradation rates is small and will be confirmed again in Chapter 5. The nonlinear UHRS of nondegrading system is also shown for Carbondale, IL, St. Louis, MO and Santa Barbara, CA in Figures 4.12-15. For Santa Barbara, CA, the nonlinear UHRS of degrading systems is given in Figure 4.16 for comparison with Memphis, TN. The effect of degradation is more obvious in Santa Barbara, CA.
4.3 Uniform Hazard Ground Motions

4.3.1 Proposed Selection Procedure

For nonlinear systems, a time history response analysis is generally required since response spectra method based on modal superposition principles no longer applies. For this purpose, suites of ground motions for each probability level are selected. The selection criterion is that the deviation of the median response spectra of the suite from the UHRS is minimized. The median value is used because it is less sensitive to sample fluctuation and facilitates the estimation of the parameters of the underlying lognormal distribution. To accomplish this, the response spectral accelerations $S_a$ at 10 key structural periods (0.05 $\sim$ 1 sec) of the simulated ground motions are compared with those of the target UHRS for a given probability of exceedance. The ten ground motions with the smallest mean square natural logarithmic ($\ln S_a$) difference are selected. The resultant suite will have a median spectral acceleration that best matches the target UHRS. The matching of the spectral acceleration has been shown by Shome and Cornell (1999) to be the most effective means of selecting ground motions for probabilistic nonlinear structural demand analysis. For performance and fragility analysis of nonlinear structures, one can first calculate the structural responses under the suites of ground motions of given probability level (e.g. 2% in 50 years). The median value of the response will then have a probability of exceedance approximately equal to the ground motion probability. A more detailed performance and fragility analysis can also be performed using the time history response and an appropriate regression analysis (Wang and Wen 1999, Shome and Cornell 1999).

4.3.2 Comparison with Deaggregation Approach

As mentioned earlier, the PSHA-based response spectrum is based on earthquakes of widely different characteristics. A single earthquake (or, design earthquake) can not be expected to represent the seismic hazard at a site. The contribution to hazard from
individual earthquakes can be better seen by means of “deaggregating” the seismic hazard into “bins” of three primary parameters, i.e. magnitude $M$, distance $R$, and ground-motion deviation $\epsilon$ (McGuire, 1995). The deaggregated hazard matrix allows one to select the representative combination of parameters for a “design” earthquake. Recently, it has been pointed out in Bazzurro and Cornell (1998) and USGS OFR (1999) that the deaggregated seismic hazard should be presented on a geographical map such that the causative faults can be identified. It is thus possible to generate suites of ground motions and conduct nonlinear dynamic time history analysis for structural performance evaluation with a single or small number of scenario earthquakes (i.e. events of given magnitude and distance corresponding to the causative fault). Nevertheless, the seismotectonic characteristics in the western and eastern United States are significantly different (e.g. source mechanism and path attenuation); as a result, the deaggregated hazard matrices could be very different. If the hazard matrix “landscape contour” is relatively smooth with no dominant peaks, the use of scenario earthquakes as defined above may not be justified. Furthermore, the deaggregated seismic hazard is a function of structural period. For instance, the seismic hazard in St. Louis at the 1.0-sec structural period is largely from the New Madrid seismic zone (USGS 1999) due to relatively slower attenuation. The seismic hazard at the 0.3-sec period however are due to local seismic event and large earthquake from the New Madrid seismic zone (USGS 1999). This implies that different deaggregation-based ground motion suites need to be provided for structures of different natural periods. The reasons of not using deaggregation method is this study in selecting the events for ground motion generation are summarized as follows:

1. Deaggregation works best when there is a dominant event of certain magnitude (M) and distance (R) and not so well when the M-R “landscape” is flat. The favorable condition does not necessarily prevail in all sites.

2. Deaggregation is dependent on structural response and probability level, i.e. events of spectral accelerations of different periods and different probabilities of exceedance have different M-R “landscape” and hence could result in
different deaggregations and suites of ground motions. It is suitable for purpose of performance evaluation of a particular structure but not the general building stock with a wide range of natural frequencies.

3. Magnitude and distance are much less important, compared to spectral acceleration, in terms of impact on the structural linear and nonlinear responses (Shome and Cornell 1999).

The locations (epicenter, focal depth) and magnitudes of selected uniform hazard ground motions for three Mid-America cities are shown in Figures 4.17-19. For Memphis, major contribution to the 2% in 50 years hazard comes from the New Madrid seismic zone (NMSZ), while local moderate events dominate the 10% in 50 years hazard. For Carbondale, seismic events in NMSZ also make up the 2% in 50 years hazard, while moderate events in both NMSZ and Wabash Valley Seismic Zone have major contribution to the 10% in 50 years hazard. For St. Louis, both small events at local areas and larger events from NMSZ contribute to the 2% in 50 years hazard, while small events at medium distances and moderate events from NMSZ and Wabash Valley Seismic Zone have contribute to the 10% in 50 years hazard. The results are quite similar to the USGS (1999) deaggregation hazard map when more than one spectral accelerations are considered.

4.3.3 Suites of Ground Motions for Memphis, TN

For each of the two hazard levels, 10% and 2% in 50 years, ten ground motions are selected from the large number of simulated motions such that the median spectral accelerations best fit the target UHRS. The selection is done for both ground surface and the bedrock (or rock outcrop). The suite of ground motion time histories for a 10% in 50 years hazard are shown in Figure C.1 (Appendix C). The source (magnitude, epicentral distance, and focal depth) and path (attenuation uncertainty) parameters associated with each ground motion are also shown in the figure. It is seen that at this probability level, seven ground motions come from magnitude-6 events at some distance, two from magnitude-5 events and one from a magnitude-8 event. The response spectra of the ten
ground motions are shown in Figure 4.20 with the target UHRS. The median constructed
from the 10 sample ground motions and the 16-to-84 percentile band are shown. According
to the theory of regression analysis (Ang and Tang 1990), the uncertainty in the median response spectra in terms of the 16-to-84 percentile band is about one third
\((1/\sqrt{10})\) of that shown in the figure. What it entails is that the median value of structural
responses under this suite of ground motions will have very small uncertainty due to
record-to-record variation and will correspond to a probability of exceedance of 10\% in
50 years. One can use it in the structural performance evaluation with some confidence.
The suite of ground motion time histories for a 2\% in 50 years hazard and the response
spectra comparison are shown in Figure C.2 and Figure 4.21 respectively. It is seen that
at such high intensity and low probability level, all ground motions come from
magnitude-8 events. This observation agrees well with the USGS deaggregation results,
which indicate that 89.2\% of the hazard is from magnitude 8 events at 1-sec period and
83\% contribution for the 0.2-sec period. The scatters in the response spectra are larger
but the maximum 16-to-84 percentile uncertainty band for the median value is still
around 10 \% or less. The sample ground motion time histories at rock outcrop (or
bedrock) and the response spectra are shown in Figures C.3-4 and Figures 4.22-23.
Without the soil amplification, the frequency content is seen to shift toward shorter
periods and the spectral accelerations are generally much lower. These ground motions
may be used as inputs to soil amplification software to obtain surface ground motions
when detailed information of the site soil profile is available.

4.3.4 Suites of Ground Motions for Carbondale, IL

The time histories and response spectra of ground motions suites for Carbondale,
Illinois soil sites are shown in Figures C.5-6 and Figures 4.24-25. There is a significant
amplification of motion in the long period range because of the deep and soft soil profile.
At the 10 \% in 50 years level, all contributions come from events of magnitude 5.8 to 7.1.
At the 2\% in 50 years level, all come from magnitude-8 events. The 16-to-84 percentile
bands are reasonably narrow. The suite of ground motions and response spectra for rock
site are shown in Figures C.7-8 and Figures 4.26-27. The trend is the same that compositions of the suites are similar to those for the soil site but the intensities are much lower.

4.3.5 Suites of Ground Motions for St. Louis, MO

The time histories and response spectra of the suites of ground motions at St. Louis are shown in Figures C.9-10 and Figures 4.28-29. The major feature of the surface ground motion is the lack of amplification for periods greater than 0.5 sec because of the thin soil layer. At the 10% in 50 years level, nine ground motions come from events of magnitude 6 to 7. At 2% in 50 years level, six come from magnitude-8 events with long duration. Smaller (magnitude 5 to 7) and closer events with shorter duration make up the rest. This observation agrees well with the USGS deaggregation results, which indicate that 68.6% of the hazard is from magnitude 8 events at 1-sec period and 36% contribution for the 0.2-sec period. Since the median spectrum has a better match with the target value in the long period range, the composition of uniform hazard ground motion suite agrees better with the USGS contribution percentage at 1-sec period. There is comparatively a much larger scatter at the peak of the response spectra (period from 0.1 to 0.2 sec). The maximum 16-to-84 percentile band for the median, however, is still around 10%. The ground motions and response spectra for rock outcrop (or bedrock) are shown in Figures C.11-12 and Figures 30-31, respectively. The compositions of the suites are similar and the ground motion levels are lower than those of the representative soil site condition.

4.4 Final Remarks

As mentioned in the foregoing, the source and path models and the quarter-wavelength method do not explicitly consider effects of surface waves (e.g., Dorman and Smalley 1994) and soil nonlinearity. Therefore, the change in frequency content with time and the nonlinear soil amplification of the ground motions are not modeled in this
simulation. However, it is pointed out that comparisons of results by Boore and Joyner (1991, 1996, 1997) with observations and analytical results generally show the robustness of their methods. Also there have not been any efficient methods of modeling surface waves and nonlinear soil effects that can be adapted in a large-scale simulation as required in this study. In addition, the uniform hazard response spectra based on the simulated ground motions in Memphis and Carbondale compare favorably with those of 1997 USGS national earthquake hazard maps and the FEMA 273 recommendations. The response spectra are the most important measure of ground motion potential of causing severe structural response and damage. The UHRS and suites of simulated ground motions generated by the proposed method, therefore, represent reasonably well the seismic hazards to buildings and other structures in these three cities. As efficient methods for modeling soil nonlinearity and surface waves become available, they can be incorporated into the simulation method.
Table 4.1 Conversion of frequently used annual and 50-year probabilities of exceedance.

<table>
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<th>50-year PE</th>
<th>Annual PE</th>
<th>Mean Return Period (years)</th>
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</thead>
<tbody>
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<td>50%</td>
<td>0.01376730</td>
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</tr>
<tr>
<td>10%</td>
<td>0.00210499</td>
<td>475.06</td>
</tr>
<tr>
<td>5%</td>
<td>0.00102534</td>
<td>975.29</td>
</tr>
<tr>
<td>2%</td>
<td>0.00040397</td>
<td>2475.42</td>
</tr>
</tbody>
</table>

VISION 2000 PERFORMANCE OBJECTIVES

BUILDING PERFORMANCE LEVELS

Figure 4.1 VISION 2000 performance objectives (after Hamburger 1997).
$T_n = 1.0$ sec, Representative soil profile, Memphis (TN)

Figure 4.2 Comparison of the predicted annual probability of exceeding values of the elastic force coefficient using Equation (4.4) with the simulation results for a 1.0-sec period SDOF system at representative soil profile, Memphis, TN ($v_l = 0.001492$, $k_1 = 1.167$, $v_2 = 0.429$, $k_2 = 1.863$, $u_c = 1.687$, $k_c = 3.072$, and $w = 0.0006984$).
strength deterioration $= \frac{\delta P}{P_u} \times 100\% = \frac{|P'_u - P_u|}{P_u} \times 100\%$

stiffness degradation $= \frac{\delta k}{k_0} \times 100\% = \frac{|k'_0 - k_0|}{k_0} \times 100\%$

Figure 4.3 Baber-Wen smooth hysteresis model.
Figure 4.4 Smooth hysteresis model: (a) nondegrading system, (b) degrading system with 5% strength and stiffness degradations.
Figure 4.5 Uniform hazard response spectra for B/C boundary at Memphis and comparison with USGS national hazard maps results.

Figure 4.6 Uniform hazard response spectra for representative soil profile at Memphis, TN and comparison with FEMA 273 response spectra.
Figure 4.7 Uniform hazard response spectra for representative soil profile at Carbondale, IL and comparison with FEMA 273 response spectra.

Figure 4.8 Uniform hazard response spectra for representative soil profile at St. Louis, MO and comparison with FEMA 273 response spectra.
Figure 4.9 Comparison of the linear elastic uniform hazard response spectra ($\xi_d = 5\%$) of Santa Barbara obtained from this study with those of L.A. for (a) soft rock (Collins et al., 1995), (b) B/C boundary or firm rock (USGS, 1997).
Figure 4.10 Nonlinear uniform hazard response spectra of nondegrading systems at representative soil condition, Memphis, TN and $\xi_d = 2\%$ and $\alpha = 3\%$ are assumed, (a) Exceedance probability = 10\% in 50 years, (b) Exceedance probability = 5\% in 50 years, (c) Exceedance probability = 2\% in 50 years.
Figure 4.11 Nonlinear uniform hazard response spectra of degrading systems (5% stiffness degradation and 5% strength degradation) at representative soil condition, Memphis, TN and $\xi_d = 2\%$ and $\alpha = 3\%$ are assumed, (a) Exceedance probability = 10% in 50 years, (b) Exceedance probability = 5% in 50 years, (c) Exceedance probability = 2% in 50 years.
Figure 4.12 Nonlinear uniform hazard response spectra of degrading systems (10% stiffness degradation and 5% strength degradation) at representative soil condition, Memphis, TN and $\xi_d = 2\%$ and $\alpha = 3\%$ are assumed, (a) Exceedance probability = 10% in 50 years, (b) Exceedance probability = 5% in 50 years, (c) Exceedance probability = 2% in 50 years.
Figure 4.13 Nonlinear uniform hazard response spectra of nondegrading systems at representative soil condition, Carbondale, IL and $\xi_d = 2\%$ and $\alpha = 3\%$ are assumed, (a) Exceedance probability = 10\% in 50 years, (b) Exceedance probability = 5\% in 50 years, (c) Exceedance probability = 2\% in 50 years.
Figure 4.14 Nonlinear uniform hazard response spectra of nondegrading systems at representative soil condition, St. Louis, MO and $\xi_d = 2\%$ and $\alpha = 3\%$ are assumed, (a) Exceedance probability = 10\% in 50 years, (b) Exceedance probability = 5\% in 50 years, (c) Exceedance probability = 2\% in 50 years.
Figure 4.15 Nonlinear uniform hazard response spectra of nondegrading systems in which $\xi_d = 2\%$, $\alpha = 3\%$ and soft rock site condition are assumed for Santa Barbara, (a) Exceedance probability = 50\% in 50 years, (b) Exceedance probability = 10\% in 50 years, (c) Exceedance probability = 2\% in 50 years.
Figure 4.16 Nonlinear uniform hazard response spectra of degrading systems (5% stiffness degradation and 5% strength degradation) in which $\xi_d = 2\%$, $\alpha = 3\%$ and soft rock site condition are assumed for Santa Barbara, (a) Exceedance probability = 50% in 50 years, (b) Exceedance probability = 10% in 50 years, (c) Exceedance probability = 2% in 50 years.
Figure 4.17 Locations (epicenter, focal depth) and magnitudes of selected uniform hazard ground motions at two hazard levels for representative soil profile of Memphis, TN (solid circles for 10% and hollow circles for 2% in 50 years).
Figure 4.18 Locations (epicenter, focal depth) and magnitudes of selected uniform hazard ground motions at two hazard levels for representative soil profile of Carbondale, IL (solid circles for 10% and hollow circles for 2% in 50 years).
Figure 4.19 Locations (epicenter, focal depth) and magnitudes of selected uniform hazard ground motions at two hazard levels for representative soil profile of St. Louis, MO (solid circles for 10% and hollow circles for 2% in 50 years).
Figure 4.20  Response spectra of 10% in 50 years ground motion suite for representative soil profile and comparison with target UHRS, Memphis, TN.
Figure 4.21 Response spectra of 2% in 50 years ground motion suite for representative soil profile and comparison with target UHRS, Memphis, TN.
Figure 4.22 Response spectra of 10% in 50 years ground motion suite for bedrock (hard rock) and comparison with target UHRS, Memphis, TN.
Figure 4.23 Response spectra of 2% in 50 years ground motion suite for bedrock (hard rock) and comparison with target UHRS, Memphis, TN.
Figure 4.24 Response spectra of 10% in 50 years ground motion suite for representative soil profile and comparison with target UHRS, Carbondale, IL.
Figure 4.25 Response spectra of 2% in 50 years ground motion suite for representative soil profile and comparison with target UHRS, Carbondale, IL.
Figure 4.26 Response spectra of 10% in 50 years ground motion suite for bedrock (hard rock) and comparison with target UHRS, Carbondale, IL.
Figure 4.27 Response spectra of 2% in 50 years ground motion suite for bedrock (hard rock) and comparison with target UHRS, Carbondale, IL.
Figure 4.28 Response spectra of 10% in 50 years ground motion suite for representative soil profile and comparison with target UHRS, St. Louis, MO.
Figure 4.29 Response spectra of 2% in 50 years ground motion suite for representative soil profile and comparison with target UHRS, St. Louis, MO.
Figure 4.30 Response spectra of 10% in 50 years ground motion suite for bedrock (hard rock) and comparison with target UHRS, St. Louis, MO.
Figure 4.31 Response spectra of 2% in 50 years ground motion suite for bedrock (hard rock) and comparison with target UHRS, St. Louis, MO.
CHAPTER 5
IMPLICATIONS IN RELIABILITY AND DESIGN

5.1 Overview

As demonstrated in the foregoing, the median response spectra of linear systems to the simulated ground motion suites closely match the target elastic spectral accelerations for a wide range of frequencies. However, whether these ground motion suites can provide satisfactory estimate of response of inelastic, nonlinear systems has important implications in performance check of buildings and structures. In addition, it is of interest to practicing engineers whether the ground motion suites can provide accurate estimate of structure performance without excessive computational efforts. These practical implications of the uniform hazard ground motion are examined. The effect of system degradation on ductility reduction factor which is useful in both analysis and design of structures in inelastic range is also examined.

5.2 Nonlinear Response Estimate by Uniform Hazard Ground Motion Suites

To verify whether 10 uniform hazard ground motions can reasonably represent the seismic hazard at a given probability level for nonlinear systems, one can compare the inelastic response spectra constructed from these 10 simulated ground motions and the entire population of 9000. In Section 4.2.2.2, uniform hazard response spectra for nonlinear systems are obtained for ductility ratios from 2 to 8. An iterative procedure is used to determine the yield capacity of a SDOF system such that ductility under the 10 uniform hazard ground motions satisfies the target value. For an inelastic SDOF system of a given fundamental period, an initial yield displacement is assumed and the maximum displacement and ductility are calculated. The yield displacement is then modified and
the trial and error procedure is repeated until the ductility converges to the target value within a very small error limit (0.01 is used in this study). The median value of the yield force coefficients under the 10 uniform hazard ground motions (Equation (4.10)) is then calculated. In Figures 5.1-4 the median value and the 16th to 84th percentile range are compared with the nonlinear response spectra for both degrading and non-degrading systems obtained previously from 9000 ground motions. It is seen that median inelastic spectral accelerations match the spectral values calculated in Section 4.2.2.2 closely. In performance evaluation of a given nonlinear system, such iterations are, of course, not required. For such a system, the median response of 10 time history response analyses provides an accurate estimate of the system performance. The results imply that, one can use 10 uniform hazard ground motions instead of 9000 for an accurate estimate of the nonlinear structural responses corresponding to each probability level. Verification is not performed here for nonlinear MDOF systems due to the large computational effort required of such systems. However, since the response spectra calculated from the 10 uniform hazard ground motions match the target values for a wide range of frequency, these motions are expected to yield response of nonlinear MDOF systems with good accuracy. They represent realistically the future seismic threat to the site location from causative faults, which can be clearly seen from the comparison of results with the deaggregation method in Section 4.3.2.

5.3 Ductility Reduction Factor

For design purposes, it is impractical to provide in seismic design codes both linear elastic and nonlinear inelastic uniform hazard response spectra for various values of structural parameters (e.g., damping and strain-hardening ratio) and for a large number of sites in the U.S. A common method to consider the inelastic response behavior is the ductility reduction factor $R_\mu$, by which the strength of a nonlinear system can be obtained that of the elastic system. The ductility reduction factor $R_\mu$ is defined as:
\[ R_\mu = \frac{C_e}{C_y} \] (5.1)

where \( C_e \) is the elastic force coefficient (Equation(4.3)) and \( C_y \) is the yield force coefficient (Equation(4.10)). Effects of local site soil condition, inelastic behavior of the structure (e.g. strain-hardening ratio), damping ratio, and fundamental period of the structure have been investigated by Krawinkler and Nassar (1992), Miranda and Bertero (1994), Riddell (1995), Krawinkler (1996), Shi (1997) and Han et al. (1999). The empirical formula proposed by Krawinkler and Nassar (1992) is of the following form:

\[ R_\mu = \left[ c(\mu - 1) + 1 \right]^{\psi c} \] (5.2)

where

\[ c(T_n, \alpha) = \frac{T_n^a}{1 + T_n^a} + \frac{b}{T_n} \] (5.3)

where a damping ratio of 5% is assumed; \( T_n \) is structural period; \( \mu \) is displacement ductility; \( \alpha \) is the post-to-preyield stiffness ratio. The regression parameters \( a \) and \( b \) were obtained for different post-to-preyield stiffness ratios as follows:

- \( \alpha = 0\%: \quad a = 1.00 \quad b = 0.42 \)
- \( \alpha = 2\%: \quad a = 1.00 \quad b = 0.37 \)
- \( \alpha = 10\%: \quad a = 1.00 \quad b = 0.29 \)

The results from the aforementioned studies indicate that ductility reduction factor \( R_\mu \) generally depends on natural period, soil condition and degree of degradation. The effect of exceedance probability level on \( R_\mu \) has been investigated by Collins et al. (1996) and found to be unimportant. The ductility reduction factor based on simulated ground motions in Mid-America and Santa Barbara will be examined and compared with previous empirical results based on West Coast data (Equations (5.2) and (5.3)).

In Figure 5.5-10, the ductility factor \( R_\mu \) of the nondegrading and degrading systems for three exceedance probability levels (solid lines) are shown. It is seen that \( R_\mu \) is insensitive to change in exceedance probability levels, as shown in Collins et al. (1996).
There is a significant reduction of $R_{ij}$ values due to system degradation at Santa Barbara, CA (Figures 5.9-10) but not at Memphis, TN (Figures 5.5-6). This may be attributed to the effects of ground motion duration, frequency content and attenuation for each site. The Santa Barbara results agree with Han et al. (1999), which are largely based on West Coast data. Also shown for comparison are empirical formulae by Krawinkler and Nassar (1992) and Miranda and Bertero (1994). Note that Krawinkler and Nassar (1992) and Miranda and Bertero (1994) did not include degradation in their empirical models. Since the simulation results fit better the Krawinkler-Nassar formula, their empirical equation is used.

A regression analysis is performed to obtain the $a$ and $b$ values (Table 5.1) and the regression results are compared with Krawinkler-Nassar curves in Figures 5.11-16. The agreements are generally good. Krawinkler and Nassar (1992), however, found that for MDOF systems an increase in the strength is necessary to meet the drift requirements of current model building codes due to the higher mode contribution. A more thorough and systematic investigation of MDOF systems, therefore, is needed before a general reduction factor for degrading systems can be developed. In addition, it is observed in Figure 5.14 for St. Louis, that they are much higher than the Krawinkler-Nassar curve in the period range below 0.25 seconds due to the thin soil layer in St. Louis. It indicates that the ductility reduction factor is highly site-dependent which needs to be taken into consideration carefully.

$R_{ij}$ factors in current code procedures allow one to obtain nonlinear structural responses via a linear static analyses. It is a computationally efficient method commonly used by practicing engineers provided $R_{ij}$ is correctly calibrated with respect to building types. As shown by Wen and Song (1999), $R_{ij}$ can also be used to construct probabilistic performance (fragility) curves for buildings, e.g. a single story steel moment frame in Carbondale, Illinois in Figure 5.17. The small solid diamonds are drift ratios calculated by nonlinear time history analyses at a given probability level and the hollow circles are the median drift ratios. One can perform first a linear static analysis of the structure and then use the UHRS and the empirical $R_{ij}$ to evaluate the drift ratio. The probabilistic
Peak Ground Acceleration versus Spectral Acceleration in Fragility Analysis

Fragility analyses are commonly used for evaluation of structural performance and loss estimation during future seismic events (e.g., Wen and Song 1999, Abrams and Shinozuka 1997). Both peak ground acceleration \((PGA)\) and spectral acceleration \((S_a)\) have been commonly used as the measure of earthquake intensity in the fragility analysis. They may yield significant different results. The accuracy of these two methods is investigated by considering the probability of limit state given by:

\[
P(Y > y) = \int_0^\infty G_{\gamma,\lambda}(Y > y \mid x) \cdot h(x) \, dx
\]

where \(P(Y > y)\) is the probability of structural response \(Y\) exceeding a given limit \(y\); \(x\) is the earthquake intensity, measured by \(S_a\) or \(PGA\); \(G_{\gamma,\lambda}(Y > y \mid x)\) is the conditional response probability given the hazard intensity, or commonly referred to as the fragility curve; \(h(x)\) is the seismic hazard probability density at the site (Equation(4.4)). To evaluate the conditional probability, the following equation is used:

\[
Y = \alpha \cdot X^\beta \cdot \varepsilon
\]

where \(Y\) is the structural response; \(X\) is the earthquake intensity; \(\alpha\) and \(\beta\) are the regression parameters; and \(\varepsilon\) is the error term with median equal to 1 and standard deviation equal to \(\delta_\varepsilon\). The conditional response probability function \(G_{\gamma,\lambda}(Y > y \mid x)\) is described by lognormal distribution. The fragility obtained from \(S_a\) and \(PGA\) are compared with the "exact" solution based on 9000 simulations.

To compare two approaches, we assume that 100 ground motions with \(PGA\) between 0.15 ~ 1.15 g are available at the site and are randomly selected from the population of simulated motions at Memphis. Two nondegrading SDOF systems of a
natural period 0.3-sec and 1.0-sec are used. The ductility ratio is used for the measure of structural response $Y$. The prediction of ductility ratio as a function of spectral acceleration or PGA is shown in Figures 5.20 to 5.25. The regression parameters $\alpha$, $\beta$ and scatter measure $\delta_e$ are listed in Table 5.2. The hazard functions corresponding to 0.3-sec and 1.0-sec spectral accelerations and PGA are shown in Figure 5.18, Figure 4.2 and Figure 5.19, respectively. While PGA may be used as a reliable response measure of short period buildings (e.g., a coefficient of variation (COV) of about 27% at 0.3-sec), its accuracy drops dramatically for long period buildings (e.g., the COV increases to 108% at 2.0-sec). On the other hand, the response COV remains almost constant at 27% when the spectral acceleration is used. When compared the “exact” solutions (Figures 5.26-28), $S_a$ gives excellent estimates except at the low exceedance probability level due to the limitation of the power law regression. Other mathematical regression form may be used to improve the accuracy. On the other hand, PGA gives poor results for intermediate to long period structures (Figures 5.27-28). In light of the severe shortcomings, PGA is therefore not recommended for the evaluation of structural fragility.

5.5 Final Remarks

It is shown that uniform hazard ground motions provide an unbiased estimate of nonlinear structural dynamic response for SDOF systems. Since they match the target spectra over a wide range of frequencies, uniform hazard ground motions may be used for MDOF systems as well. Secondly, system degradation is shown to have significant influence on the ductility reduction factor for Santa Barbara and is recommended for further study for nonlinear MDOF systems. An efficient method to estimate structural probabilistic performance using uniform hazard response spectra, ductility reduction factor and a lognormal probability fit is demonstrated. Using spectral accelerations to estimate structural fragility (Shome and Cornell, 1999) is shown to yield satisfactory results as well.
Table 5.1  Regression parameters for ductility reduction factor using the Krawinkler-Nassar formula (Equations 5.2 and 5.3).

<table>
<thead>
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<th>Memphis (TN)</th>
<th>Carbondale (IL)</th>
<th>St. Louis (MO)</th>
<th>Santa Barbara (CA)</th>
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<tr>
<td>(a)</td>
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<td>(b)</td>
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<td></td>
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Table 5.2  Regression parameters for \(S_a - Y\) and \(PGA - Y\) relations.

<table>
<thead>
<tr>
<th></th>
<th>(T = 0.3) sec, (C_y = 0.37)</th>
<th>(T = 1.0) sec, (C_y = 0.15)</th>
<th>(T = 2.0), (C_y = 0.07)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(Sa)</td>
<td>PGA</td>
<td>(Sa)</td>
</tr>
<tr>
<td>(a)</td>
<td>3.88</td>
<td>14.60</td>
<td>7.21</td>
</tr>
<tr>
<td>(\beta)</td>
<td>1.26</td>
<td>1.44</td>
<td>1.00</td>
</tr>
<tr>
<td>(\delta_\varepsilon) (COV)</td>
<td>0.265 (0.270)</td>
<td>0.261 (0.266)</td>
<td>0.269 (0.274)</td>
</tr>
</tbody>
</table>

P.S. (1) \(\delta_\varepsilon\) is standard deviation of the error term \(\varepsilon\); \(\delta_\varepsilon = \sigma_{\ln Y S_a}\) or \(\sigma_{\ln Y PGA}\).

(2) COV is the coefficient of variation of structural response \(Y\). For quick reference, generally, \(\delta_\varepsilon \approx \text{COV}\) when \(\delta_\varepsilon < 0.30\).
Figure 5.1 Comparison of spectral accelerations calculated from 10 ground motions with target values at 10% in 50 years probability level for nondegrading SDOF systems, representative soil profile in Memphis, TN.
Comparison of spectral accelerations calculated from 10 ground motions with target values at 2% in 50 years probability level for nondegrading SDOF systems, representative soil profile in Memphis, TN.
Figure 5.3  Comparison of spectral accelerations calculated from 10 ground motions with target values at 10% in 50 years probability level for degrading SDOF systems, representative soil profile in Memphis, TN.
Figure 5.4  Comparison of spectral accelerations calculated from 10 ground motions with target values at 2% in 50 years probability level for degrading SDOF systems, representative soil profile in Memphis, TN.
Figure 5.5 Comparison of ductility reduction factors of nondegrading systems for uniform hazard response spectra (this study) with empirical formulae by Krawinkler and Nassar (1992) and Miranda and Bertero (1994); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), Memphis TN.
Figure 5.6  Comparison of ductility reduction factors of degrading systems for uniform hazard response spectra (this study) with empirical formulae by Krawinkler and Nassar (1992) and Miranda and Bertero (1994); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), Memphis TN.
Figure 5.7  Comparison of ductility reduction factors of nondegrading systems for uniform hazard response spectra (this study) with empirical formulae by Krawinkler and Nassar (1992) and Miranda and Bertero (1994); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), Carbondale IL.
Figure 5.8  Comparison of ductility reduction factors of nondegrading systems for uniform hazard response spectra (this study) with empirical formulae by Krawinkler and Nassar (1992) and Miranda and Bertero (1994); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), St. Louis MO.
Figure 5.9 Comparison of ductility reduction factors of nondegrading systems for uniform hazard response spectra (this study) with empirical formulae by Nassar and Krawinkler (1992) and Miranda and Bertero (1994); (a) target ductility ratio = 2, (b) target ductility ratio = 4, (c) target ductility ratio = 6, (d) target ductility ratio = 8 ($\xi=5\%$ and $\alpha=3\%$ are assumed for all cases), Santa Barbara CA.
Figure 5.10 Comparison of ductility reduction factors of degrading systems for uniform hazard response spectra (this study) with empirical formulae for nondegrading systems by Nassar and Krawinkler (1992) and Miranda and Bertero (1994); (a) target ductility ratio = 2, (b) target ductility ratio = 4, (c) target ductility ratio = 6, (d) target ductility ratio = 8 ($\xi=2\%$ and $\alpha=3\%$ are assumed for all cases), Santa Barbara CA.
Figure 5.11 Regression ductility reduction factors of nondegrading systems using the Krawinkler-Nassar empirical formula and comparison with the original Krawinkler and Nassar curves (1992); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), Memphis TN.
Figure 5.12 Regression ductility reduction factors of degrading systems using the Krawinkler-Nassar empirical formula and comparison with the original Krawinkler and Nassar curves (1992); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), Memphis TN.
Figure 5.13 Regression ductility reduction factors of nondegrading systems using the Krawinkler-Nassar empirical formula and comparison with the original Krawinkler and Nassar curves (1992); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), Carbondale IL.
Figure 5.14 Regression ductility reduction factors of nondegrading systems using the Krawinkler-Nassar empirical formula and comparison with the original Krawinkler and Nassar curves (1992); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), St. Louis MO.
Figure 5.15 Regression ductility reduction factors of nondegrading systems using the Krawinkler-Nassar empirical formula and comparison with the original Krawinkler and Nassar curves (1992); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 (\(\xi_d = 2\% \) and \(\alpha = 3\%\) are assumed for all cases), Santa Barbara CA.
Figure 5.16 Regression ductility reduction factors of degrading systems using the Krawinkler-Nassar empirical formula and comparison with the original Krawinkler and Nassar curves (1992); target ductility ratio (a) = 2, (b) = 4, (c) = 6, (d) = 8 ($\xi_d = 2\%$ and $\alpha = 3\%$ are assumed for all cases), Santa Barbara CA.
Figure 5.17 Fragility curves for 1-story standard buildings before/after retrofit in Carbondale, IL (after Wen and Song, 1999).
$T_n = 0.3 \text{ sec}, \text{ Representative soil profile, Memphis (TN)}$

![Graph showing the comparison of the predicted annual probability of exceeding values of the elastic force coefficient using Equation (4.4) with the simulation results for a 0.3-sec period SDOF system at representative soil profile, Memphis, TN. Parameters used in the simulation: $v_l = 0.00485$, $k_l = 1.031$, $v_c = 1.627$, $k_c = 1.737$, and $w = 0$.]

Figure 5.18 Comparison of the predicted annual probability of exceeding values of the elastic force coefficient using Equation (4.4) with the simulation results for a 0.3-sec period SDOF system at representative soil profile, Memphis, TN ($v_l = 0.00485$, $k_l = 1.031$, $v_c = 1.627$, $k_c = 1.737$, and $w = 0$).
Figure 5.19 Comparison of the predicted annual probability of exceeding values of peak ground acceleration using Equation (4.4) with the simulation results representative soil profile, Memphis (TN) ($v_I = 0.002, k_I = 1.07, \nu_2 = 0.3, k_2 = 1.90, \nu_c = 0.9, k_c = 1.20$, and $w = 0.0002$).
Figure 5.20 Regression of ductility ratio on median spectral acceleration through 100 simulated ground motions (nondegrading SDOF with \(T = 0.3\) sec and \(C_y = 0.37\) g).

Figure 5.21 Regression of ductility ratio on median peak ground acceleration through 100 simulated ground motions (nondegrading SDOF with \(T = 0.3\) sec and \(C_y = 0.37\) g).
Figure 5.22 Regression of ductility ratio on median spectral acceleration through 100 simulated ground motions (nondegrading SDOF with $T = 1.0$ sec and $C_y = 0.15$ g).

Figure 5.23 Regression of ductility ratio on median peak ground acceleration through 100 simulated ground motions (nondegrading SDOF with $T = 1.0$ sec and $C_y = 0.15$ g).
Figure 5.24 Regression of ductility ratio on median spectral acceleration through 100 simulated ground motions (nondegrading SDOF with $T = 2.0$ sec and $C_y = 0.07$ g).

Figure 5.25 Regression of ductility ratio on median peak ground acceleration through 100 simulated ground motions (nondegrading SDOF with $T = 2.0$ sec and $C_y = 0.07$ g).
Figure 5.26 Comparison of using spectral acceleration and peak ground acceleration in fragility analysis (Shome and Cornell, 1999) with "exact" solution (a 0.3-sec nondegrading SDOF system is used).
Figure 5.27 Comparison of using spectral acceleration and peak ground acceleration in fragility analysis (Shome and Cornell, 1999) with "exact" solution (a 1.0-sec nondegrading SDOF system is used).
Figure 5.28 Comparison of using spectral acceleration and peak ground acceleration in fragility analysis (Shome and Cornell, 1999) with "exact" solution (a 2.0-sec nondegrading SDOF system is used).
CHAPTER 6
SUMMARY AND CONCLUSIONS

6.1 Summary and Conclusions

A simulation method is proposed to generate uniform hazard response spectra and ground motions. It allows efficient evaluation of structural performance and fragility analysis for loss estimation under future earthquakes. The seismotectonic parameters used in this study to simulate earthquakes are largely based on USGS OFR-96-532 for Mid-America and the 1995 WGCEP report for western United States. The method can be updated as more is known of the seismotectonic features such as in the national seismic hazard maps updating effort at a 3-year interval by U.S. Geological Survey. 9000 ground motions are generated at three Mid-America cities (Memphis, TN, St. Louis, MO, Carbondale, IL) and 1815 in Santa Barbara, CA according to the regional seismicity, from which uniform hazard response spectra are constructed. A least-square procedure is then proposed to select 10 uniform hazard ground motions for a given probability level at each site, whose median spectral accelerations match the target response spectra corresponding to 10% and 2% in 50 years probability of exceedance. Ground motions for both rock sites and soil sites with a given soil profile are generated. Based on the results, the following conclusions are drawn:

1. A structural period independent procedure is used to select uniform hazard ground motions from a large pool of simulated ground motions. Results of investigation of various nonlinear SDOF systems (e.g. structural period, deterioration) indicate that these ground motions can be used for unbiased estimates of structural responses with small uncertainty (standard error). Since uniform hazard ground motions match the target response spectra for a wide
range of frequencies, they can be used for reliable response estimates of MDOF systems as well.

2. Since only one component of earthquake motion is generated and the stochastic source model is based on field observations of shear waves, the uniform hazard ground motions provided in this study are suitable for structural time history analyses of standard frame buildings with short to intermediate natural periods (0.2 – 2 sec) and no significant 3-dimensional and torsional motions.

3. In all three Mid-America cities, small to medium size earthquakes have major contribution to the hazard at the 10% in 50 years level. At the 2% in 50 years level seismic hazard in Memphis and Carbondale is dominated by magnitude 8 earthquakes in the NMSZ, while in St. Louis small to medium size earthquakes from various distances also contribute. These observations are in general agreement with USGS deaggregation results.

4. Due to the influence of local site condition, spectral shape varies considerably among three Mid-America cities. In Memphis and Carbondale, there are significant spectral accelerations in the intermediate to long period range, which is more damaging to medium- to high-rise buildings. In St. Louis, on the other hand, large spectral accelerations are primarily within short period range, which is more damaging to low-rise buildings. In Santa Barbara, the frequency contents of the uniform hazard spectra indicate that medium- and high-rise buildings are more vulnerable.

5. The ductility reduction factor $R_d$ calculated from the simulated ground motions for Memphis, Carbondale and Santa Barbara is shown to be in general agreement with results from recorded earthquake ground motions by Krawinkler and Nassar (1992) and Miranda and Bertero (1994). One can construct probabilistic performance (fragility) curves using $R_d$ factors within the framework of current spectra-based code procedures (e.g. 1997 NEHRP) without having to do a large number of nonlinear time history analyses.
6. Spectral acceleration is a more reliable measure of structural responses and therefore should be used for evaluation of structural fragility. Peak ground acceleration is a poor measure of long period structure responses and is not recommended for fragility analysis.

6.2 Recommendations for Future Studies

The simulation methodology proposed herein provides a basic framework for constructing uniform hazard response spectra and ground motions. However, there are several issues that require further investigation:

1. The tectonic and seismological information in the CEUS is insufficient for an accurate estimate of seismic hazard at a low probability level (e.g. 2% in 50 years). Large magnitude (7.5 to 8) earthquakes in the New Madrid seismic zone are generally dominant at this level but their occurrence rates remain a controversial topic (Johnston 1996b, Newman et al. 1999, Mueller et al. 1999). With more geodetic, geologic and seismic information, the proposed simulation methodology can be refined to generate more realistic ground motions.

2. Soil nonlinearity is important for high intensity earthquakes but has not been considered in this study. Basically, the quarter wavelength method considers only elastic soil properties, whereas the program SHAKE accounts for inelastic soil properties via an equivalent linear model, which can not adequately account for period shifting and response amplification due to soil nonlinearity, and is best suited to a shallow soil deposit. Although there are truly nonlinear soil analysis software available (e.g. D-Mod_D, CyberQuake, etc.), due to very limited information on soil boring log data, more verification study is still needed before a reliable nonlinear soil model can be used (Hashash and Park, 1999).

3. The phenomenological stochastic model does not consider surface waves, which generally have long period motions and cause large responses in long
period structures, such as bridges and high-rise buildings. Also, when a structure deteriorates under earthquake excitation, its fundamental period lengthens and as a result is more vulnerable to surface waves. To include surface waves, a wave propagation model such as in Saikia and Somerville (1997) needs to be used.

4. This study does not consider 2- or 3-component ground motion simulation, which is important for investigation of 3-dimensional frame building behavior such as torsion effects, biaxial interaction, etc. (Wang and Wen, 1998).

5. For lifeline engineering (bridges, pipelines, etc) and large-scale structures, spatial variation of ground motions needs to be considered. For this purpose, ground motion coherence function is required in the stochastic approach to account for differential movement due to phase delay.

6. In reality, a large number of aftershocks often occur within a period of one to two months after a large magnitude main shock. In such a short period, many of the damaged buildings are unlikely to be repaired to survive the aftershocks, e.g. the 1985 \( M_w-8 \) Michoacan (Mexico) and the 1999 \( M_L-7.3 \) Chi-Chi (Taiwan) earthquakes, etc. For essential and hazardous facilities, this consideration becomes even more important since they need to be functional after a damaging earthquake. It is reasonable to consider aftershocks if the main shock has a magnitude greater than 7 (e.g., at the 2% in 50 years hazard level).

7. System degradation has important effects on ductility reduction factors and therefore needs to be considered in the design code. Further investigation on MDOF systems is recommended to avoid under-design when spectra-based design procedure is used.
APPENDIX A
TRUNCATED NORMAL DISTRIBUTION

Lognormal distribution generally has a long tail. To avoid physically unrealistic values in large-scale simulations, cut-off limits are necessary. The lognormal distribution is related to the standard normal probability density function (PDF) by a logarithmic transformation (Ang and Tang, 1990). To avoid sharp cutoff limits with discontinuity, the following modified standard normal probability density function is used, which gives smooth truncations (Loh and Jean, 1994):

\[
\mathcal{N}(x) = \begin{cases} 
\frac{1}{\sqrt{2\pi}} \exp\left(-\frac{x^2}{2}\right) + A \cos(\gamma x) + B & -\varepsilon_c < x < \varepsilon_c \\
0 & |x| > \varepsilon_c
\end{cases}
\] (A.1)

where \(\varepsilon_c\) is the cut-off limit; \(A, B\) and \(\gamma\) are unknown constants and must satisfy the following boundary conditions:

\[
\mathcal{N}(x = -\varepsilon_c) = \mathcal{N}(x = \varepsilon_c) = 0 \\
\left[\frac{d\mathcal{N}(x)}{dx}\right]_{x=\varepsilon_c} = 0 \\
\int_{-\varepsilon_c}^{\varepsilon_c} \mathcal{N}(x)dx = 1
\] (A.2.1, A.2.2, A.2.3)

In this study, \(\varepsilon_c = 3\) is assumed, considering the large uncertainty due to lack of field records in the CEUS. Solving the above equations, one obtains \(A = 0.01351\), \(B = 0.00422\), and \(\gamma = 13467\). The resulting distribution is shown in Figure A.1 and compared with the standard normal PDF. To generate the corresponding truncated lognormal random numbers, an inverse transform method is employed; details can be referred to Ang and Tang (1990).
Figure A.1 Truncated normal probability density function.
APPENDIX B
MODIFIED TYPE II EXTREME VALUE DISTRIBUTION FUNCTION

To obtain spectral acceleration according to a prescribed hazard level, one can construct the hazard curve by running a large number of linear/nonlinear structural analyses and then calculate the required spectral value through interpolation without regression analysis. In doing so, a large computational effort is required, especially for nonlinear analysis. To alleviate the computational burden, this study performs a moderate number of structural analyses and then constructs hazard curves via a curve fitting technique.

To determine the tail behavior of simulated results in this study, a generalized extreme value distribution function proposed by Maes and Breitung (1993) was first used. It was found that a Type II distribution fits the simulated data the best, which had been observed by earthquake researchers in the past. However, because of the cut-off limits in magnitude and attenuation and also a distribution gap between $M_w$ 7.5 and 8, the resulting distribution is deviate from standard Type II tail behavior. This is especially true for probability lower than 5% in 50 years. In view of this, the tail of the distribution is modified as follows:

$$P_{ex}(C_f) = \left\{ 1 - p_0 - w \right\} \left[ 1 - \exp \left( - \frac{\nu_1}{C_f} \right)^{k_1} \right] + \left[ w \cdot \left[ 1 - \exp \left( - \frac{\nu_2}{C_f} \right)^{k_2} \right] \right] \cdot \left[ 1 - \exp \left( - \frac{\nu_c}{C_f} \right)^{k_c} \right]$$

(B.1)

where $C_f$ is design force coefficient (i.e. $C_f = C_e$ in elastic SDOF, $C_f = C_y$ in nonlinear SDOF); $P_{ex}(C_f)$ indicates annual exceedance probability; $p_0$ is annual probability of earthquakes with magnitude less than $m_b$ 5; $w$ is a weight parameter for the influence of magnitude 8 earthquakes; $\nu_1$, $\nu_2$ and $\nu_c$ are corner parameters; $k_1$, $k_2$ and $k_c$ are slope parameters. Subscript 1 indicates the influence of earthquakes smaller than magnitude 7.5; subscript 2 indicate the influence of magnitude 8 earthquakes; subscript c indicates
the influence of cut-off limits. When $w = 0$ and $\nu_c \to \infty$ (i.e. $\nu_c \gg \max(C_e)$), Equation (B.1) follows the original Type II extreme value distribution (Ang and Tang, 1990). When $w = 0$, there is no magnitude gap and therefore no sag segment in the exceedance probability. When $\nu_c \to \infty$, it means no cut-off limits introduced in the simulation and therefore no secondary slope. The goodness of fit using this generalized exceedance probability function can be clearly seen in Figure B.1 and parameter values are listed in Table B.1.

Table B.1 Parameters for the modified extreme value distribution function describing the annual probability of exceeding a target ductility of 4 as a function of yield force coefficient in a case of nondegrading SDOF systems at representative soil site, Memphis, TN.

<table>
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<th>Period</th>
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<th>$k_1$</th>
<th>$\nu_2$</th>
<th>$k_2$</th>
<th>$\nu_c$</th>
<th>$k_c$</th>
<th>$w$</th>
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<td>—</td>
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<td>0.278</td>
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Figure B.1 Comparison of the predicted annual probability of exceeding values of the yield force coefficient with the simulation results for nondegrading SDOF systems at representative soil profile, Memphis, TN. ($\xi_f = 2\%$, $\alpha = 3\%$, $n = 5$, target ductility = 4).
Figure B.1 (continued).
APPENDIX C

UNIFORM HAZARD GROUND MOTIONS FOR MID-AMERICA CITIES
Figure C.1  Suite of 10% in 50 years ground motions for representative soil profile, Memphis, TN.
Figure C.1 (continued).
Figure C.2  Suite of 2% in 50 years ground motions for representative soil profile, Memphis, TN.
Figure C.2 (continued).
Figure C.3 Suite of 10% in 50 years ground motions for bedrock (hard rock), Memphis, TN.
Figure C.3 (continued).
Figure C.4 Suite of 2% in 50 years ground motions for bedrock (hard rock), Memphis, TN.
Figure C.4 (continued).
Figure C.5 Suite of 10% in 50 years ground motions for representative soil profile, Carbondale, IL.
Figure C.5 (continued).
Figure C.6 Suite of 2% in 50 years ground motions for representative soil profile, Carbondale, IL.
Figure C.6 (continued)
Figure C.7 Suite of 10% in 50 years ground motions for bedrock (hard rock), Carbondale, IL.
Figure C.7 (continued).
Figure C.8  Suite of 2% in 50 years ground motions for bedrock (hard rock), Carbondale, IL.
Figure C.8 (continued).
Figure C.9  Suite of 10% in 50 years ground motions for representative soil profile, St. Louis, MO.
Figure C.9 (continued).
Figure C.10 Suite of 2% in 50 years ground motions for representative soil profile, St. Louis, MO.
Figure C.10 (continued).
Figure C.11 Suite of 10% in 50 years ground motions for bedrock (hard rock), St. Louis, MO.
Figure C.11 (continued).
Figure C.12 Suite of 2% in 50 years ground motions for bedrock (hard rock), St. Louis, MO.
Figure C.12 (continued).
References


Hashash, Y., “Typical Soil Profiles for Mid-America,” Personal Communication, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, May 3 1999.


Hwang, H., M.C. Chien and Y.W. Lin, Investigation of Soil Conditions in Memphis, Tennessee, Center for Earthquake Research and Information, the University of Memphis, July 1999.


Jennings, P.C., G.W. Housner, and N.C. Tsai, Simulated Earthquake Motions, Earthquake Engineering Research Laboratory, California Institute of Technology, April 1968.


Trifunac, M.D. and V.W. Lee, *Frequency Dependent Attenuation of Strong Earthquake Ground Motion*, University of Southern California, Department of Civil Engineering, Report No. CE 85-02, October 1985.


Yeh, C.H. and Y.K. Wen, Modeling of Nonstationary Earthquake Ground Motion and Biaxial and Torsional Response of Inelastic Structures, Civil Engineering Studies, Structural Research Series Report No. 546, Department of Civil Engineering, University of Illinois at Urbana-Champaign, August 1989.