I. INTRODUCTION

by I.D. Gupta[1] and M.I. Todorovska[2]

1. Background

For earthquake-resistant design of important engineered structures, like dams, nuclear power plants, long-span bridges, high-rise buildings etc., located in seismically active areas, it is imperative to have a very reliable estimate of the site-specific design motion. There are two basic philosophies in defining design motions: deterministic and probabilistic (largest possible versus most probable). The former proposes design for the "maximum earthquake", that is the one that will produce most severe ground motion at the site. The latter advocates that likelihood of occurrence should also be considered in view of the fact that the life of a structure is very short compared to the recurrence intervals of large events. Both of these analyses require a comprehensive database on seismicity, tectonics, geology and attenuation characteristics of the seismic waves in the area of the structure. In most of the cases in India, many of the required data are either completely lacking, or are far less than adequate. Further, they are generally not of uniformly good quality, and are associated with very large uncertainties. Because the generation of most such data is a very long term process, one has to make decisions with whatsoever data are available readily, or with data which could be collected by short term studies. Consequently, in many cases, such decisions are characterized by personal biases, in the form of expert opinions and engineering judgements, and may therefore be either too conservative or even unsafe in some cases. A rational objective way to account for the uncertainties due to lack of knowledge and poor quality of data is to describe the various quantities by suitable statistical distributions and obtain the resulting distributions for the ground motion parameters and the structural response. Then confidence can be assigned to predictions of site response or structural response (e.g., confidence that the structural response parameter will not exceed some level during its service time. This would provide a basis to evolve cost-effective design criteria by balancing the cost of hazard reduction against the consequences of failure and poor performance of a structure or a facility.

It has become customary in earthquake engineering and related areas to distinguish between earthquake hazard and earthquake risk, although the semantics of both words is the same. Earthquake hazard is used to describe severity of ground motion at a site, regardless of the consequences, while the definition of earthquake risk is based also on the consequences. High hazard does not automatically imply high risk. For example, the hazard may be high at a site close to an active fault, whereas the risk may not be high

---

[1] Chief Research Officer, Vibration Technology Division, Central Water & Power Research Station, Khadakwasla, Pune, India.

[2] Research Assistant Professor, Civil Engineering Department, Univ. of Southern California, Los Angeles, California.

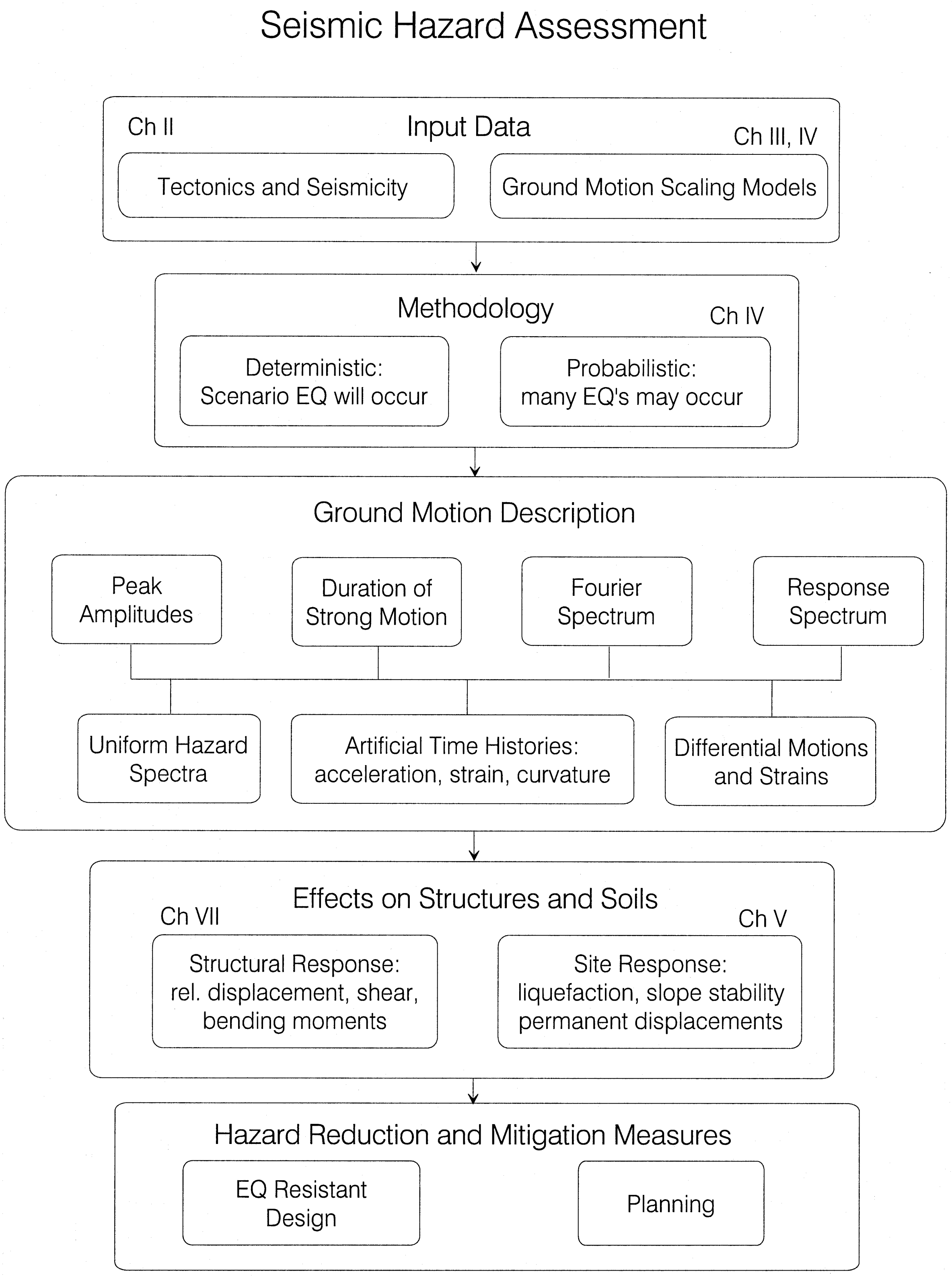
Seismic Hazard Assessment
Input Data
Ch II
Ch III, IV
Tectonics and Seismicity
Ground Motion Scaling Models
Methodology
Ch IV
Deterministic: Scenario EQ will occur
Probabilistic: many EQ's may occur
Ground Motion Description
Peak Amplitudes
Duration of Strong Motion
Fourier Spectrum
Response Spectrum
Uniform Hazard Spectra
Artificial Time Histories: acceleration, strain, curvature
Differential Motions and Strains
Effects on Structures and Soils
Ch VII
Ch V
Structural Response: rel. displacement, shear, bending moments
Site Response: liquefaction, slope stability permanent displacements
Hazard Reduction and Mitigation Measures
EQ Resistant Design
Planning

Fig. I.1.1

if there are no settlements and industrial facilities. Along the same line, the earthquake risk is large at the site of a critical facility, such as a dam or a nuclear power plant, even if these are not so close to active faults and even if there is not much evidence of historic earthquake activity (the eastern United States, and peninsular shield of India, for example). Considering the catastrophic consequences, the earthquake risk for an inadequately designed nuclear power plant is large even if the probability of a large magnitude earthquake occurring reasonably close to the plant is small. To be consistent with this terminology, we will use the term hazard when we refer only to ground motion or structural response with no regard for the consequences.

The various elements of seismic hazard analysis are illustrated in Fig I.1.1. From top to bottom, the first block contains the input data on tectonics and seismicity and on ground motion scaling models. The second block represents the methodology which may be characterized as deterministic (scenario earthquake) or probabilistic (an ensemble of earthquakes). The third block contains ground motion parameters for description of ground motion (e.g., peak amplitudes, artificial time histories, duration of shaking, Fourier and response spectra, differential motions ...). The fourth block corresponds to effects of earthquake shaking on structural response (displacement, shear and bending moment envelopes) and site response (liquefaction occurrence, slope stability, permanent displacements, ...). Finally, the fifth block corresponds to societal response, i.e. measures for seismic hazard reduction. The material in this report addresses most of the elements enclosed in blocks one to four. The remaining of this chapter discusses some important general aspects of probabilistic seismic hazard analysis, while Chapters II through VII describe in detail specific issues.

2. Probabilistic Seismic Hazard Analysis

The general methodology of probabilistic seismic hazard analysis involves integrating the probabilities of experiencing a particular level of a ground motion parameter due to all the different earthquakes expected to occur in the area of a structure during its estimated life period (Anderson and Trifunac, 1977; 1978). This in turn requires knowledge of various seismogenic sources in the area, the seismicity rate for each source zone, the geological and local soil conditions, and appropriate attenuation equations for various ground motion parameters of interest. As it is not always possible to define some or all of the factors unambiguously, one also needs to specify the various alternate possibilities or the probability distribution functions to take care of the uncertainties or randomness in each of the factors. In the following, we summarize the state of the art methodology applied to set up the various components needed for a probabilistic seismic hazard evaluation for a project site of interest.

(a) Seismic Sources

The first step in seismic hazard analysis is to characterize the various seismic sources in the area which may affect the site of a structure. The seismogenic sources are generally defined from the spatial distribution of past earthquakes, or from knowledge of various faults and lineaments in the area, or both. A source zone may be an idealized point source, a line source, a diffused area source, a volume source or a dipping plane (Lee and Trifunac, 1985).

The epicentral locations of most earthquakes before around early 1960's in India are either based on teleseismic recordings or on macroseismic observations, and hence are not accurate. In many cases, the locations may be in error by several tens of kilometers. Focal depths are either lacking or are just conjectural for many major historic earthquakes. Even for many recent earthquakes, recorded by regional seismological networks, the epicentral locations, and particularly the focal depths, have not improved to the desired level of accuracy due to poor knowledge of the crustal structure in different parts of the country. Therefore, the epicentral distribution of available earthquake data is generally characterized by very large scattering, and in many cases does not show definite correlation with the known tectonic features. Thus, it is very difficult to define the boundaries of the seismic sources, compatible with both the distribution of seismicity and the location of tectonic features.

Further, the fine structure of the Indian tectonic features, in general, has not been studied in detail, and is confined only to very specific localized areas. Even the results of these limited studies are not available readily most of the time. The most commonly available information even for the highly seismic Himalayan plate-boundary zone are the traces of the Himalayas foot-hill-thrusts (FHT), main boundary fault (MBF), and the main central thrust (MCT), each as a continuous unit traversing more than 2,000 km all along the Himalayas. Each of these mega-faults is locally consisting of a system of faults, the details of which are necessary for an accurate delineation of the seismic sources for the purpose of risk analysis. The Himalayan plate boundary is segmented into smaller units by several transverse faults. Each of these segments is perhaps characterized by different strain rates and the angles of underthrusting. All these details must be known in more detail for better definition of the seismic sources.

In the case of Peninsular Shield of India, the situation is further more complicated. The seismicity of this part of India is mostly at very low level, and is known to be confined to a few isolated areas. The available data on past earthquakes are very limited and are inadequate to decipher seismic sources with any confidence. Moderate to large magnitude earthquakes in these seismic zones perhaps occur with extremely long recurrence intervals. The seismic sources in peninsular India, far away from the Himalayan plate boundary, are more of localized nature and in most cases are identified only after the occurrence of a large earthquake. Thus, neither the available earthquake data nor the knowledge of tectonic features are sufficient to identify the seismic sources. Until detailed information on all the faults and weak zones becomes available, surprises like the Koyna and the Killari earthquakes will continue to happen in this part of the country.

(b) Seismicity

Next, it is necessary to define the seismicity expected to occur in each of the seismic sources during a specified period. By seismicity here we mean the expected rate of occurrence of earthquakes with different magnitudes. This is generally accomplished by studying the magnitude-frequency relation using the available data on past earthquakes (Gupta and Ramakrishna, 1984; Gupta, 1991). When data on earthquake occurrences are insufficient, the knowledge of slip rates on a fault can also be converted into expected seismicity (Todorovska and Jordanovski, 1994). In fact, both these sources of information complement each other and should be considered for better and more reliable estimation of seismicity. Unfortunately, in India, the slip rates are not adequately known for the major faults and also the available data, except for some parts in the highly seismic Himalayan region, are not adequate for direct evaluation of seismicity. Even when adequate data are available, it is not of uniform quality, and is usually associated with large uncertainties. The magnitudes of historic earthquakes are mainly defined from Modified Mercalli Intensity, and have not been calibrated in terms of instrumental measurements. The magnitudes of all major past earthquakes may be in error by 0.5 to 1.0 magnitude units. Also, due to the lack of precise calibration of the instruments, and uncertain description of the attenuation of seismic wave amplitudes with distance, magnitudes of even instrumentally recorded earthquakes may be at some variance with the exact values (see discussion on this subject with examples for south-eastern Europe, in Trifunac and Herak, 1992). Further, due to wide separation between the existing seismological observatories, data on small magnitude earthquakes are not reported completely. In many cases, this makes it difficult to define the frequency-magnitude relationships in an unbiased way. For areas where seismicity occurs frequently, short-term (2 to 5 years) micro-earthquake surveys may be helpful to fill the gap on small magnitude earthquakes. Also, the location of micro-earthquake events using a network of about 5 to 10 stations may be helpful to identify precisely some of the local seismic sources.

(c) Attenuation Characteristics

From teleseismic seismological records of an earthquake, one can get information on the earthquake magnitude and location only. To obtain ground motion parameters for engineering use (peak amplitudes, Fourier and response spectra, strong motion duration, seismic wave energy, etc.), it is necessary to have strong motion accelerogram records. Using such data from a large number of earthquakes in an area of interest, it is possible to derive empirically the attenuation characteristics of various ground motion parameters as functions of the source-to-site distance. Unfortunately, strong motion records in India can be considered to have just started to evolve with a mere 99 records available for the entire Himalayan zone (Chandrasekaran and Das, 1993). Earlier, some records have been obtained at the Koyna dam site for a highly localized area in the peniusular shield of India (Gupta et al., 1993). The available data are not sufficient to develop attenuation relations for the Indian conditions. Therefore, attenuation relations developed for other parts of the world are commonly used in various applications in India.

A comparison of the attenuation relationships for many different areas shows that the attenuation characteristics may differ significantly from one region to another due to differences in geological characteristics and seismic source properties. Indiscriminate use of attenuation relations from another region, as is commonly the case in India, may lead to biased results. For accurate evaluation of seismic hazards, it is therefore essential to have region-dependent attenuation relations based on strong motion accelerograph records for that region only. Until such data become available for different parts of India, one should use attenuation relations from other regions with caution. One possible way to select a representative attenuation relation for a particular region of India may be to use a relation from another region with similar attenuation characteristics for Modified Mercalli Intensity with distance. Alternately, one may use the probabilistic relations for the attenuation of MMI with distance in India (Gupta and Trifunac, 1988) and apply scaling equations for strong motion parameters in terms of site intensity determined for another region which has similar definition of intensity. Recent comparisons of strong motion characteristics in California with those in former Yugoslavia, and especially the observed differences in the attenuation laws (Trifunac et al., 1991; Trifunac and Lee, 1990; Trifunac and Živčić, 1991; Lee and Trifunac 1992; Todorovska et al., 1995ab; Lee et al., 1990) suggest that careful analyses and comparisons of the differences in geologic structure and age should be carried out, before some "foreign" attenuation laws are used for estimation of strong motion amplitudes in India.

3. Seismic Zoning

The foregoing description of seismic hazard evaluation refers to a single site, which may be the site of an existing structure whose seismic safety has to be ascertained, or the site of an important prospective structure. To prepare a seismic zoning map for general usage, the same methodology can be applied to a grid of closely spaced sites, and zones of equal seismic hazard can then be contoured (Trifunac, 1990; Lee and Trifunac, 1987).

The seismic zoning map of the Indian Standards code for aseismic design of structures (IS: 1893–1984) is not based on this type of analysis. It is based on the observed intensities at different locations during major past earthquakes. For regions where earthquake data are not available, some weight has been assigned to the known tectonic features to shape the geometry and the extent of various seismic zones. Thus, this map can at best be considered as a scenario map and does not assign any probability or time period to the levels of shaking. Some individual studies have also presented seismic zoning maps of India or of some parts of India. Many such early studies had adopted the same principles as followed in the preparation of the map of the Indian code. Some later studies, which have attempted to use the probabilistic hazard analysis approach, are based on very small number of data and on approximate attenuation relations. All of these define the ground motion in terms of peak acceleration, which is not a suitable parameter for relating to seismic response of intermediate and long period structures (Trifunac, 1992). Also, these studies have not accounted for various uncertainties in the input parameters in a realistic way, and hence can be considered only to be of academic interest, and not for actual applications.

4. What Ground Motion Parameters to Consider for Hazard Mapping?

Ground shaking at a site can be described by a variety of parameters. Before instrumental measurements of strong ground motion became available, various intensity scales (MMI, MKS, etc.), based on the description of observed damage, were used to describe the severity of ground motion. Intensity data are (and should be) still used as a supplement to instrumental data. More recently, the peak ground acceleration and, to a much lesser extent, the peak velocity and displacements have been popular instrumental measurements of ground motion. Most of the existing code provisions and design procedures have been developed in terms of peak acceleration or some equivalent (e.g., design seismic coefficient).

Three-component acceleration time histories represent by far the most comprehensive and complete description of earthquake ground motion at a site. But such records are not always available for all the required conditions of earthquake parameters, geological conditions and source-to-site path characteristics. Even if a desired accelerogram for a site were available, it cannot be used directly in design applications due to large and uncertain fluctuations in the frequency content of ground motion recorded under similar conditions. In practical applications, these fluctuations are averaged out, and amplitudes corresponding to a higher confidence level are normally used. There are no general methods to perform such averaging directly on the time histories. The process of smoothing random fluctuations is performed on the Fourier amplitude spectra or response spectra, which are then inverted back to a time history (Wong and Trifunac, 1979; Lee and Trifunac, 1989; Gupta and Joshi, 1993). The Fourier and response spectra also provide means to study the effects of various earthquake, site, and path parameters on the ground motion. Thus, the description of ground motion in terms of properly scaled Fourier and response spectra along with some representative recorded time histories can be considered most useful for practical applications.

In most routine design applications, the input is commonly specified in the form of a normalized standard spectral shape to be scaled by peak ground acceleration. The problem in using peak acceleration for this purpose is that it is not related well to structural response and earthquake damage, that it is a highly unstable parameter, and that it is not consistent with earthquake magnitude and distance. Further, a standard spectral shape is not able to take into account the fact that earthquakes with different magnitudes have different predominant frequency content, and that different frequency waves attenuate differently with distance. Therefore, to properly include the effects of earthquake magnitude and distance on the spectral shape, and to do away with the peak acceleration scaling, one should directly define the spectral amplitudes at different frequencies by using frequency-dependent scaling equations for the spectral amplitudes (Trifunac and Lee, 1987). For the seismic zoning, one should thus prepare a separate zoning map in terms of response spectrum amplitude at each frequency (Trifunac, 1990).

The commonly used response spectrum superposition methods in design applications give the maximum elastic response of multi-degree-of-freedom structures. But, under severe seismic excitations, most structures will experience response well into the inelastic range. For actual non-linear response calculations, it is necessary to specify the ground motion in terms of complete acceleration time histories of ground motion, known as design accelerograms. Design accelerograms can be synthesized from a given elastic response spectrum or from the Fourier spectrum of ground motion (Wong and Trifunac, 1979). Thus, it is also useful to consider the Fourier spectrum amplitudes at different frequencies for the purpose of seismic zoning.

Some response analyses are also based on equivalent linearization method, where the effects of non-linearity are considered by scaling down the elastic design response spectrum in terms of specified ductility values (Vidić et al., 1994). A proper way to carry out this task is by finding reduction factors for different frequencies based on the severity of ground motion compatible with the response spectrum. However, the response spectra given in the existing Indian Standards code have latently included the reduction for inelastic behavior, which is uniform over the entire frequency range. This may underestimate the strength demands for very high frequency structures. In general, the inelastic strength demand may be much lower than the elastic strength demand, and a design engineer has to clearly understand which strength he is referring to. Because the Indian code spectra have been reduced for inelastic behavior, the corresponding permissible stresses in reality refer to the yield stresses, and those should not be confused with the actual design strength. If site-specific design spectra are considered for the response analysis of a structure, the corresponding permissible stresses should also be taken equal to the design strength.

5. Soil-Structure Interaction Effects

The actual input motion for analysis of response of structures is different from the motion of the ground, due to interaction between the structure, the foundation and the soil. The change in the motion due to interaction is usually represented as a perturbation to be added to the motion of the ground in the absence of the foundation and the structure (Todorovska, 1993a). The motion of the ground without foundation and without excavation for the foundation is referred to as the free-field motion. The motion of the ground with excavation (or with massless foundation) is referred to as the foundation input motion. If the mass of the foundation is small compared to the mass of the structure so that the inertia forces of the foundation can be neglected, then the foundation input motion is the actual motion exciting the structure (Todorovska, 1993b).

The effects of the soil-structure interaction can be classified as kinematic or as dynamic. The kinematic effects are due to scattering of waves from the foundation (Todorovska, 1993a), and depend on the geometry and depth of the foundation (Todorovska, 1992ab), and on the type of incident waves, direction of approach, and wavelength (compared to the size of the foundation). In a simple way, those can be described as follows. Large and stiff foundations (much more stiff than the soil and moving much like a rigid body) filter out short wavelength components of the input motion, so that the resultant translations are smaller in amplitude. There is no fixed boundary between short and long wavelengths. For example, an incident wave can be considered as short if its wavelength is shorter than twice the width of the foundation. The reduction of amplitudes is larger for deeper foundations, as those are more efficient scatterers of waves. The effects of the kinematic interaction depend on the shear and compressional wave velocities of the soil as, for given frequency, the wavelength depends on these velocities. There is another kinematic effect that is worth noting. For longer incident waves (about four times the width of the foundation), the foundation may respond in rocking even if the free-field motion has zero point rotation. This occurs for embedded foundations as a result of a particular phase difference of the incident motion along the contact boundary with the soil. This effect changes the nature of the input motion to the structure, as the structure is actually subjected also to rocking motion in addition to the translations. This effect is usually neglected when the soil-structure interaction effects are accounted for in a simplistic manner.

The structure, foundation and soil act as a system with its own dynamic characteristics (Todorovska, 1992a; Todorovska and Trifunac, 1992). The system characteristics (system frequencies and system damping) can be obtained from models of the dynamic interaction between the elements in the system, or experimentally from ambient or forced vibration measurements (Trifunac, 1972; Luco et al., 1986; Moslem and Trifunac, 1986; Ivanovic and Trifunac, 1995). The mathematical models are based on balance of inertia forces and feedback forces from the soil in response to deformations by the moving foundation. Complex stiffness matrix of the foundation is evaluated as a function of frequency (wavelength) of motion. The components of this matrix are dynamic feedback forces and moments from the soil for unit translations and rotations of the foundation. The real part of the entries are the foundation stiffness coefficients (dynamic). From the imaginary part, damping coefficients are evaluated. The dissipation mechanism of this "damping" is radiation of wave energy into the semi-infinite soil medium. Flexible foundation medium "softens" the system. The fundamental mode is most affected both by the "damping" and by the change in stiffness. The first system frequency is always smaller than the first fixed-base frequency. The shift is greater if the soil is softer. The shift of the frequencies of the higher modes is usually small. Due to the additional "damping", the amplitude of the first mode is usually reduced.

The soil-structure interaction effects have not been incorporated yet into the building codes. There is a tendency, however, to assume lower design loads due to these effects.

6. Response Analysis of Structures

Even after carrying out a probabilistic seismic hazard study and defining the ground motion with an appropriate confidence level to account for various uncertainties, the inherent random nature of ground motion in the resulting structural response is not taken into account. The seismic waves reach a site after multiple reflections and diffractions, and hence the ground motion records obtained even under apparently identical conditions, differ in their non-repeatable features. In simulation of synthetic accelerograms, this can be simulated via random phases uniformly distributed between $-\pi$ and π in addition to the phases associated with source-to-site propagation effects (Wong and Trifunac, 1979). Hence, the probabilistic nature of structural response can be considered in two parts. The first part is formulation of the probability distribution function on account of the random nature of earthquake parameters and scattering due to not explicitly considering the effects of some of the unknown parameters and even of some known parameters due to lack of data. The second part is to give an appropriate probabilistic description to the structural response due to random nature of ground motion for fixed values of all the governing parameters of ground motion. The standard deviations of structural response amplitudes on account of the random nature of structural properties are usually much smaller than those caused by random scattering in the input ground motion. The present report describes the aspects of the random nature both of ground motion and of structural response. Though the effects of the random nature of structural response are illustrated only for simple hypothetical structures, the conclusions drawn apply to the real structures as well.

7. Organization of this Report

This report is organized in seven chapters, following this chapter. Chapter II describes the major tectonic features responsible for seismic activity in India, with reference to plate tectonics. This information is necessary to define various seismic sources and their seismicity for seismic hazard analyses.

Chapter III presents frequency-dependent attenuation relationships for Pseudo Velocity Spectrum amplitudes in the California area. A brief history of the development of these relations is also discussed. Such relations provide a basis to evaluate the probability of exceeding a given spectral amplitude due to specified values of earthquake magnitude and source-to-site distance, which forms an essential component for probabilistic seismic hazard studies.

Chapter IV gives the formulation for seismic hazard computations. A detailed description is presented on evaluating the input seismicity from historic data as well as from moment rates on different faults. To demonstrate the methodology and to show the effects of various parameters, example results on Pseudo Velocity Response spectra and Fourier amplitude spectra are presented for the California area and the north-east Indian region.

Chapter V first reviews the history of seismic zoning studies in India and then presents several examples of microzoning in terms of Pseudo Velocity spectrum amplitudes, peak strains and liquefaction opportunity for the Los Angeles metropolitan area in southern California.

Chapter VI describes in detail, with examples, the methodology of synthesizing time-histories of strong earthquake ground motion at a site for given earthquake parameters and site geological conditions. Methodology is presented to obtain translational and rotational components of ground acceleration, strain time histories and curvograms of ground motion.

Chapter VII presents methodology for stochastic seismic response analysis of structures subjected to random input excitation. A formulation is presented to compute the amplitudes of all the significant peaks in the response of MDOF structures by application of order statistics. The effects of soil-structure interaction are also considered.

8. References

[1] Anderson, J.G. & M.D. Trifunac (1977). On uniform risk functionals which describe strong earthquake ground motion: definition, numerical estimation and an application to the Fourier amplitude of acceleration, Rep. No. CE 77-02, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[2] Anderson, J.G. & M.D. Trifunac (1978). Uniform risk functionals for characterization of strong earthquake ground motion, *Bull. Seism. Soc. Amer.*, **68**, 205–218.

[3] Chandrasekaran, A.R. & J.D. Das (1993). Strong earthquake ground motion data in EQINFOS for India: part 1B, edited by M.D. Trifunac, M.I. Todorovska and V.W. Lee, Rep. No. CE 93-04, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[4] Gupta, I.D. & T.V.S. Ramkrishna (1984). Evaluation of risk Fourier spectra for a site in Assam seismic gap, Northeast India, *Bull. Ind. Soc. Earthq. Tech.*, **21**, 62–72.

[5] Gupta, I.D. & M.D. Trifunac (1988). Attenuation of intensity with epicentral distance in India, *Soil Dynam. & Earthqu. Engng*, **7**(3), 162–169.

[6] Gupta, I.D. (1991). A note on computing uniform risk spectra from intensity data on earthquake occurrence, *Soil Dynam. & Earthqu. Engng*, **10**(8), 407–413.

[7] Gupta, I.D. & R.G. Joshi (1993). On synthesizing response spectrum compatible accelerograms, *Europ. Earthqu. Engng*, **6**(2), 25–33.

[8] Gupta, I.D., V. Rambabu & R.G. Joshi (1993). Strong earthquake ground motion data in EQINFOS for India: Part 1A, edited by M.D. Trifunac, M.I. Todorovska and V.W. Lee, Rep. No. CE 93-03, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[9] IS:1893-1984. Indian standard recommendations for earthquake resistant design of structures, Indian Standards Institute (Now, Bureau of Indian Standards); Delhi.

[10] Ivanovic, S.S. & M.D. Trifunac (1995). Ambient vibration survey of full scale structures using personal computers (with examples in Kaprielian Hall), Rep. No. 95-05, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[11] Lee, V.W. & M.D. Trifunac (1985). Uniform risk spectra of strong earthquake ground motion, Rep. No. CE 80-05, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[12] Lee, V.W. & M.D. Trifunac (1987). Microzonation of a metropolitan area, Rep. No. CE 87-02, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[13] Lee, V.W. & M.D. Trifunac (1989). A note on filtering strong motion accelerograms to produce response spectra of specified shape and amplitude, *Europ. Earthqu. Engng*, **2**, 38–45.

[14] Lee, V.W., M.D. Trifunac, D. Herak, M. Herak & M. Živčić (1990). M_L^{SM} in Yugoslavia, *Earthqu. Engng & Struct. Dynam.*, **19**(8), 1167–1179.

[15] Lee, V.W. & M.D. Trifunac (1992). Frequency dependent attenuation of strong earthquake ground motion in Yugoslavia, *Europ. Earthqu. Engng*, Vol. VI, n.1), 3–13.

[16] Luco, J.E., M.D. Trifunac & H.L. Wong (1986). Soil structure interaction effects on forced vibration tests, Rep. No. CE 86-05, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[17] Moslem, K. & M.D. Trifunac (1986). Effects of soil structure interaction on the response of building during strong earthquake ground motions, Rep. No. CE 86-04, Dept. of Civil Engng, Univ. of Southern California, Los Angeles, California.

[18] Todorovska, M.I. (1992a). Effect of the depth of the embedment on the system response during building-soil interaction, *Soil Dynam. & Earthqu. Engng*, **11**(2), 111–123.

[19] Todorovska, M.I. (1992b). Radiation damping during two-dimensional building soil interaction, *Proc. 10th World Conf. Earthq. Eng.*, 19-24 July, 1992, Madrid, Spain, 1549-1554.

[20] Todorovska, M.I. (1993a). In-plane foundation-soil interaction for embedded circular foundations, *Soil Dynam. & Earthqu. Engng.*, **12**(5), 283–297.

[21] Todorovska, M.I. (1993b). Effects of the wave passage and the embedment depth during building-soil interaction, *Soil Dynam. & Earthqu. Engng.*, **12**(6), 343–355.

[22] Todorovska, M.I. & L.R. Jordanovski (1994). A probabilistic model for assessment of the total earthquake losses for a building, *Proc. 10th Europ. Conf. Earthq. Eng.*, Aug. 28- Sept. 2, 1994, Vienna, Austria, 1017–1022.

[23] Todorovska, M.I. & M.D. Trifunac (1992). The system damping, the system frequency and the system response peak amplitudes during in-plane building soil interaction, *Earthqu. Engng & Struct. Dynam.*, **21**(2), 127–144.

[24] Todorovska, M.I., E.I. Novikova & M.D. Trifunac (1995a). Frequency dependent duration of strong earthquake ground motion on the territory of Former Yugoslavia, Part I: magnitude models, *Europ. Earthqu. Engng.*, Vol. VIII, 11–25.

[25] Todorovska, M.I., E.I. Novikova & M.D. Trifunac (1995b). Frequency dependent duration of strong earthquake ground motion on the territory of Former Yugoslavia, Part II: local Intensity models, *Europ. Earthqu. Engng.*, Vol. VIII, 26–37.

[26] Trifunac, M.D. (1972). Comparison between ambient and forced vibration experiments, *Earthqu. Engng & Struct. Dynam.*, **1**, 133–150.

[27] Trifunac, M.D. (1990). A microzonation method based on uniform risk spectra, *Soil Dynam. & Earthqu. Engng.*, **9**(1), 34–43.

[28] Trifunac, M.D. (1992). Should peak accelerations be used to scale design spectrum amplitudes? *Proc. 10th World Conf. on Earthq. Eng.*, Madrid, Spain, **10**, 5817–5822.

[29] Trifunac, M.D. & V.W. Lee (1987). Direct empirical scaling of response spectral amplitudes from various site and earthquake parameters, Report NUREG/CR-4903, Vol. 1, U.S. Nuclear Regulatory Commission, Washington, DC.

[30] Trifunac, M.D. & V.W. Lee (1990). Frequency dependent attenuation of strong earthquake ground motion, *Int. J. Soil Dynam. & Earthqu. Engng*, **9**(1), 3–15.

[31] Trifunac, M.D. & M. Živčić (1991). A note on instrumental comparison of the Modified Mercalli Intensity (MMI) in the Western United States and the Mercalli-Cancani-Sieberg (MCS) Intensity in Yugoslavia, *Europ. Earthqu. Engng*, Vol. V, n.1, 22–26.

[32] Trifunac, M.D., V.W. Lee, M. Živčić & M. Manić (1991). On the correlation of Mercalli-Cancani-Sieberg Intensity Scale in Yugoslavia with the peaks of recorded strong earthquake ground motion, *Europ. Earthqu. Engng*, Vol. V, n.1, 27–33.

[33] Trifunac, M.D. & D. Herak (1992). Relationship of M_L^{SM} and magnitudes determined by regional seismological stations in Southeastern and Central Europe, *Soil Dynam. & Earthqu. Engng*, **11**(4), 229–241.

[34] Vidić, T., P. Fajfar & M. Fischinger (1994). Consistent inelastic design spectra: strength and displacement, *Earthq. Engng & Struct. Dynam.*, **23**(5), 507–522.

[35] Wong, H.L. & M.D. Trifunac (1979). Generation of artificial strong motion accelerograms, *Earthqu. Engng & Struct. Dynam.*, **7**, 509–527.